MEASURABLE PARAMETERS FOR PERFORMANCE OF CORRODED AND REPAIRED RC BEAMS UNDER LOAD

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A thesis submitted to the Faculty of Engineering, University of Cape Town in fulfilment of the requirements for the degree of Doctor of Philosophy.

Cape Town, 2010.
I declare that this thesis is essentially my own work and is being submitted for the degree of Doctor of Philosophy at the University of Cape Town. It has not been submitted before for any degree or examination in any other university.

Signed

On _______ day of __________________________
ABSTRACT

Structural engineers and asset managers rely on measurable parameters developed by researchers to predict residual load-bearing capacities of corroding in-service RC structures as well as to assess repair-effectiveness. Unfortunately, laboratory research that was used to develop these measurable parameters varied between researchers and in most cases, did not represent in-service conditions. As a result, they found different relations between measurable and non-measurable parameters which are unsafe and/or engineers find difficult to apply to in-service structures. A holistic research that emulates in-service conditions was therefore developed. Its objectives were to provide structural engineers and asset managers with more-useful measurable parameters for performance of corroding and repaired RC structures. Parameters that were looked at were corrosion crack widths and longitudinal strains together with their derivatives.

To best represent in-service conditions, two sequential corrosion processes were used; firstly accelerated corrosion by impressing an anodic current followed by natural steel corrosion, mostly after repairs. Accelerated corrosion was divided into two processes; one with four-day wetting cycles with salt solution followed by two-day drying cycles and another where four-day drying cycles were used instead. Large mass losses of steel were found when long drying cycles were used. Since it is more representative of in-service conditions, it was recommended that laboratory tests on steel corrosion should entail long drying cycles. Natural steel corrosion after FRP repairs had a maximum rate of 66 µA/cm². However, no further corrosion was found after patch repairs.

Load-bearing capacity reduced with an increase in the level of steel corrosion. It was related to maximum mass loss of steel instead of average mass loss. It was shown that 1% mass loss of steel reduced load-bearing capacity by 0.8%. Patch repairs did not significantly increase the capacity of corroded beams. However, a significant increase (up to 50%) in load-bearing capacity was recorded after FRP repairs. From results on natural steel corrosion and load-bearing capacity of repaired beams, it was recommended that repair of corroded RC structures should entail combined patch and FRP repairs.
During corrosion, longitudinal strains were found to be mostly related to corrosion crack patterns rather than the level of steel corrosion. After repairs, they either did not vary or their variation was deceiving. Therefore, structural engineers cannot use them to predict residual load-bearing capacities of corroding RC structures or to assess repair-effectiveness.

The depth of the neutral axis from longitudinal strains was found not to sufficiently vary with increased corrosion. It was concluded that it is a poor parameter and structural engineers should avoid measuring it. Flexural stiffness only varied at levels of steel corrosion below 3.8%. Similar to longitudinal strains, after repairs it did not significantly vary or its variation was deceiving. Structural engineers were advised to use it only at low levels of steel corrosion.

Corrosion crack widths varied sufficiently with increase in corrosion level to be used as indicators of residual load-bearing capacity of corroding RC structures. Even prior to cover cracking, adequate expansion of the cover concrete was recorded and used to calibrate an analytical model for time to cover cracking. To be conservative, it was shown that a crack width of 1 mm corresponds to mass loss of steel of 8%. However, application of this relation requires some understanding of the relation between crack patterns and the rate of widening of corrosion cracks. For example, when a structure exhibits corrosion cracks on two nearing faces, an equivalent crack should be taken as the sum of the two cracks. This was because the first crack stops widening when the second one develops.

FRP repairs did not have an effect on the rate of widening of corrosion cracks on beams that exhibited side cracks. Beams that had uncracked side faces however, developed new cracks after repair which then widened at the same rate as those found in non-repaired beams. Therefore, rather than controlling cracking of the cover concrete, FRP repairs changed the pattern of corrosion cracks. It was again recommended that repair of corroded RC structures with FRPs should be carried out subsequent to patch repairs.
DEDICATION

This thesis is dedicated, with love, to my daughter Happy
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CHAPTER ONE

INTRODUCTION

1.1 Background

The majority of durability problems that face RC structures, particularly within the marine environment emanate from corrosion of embedded reinforcing steel. Concrete normally has an alkaline environment that protects the steel from corroding. This environment can be destroyed by carbonation or by chloride attack. Once the alkaline environment inside the concrete is destroyed and corrosion agents such as oxygen and moisture are available, corrosion of steel may commence. Resulting corrosion products are voluminous in nature compared to the parent metal so that their continued production applies tensile stresses on the surrounding concrete which can eventually lead to cracking of the cover concrete. Steel corrosion therefore harms the durability and load-bearing capacity of RC structures by reducing the area of steel as well as cracking the cover concrete. For those with an interest in the service life of these structures such as researchers, structural engineers and asset managers, it is important to identify criteria that could be used to define end-of-service life. It is even more important to initiate measures to repair corroding RC structures, and further identify criteria to measure repair-effectiveness.

The most common of these criteria are the time to depassivation of steel [1,2]; cracking of cover concrete [3-5]; and the point at which residual load-bearing capacity of a structure is deemed unsafe [6-8]. For structures that are already-corroding, the criterion of depassivation of steel is of little value. Crack widths and residual load-bearing capacities are therefore often used to define end-of-service life of such structures. Based on results by Alonso et al. [3] and by Andrade and Alonso [9] and studies by Torres-Acosta et al. [8] and O’Flaherty et al. [10] it can be shown that the level of steel corrosion that causes first concrete cover cracking, has a minimal impact on the load-bearing capacity of a structure. Thus, end-of-service life of a corroding RC structure based on the criterion of
residual load-bearing capacity of the structure extends well beyond the end-of-service life based on the criterion of crack widths. This helps to explain why structural collapse due to steel corrosion is rare, and yet there are numerous in-service structures that are severely damaged by steel corrosion. Researchers have illustrated this concept using Figure 1.1 [2,4,5,8]. The free expansion period in the figure corresponds to the time after depassivation of steel when corrosion products must firstly fill voids in concrete. During this time, there is no pressure applied on the surrounding concrete. The stress-build up stage corresponds to the time when the voids in the cover concrete have been fully-filled by corrosion products but the cover concrete is still uncracked. More discussion on these concepts is in Chapters Two and Six.

Repair of corroded RC structures is commonly undertaken to maintain or restore the serviceability of the structures [1,11]. From the above discussion, repair-effectiveness can be evaluated using the criteria of either corrosion crack widths, or residual load-bearing capacity. Interestingly, all these criteria are closely related to the level of steel corrosion. It is therefore also important to evaluate the effectiveness of repairs to extend service life of corroded structures based on their ability to control further steel corrosion.

According to Mays [11] and Rio et al. [12], once cracking of the cover concrete due to steel corrosion has occurred, the serviceability state of the structure can be restored by patch repairing it. Other measures to control further corrosion (especially due to varying electrochemical properties between the substrate concrete and the repair material) such as the application of corrosion inhibitors, cathodic protection and chloride extraction are at times undertaken at this stage. From Figure 1.1, if all these repair steps are precisely carried out at the time of appearance of the first visible corrosion crack, the service life of a structure is expected to be refurbished to near its original state. If on the other hand, the level of corrosion is such that the residual load-bearing capacity of the structure is reduced substantially, patch repair process may deter further steel corrosion but it will not enhance its load-bearing capacity [12,13]. Amongst various strengthening techniques in the literature as well as in practice, fibre reinforced polymers (FRPs)
Figure 1.1 Service life of corroding RC structures [2,4,5,8]
bonded to external faces of damaged RC structures has recently emerged as the state-of-the-art strengthening technique [14-17]. This is primarily because FRPs have superior advantages (such as handling, durability and improved mechanical properties) over the previous methods. They however, have a poor fire resistance.

Previous work on strengthening corroded RC structures with FRPs has often excluded the patch repair process [18-22]. Studies by Badawi and Soudki [18] and by Gadve et al. [21] have however, indicated evidence of continued steel corrosion after strengthening corroded RC structures with FRPs. In terms of the load-bearing capacity, Badawi and Soudki [18]; El Maaddawy and Soudki [19]; Soudki and Sherwood [20]; and Al-Saidy et al. [22] have shown that FRP strengthening without patch repairs of corroded RC structures is still able to improve the original load-bearing capacity of corroded structures. With an increased load-bearing capacity after FRP strengthening, the load on a strengthened structure is likely to be increased [16,17]. Little work has however, been done on the effectiveness of repairs to resolve the durability problems of RC structures that are corroded and repaired whilst under a sustained load, especially when steel is allowed to corrode naturally.

1.2 Statement of the problem

Structural engineers and asset managers often rely on measurable parameters of corroding RC structures, such as corrosion crack widths, to predict their residual load-bearing capacities. This involves using relations developed by researchers such as one indicated by Figure 1.1. Regrettably, researchers only agree that as steel corrosion continues, corrosion crack widths widen and load-bearing capacities of structures reduce. They however, provide different relations between corrosion crack widths and load-bearing capacities. Bear in mind that Figure 1.1 was drawn based on results where specimens were corroded under laboratory conditions and in the absence of a sustained load. If researchers find it difficult to provide accurate relations between measurable parameters of laboratory specimens (corroded under controlled conditions) and their residual load-bearing capacities, then most certainly, structural engineers are faced with an even bigger problem of applying those relations to in-service structures. Their problems are
compounded when they have to design for repairs that would not aggravate owners of structures by premature failure, or being costly, or even worse, both. It is therefore appropriate for further research to be carried out to, firstly, understand behaviour of corroding RC structures under simulated in-service conditions, and secondly, study the effectiveness of repairs to restore the service life of corroded RC structures also under simulated in-service conditions.

1.3 Thesis objectives

The thesis has two principal objectives;

1. To develop relations between various measurable parameters of RC structures corroding under load with their residual load-bearing capacities. Most importantly, these relations should be relatively easy for structural engineers as well as asset managers to apply them to in-service structures.

2. To develop measurable parameters of repaired RC structures that can be used to assess effectiveness of repairs carried out under load to increase their service life. In addition, to provide best repair practice that can be used to avoid further structural deterioration after repair.

Understandably, measurable parameters of corroding RC structures should be similar to criteria that are used to indicate their end-of-service life. Therefore, one of the measurable parameters that will be looked at in this research is cracking of the cover concrete. Another parameter that will be assessed here is longitudinal strains and their derivatives such as the depth of the neutral axis and stiffness. These parameters will also be used to assess repair effectiveness.

Figure 1.2 shows a time chart of the objectives of the research. As already-mentioned, the figure indicates loss in load-bearing capacity of a RC structure during corrosion propagation. The intention of the research is to associate this loss in load-bearing capacity with measurable parameters of the structure. After repair, the research firstly examines the ability of repairs to increase the load-bearing capacity that was reduced by steel corrosion. Most importantly, it then looks into measurable parameters that can be used to assess performance of repairs.
1.4 Scope of the thesis

As already mentioned, this thesis provides measurable parameters that can be used to predict load-bearing capacities of corroding RC structures. Information on performance of repaired structures using similar parameters as for the non-repaired structures is also presented. This information is meant to help structural engineers who need to understand well when to carry out repairs on corroding RC beams and how to assess performance of repairs.
A holistic experimental research that emulates in-service conditions as well as an extensive review of previous research serves as the basis for this thesis. It includes experimental results and analytical models. As will be shown in subsequent chapters, the majority of previous research had limitations that made it difficult to use most of their results. The bulk of results used in this thesis are from experimental research that was conducted in this research. Some of the results and analytical models provided have therefore, not been thoroughly proven on in-service structures. They however, provide a basis for understanding behaviour of corroding and repaired RC beams. The following point out broad areas looked at in this thesis that still requires research. Specific areas are outlined in Chapter 7.

Despite some phases of natural steel corrosion, the majority of results used in this research were from accelerated steel corrosion. This is because natural steel corrosion is a slow process which requires tens of years to produce desired structural damage. However, the level of impressed current density used followed guidelines from previous researchers to avoid un-proportional structural damage. These results therefore need to be proven on in-service structures.

Results and analytical models given in this thesis are from research conducted on quasi-full-scale RC beams. While the majority of them such as time of cover cracking and corrosion crack widths are likely to apply to other structural elements such as RC columns and RC slabs, this has not been validated. Caution is advised when structural engineers have to assess those structural elements. Caution is also advised when structural engineers assess pre-stressed concrete structures where structural collapse due to steel corrosion might occur at narrow corrosion cracks.

Corrosion of steel that is embedded in concrete is mainly caused by two different processes, carbonation attack and chloride attack. Response of concrete structures to each type of corrosion is different. This research is limited to corrosion due to chloride attack. Further research is needed to develop measurable parameters of RC structures that can be used to predict their performance due to carbonation.
This thesis provides new information on behaviour of RC beams that corrode and are repaired under load. Despite a few instances where applied loads were increased or reduced, the majority of results are from constant sustained loads. They may therefore underestimate corrosion damage as well as overestimate performance of repairs on RC structures with excessively varying loads. It is envisioned that future research will look into corrosion and repair of RC structures under cyclic loads. Results here provide a basis for such research.

1.5 Outline of the thesis document

Chapter Two reviews and provides a critical discussion from the literature on behaviour of corroded and repaired RC structures. In this chapter, potential measurable parameters on a corroding RC structure that can be used to predict its non-measurable parameters as well as its end-of-service life are identified. Parameters after repairs of corroded structures to measure their effectiveness are also looked at.

Chapter Three outlines a research that was designed to extensively assess the performance of RC beams that were corroded under various levels of sustained service loads. Probably most important, it describes a test set-up that was specifically designed to allow for patch and FRP repairs to be carried out on corroded RC beams under load. In addition to experimental tests carried out, the chapter discusses a detailed monitoring system that would provide in-depth information on the performance of RC specimens during corrosion as well as after repair.

Chapter Four discusses relations between the level of steel corrosion and load-bearing capacity of RC beams. It also looks into the ability of patch and FRP repairs under load, to prevent further steel corrosion as well as increase the load-bearing capacity of corroded RC beams.

Chapter Five gives a detailed discussion on the variation of longitudinal strains and their derivatives on RC beams that were corroded and repaired with patch
mortars and FRPs under sustained service loads. It firstly examines the potential of using the variation of longitudinal strains and their derivatives such as stiffness and the depth of the neutral axis of corroding RC structures as measurable parameters to indicate the level of steel corrosion. The chapter then discusses the potential of using them to measure the effectiveness of patch and FRP repairs under sustained service loads to increase the service life of corroded RC structures.

Chapter Six presents results and a discussion on time of first cracking of the cover concrete. It then examines the potential of relating the rate of widening of corrosion cracks with the rate of steel corrosion. It also looks into the ability of repairs (carried out under load) to control cover cracking.

A summary of research findings in a simplified form that is useful to structural engineers is given in Chapter Seven.

Appendix A presents values obtained in the literature on relations between actual mass loss of steel with predicted mass loss from Faraday’s Law. A summary of conditions of the experiments used by researchers is also provided.

Appendix B lists journal papers that were published on this research work.

The structure of this thesis is such that references are provided at the end of each chapter. Appendix C presents a list of all references that were used in the thesis.

1.6 Terminology

The following definitions clarify terms used in this thesis that are either not commonly used in RC practice or their use here does not necessarily apply to normal use.

**Accelerated steel corrosion** – Steel corrosion at a faster-than-normal rate by subjecting steel to anodic current.
Active corrosion cracks - Those corrosion cracks that widen with continued steel corrosion.

Anode - The electrode in electrolysis where steel corrosion occurs.

Bar spacing - The distance between parallel reinforcing bars, measured centre-to-centre of the bars perpendicular to their longitudinal axes.

Bond strength - Resistance to separation of a repair from substrate concrete.

Bonding agent - A material applied to a suitable substrate to enhance bond between it and a succeeding layer.

Cathode - The electrode at which chemical reduction occurs.

Cathodic protection - A form of corrosion protection for reinforced concrete wherein a sacrificial metal is caused to corrode in preference to the reinforcement, thereby protecting the reinforcement from corrosion.

Chloride content - Total amount of chloride ion present in concrete or mortar.

Chloride threshold - The amount of chloride required to initiate steel corrosion expressed in percent of chloride ion by mass of cement.

Composite - A combination of two or more constituent materials such as a substrate concrete and a repair.

Concrete substrate – Normally the existing concrete. However, when FRP repair follows patch repair then it is a combination of existing concrete and patch repair.

Contaminated face – Face of a RC structure where corrosion agents ingress the concrete.
**Corrosion** - Destruction of metal by a chemical, electrochemical, or electrolytic reaction within its environment.

**Corrosion agents** – Compounds such as water and oxygen that promote corrosion.

**Corrosion crack map** – A sketch of the surface of concrete showing the pattern corrosion cracks. Often includes crack widths.

**Corrosion crack patterns** – Different ways in which corrosion cracks propagate.

**Corrosion cracks** – Cracks on concrete due to steel corrosion.

**Corrosion current density** – A measure of the rate of steel corrosion presented as the ratio of corrosion current and the corroding surface area of steel.

**Corrosion inhibitor** - A chemical compound, either liquid or powder, usually intermixed in concrete and sometimes applied to concrete, and that effectively decreases corrosion of steel reinforcement.

**Corrosion rate** – The rate of loss of steel.

**Corrosion region** – Span of the beam where corrosion occurs.

**Cover concrete** – Concrete that forms the cover depth (from the outer surface of a concrete member to the surface of reinforcement).

**Cover depth** - The least distance between the surface of the reinforcement and the outer surface of the concrete.

**Crack** - A complete or incomplete separation of concrete into two or more parts.

**Cracked section** - A section with a crack.
Creep - Time-dependent deformation resulting from a sustained load.

Cure - the process by which a compound attains its intended performance properties by means of evaporation, chemical reaction, heat, radiation, or combinations thereof.

Cure of FRP systems – The process of causing the irreversible change in the properties of a thermosetting resin by chemical reaction.

Current input side – End of corrosion region which was closest to the point application of anodic current.

Current output side – End of corrosion region which was furthest from the point application of anodic current.

Debonding – A separation at the interface between the substrate and adherent material.

Deformation – A change in shape or size.

Delamination - A separation along a plane parallel to a surface as in the separation of FRP plate from substrate concrete or cover concrete from parent concrete.

Depth of the neutral axis – A distance from the outer surface of concrete to the neutral axis (in this thesis it is often measured from the compression face).

Deterioration - Physical manifestation of failure of a material (e.g., cracking, delamination, flaking, pitting, scaling, spalling, staining) caused by service conditions or internal autogenous influences.

Dimensional compatibility - A balance of dimensions, or volumetric stability, between a repair material and the existing substrate.
**Dormant corrosion cracks** - Those corrosion cracks not currently widening despite continued steel corrosion.

**Drying cycle** – A period during corrosion where concrete surface was allowed to dry under natural laboratory conditions.

**Drying shrinkage** - Shrinkage resulting from loss of moisture.

**Durability** - The ability of a structure or its components to maintain serviceability in a given environment over a specified time.

**Electrochemical compatibility** - A balance of electrochemical properties of repair and substrate concrete.

**Electrolyte** - A conducting medium in which the flow of current is accompanied by movement of matter; examples are NaCl and CaCl₂ solution.

**End-of-service life** – End of useful life of a structure.

**Epoxy** - A thermosetting polymer that is the reaction product of epoxy resin and an amino hardener.

**Fibre reinforced polymer (FRP)** – A general term for a composite material that consists of a polymer matrix reinforced with cloth, strands, or any other fibre form.

**Homogenous material** - A material that exhibits essentially the same physical properties throughout the material.

**Incompatible** - A condition in which two or more materials (patch and substrate concrete) are unable to combine or remain together without undesirable after effects.

**Interface** - The common boundary surface between two materials, e.g., an existing concrete substrate and a bonded repair material.
**Laminate** – One or more layers of fibre bound together in a cured resin matrix. Also referred to as a plate.

**Level of steel corrosion** – Amount of steel lost due to corrosion measured here as percentage mass loss of steel or as percentage loss in the area of steel.

**Load-bearing capacity** - The maximum load that may be placed on a structure or structural element before its failure.

**Longitudinal corrosion crack** - A corrosion crack that develops parallel to the length of a member.

**Longitudinal strain** – Strain along the length of a member.

**Natural steel corrosion** – Steel corrosion that was allowed to occur without impressed current and without wetting of concrete surface with salt solution.

**Neutral axis** - A line in the plane of a structural member subject to bending where the longitudinal stress is zero.

**Overdesign** - To require adherence to structural design requirements higher than service demands, as a means of compensating for statistical variation or for anticipated deficiencies or both.

**Oxidise** - To unite with oxygen.

**Parent concrete** – Overall concrete excluding the cover concrete.

**Partial surface steel corrosion** – Steel corrosion where loss in steel is localised on one surface of steel.

**Passivation** - The process in metal corrosion by which metals become passive.
**pH** - A measure of the dity or alkalinity of a solution, with neutrality represented by a value of 7, with increasing acidity represented by increasingly smaller values and with increasing alkalinity represented by increasingly larger values.

**Pit depth** – A largest loss in bar diameter. Also referred to as penetration depth.

**Pitting** - Development of relatively small surface cavities in corrosion of steel.

**Ply** – A single layer of FRP sheets.

**Poisson's ratio** - The absolute value of the ratio of transverse strain to the corresponding longitudinal strain or vertical strain resulting from uniformly distributed axial stress below the proportional limit of the material.

**Ponding** - The creation and maintaining of a shallow pond of salt solution on the surface of a concrete.

**Porosity** - The ratio, usually expressed as a percentage of the volume of voids in a material to the total volume of the material including the voids.

**Porous zone** – An empty layer with uniform thickness around steel bars that is used in models for concrete cover cracking due to steel corrosion as a simplification of voids in concrete.

**Repair** - To replace or correct deteriorated, damaged, or faulty materials, components, or elements of a structure.

**Repair effectiveness** – Ability of repairs to increase the service life of a structure.

**Repair systems** - The materials and techniques used for repair.

**Residual load-bearing capacity** - The remaining maximum load after deterioration that may be placed on a structure or structural element before its failure.
Sacrificial anodes – Sections of steel that corrode at a much larger rate than others. Their rate of corrosion may be such that other sections are protected from corrosion.

Sandblast - a system of cutting or abrading a surface of concrete by a stream of sand ejected from a nozzle at high speed by compressed air.

Service life - An estimate of the remaining useful life of a structure based on the current rate of deterioration or distress, assuming continued exposure to given service conditions without repairs.

Service load - Load specified by general building codes that will not normally cause loss of serviceability of structures.

Serviceability – A state of a structure where specified service requirements are met. In this thesis it is mainly associated with cracks and deflections that do not affect appearance.

Sheet – A dry, flexible ply used in wet layup FRP systems. Unidirectional FRP sheets consists of continuous fibres aligned in one direction and held together in-plane to create a ply of finite width and length.

Side cover concrete – Concrete that forms the side cover depth (from the outer surface on the side face of a concrete member to the surface of reinforcement).

Spall – A fragment of the cover concrete that is detached from the parent concrete.

Stable corrosion products – Corrosion products that are formed after oxidation of unstable corrosion products. Examples are haematite and magnetite.

Stiffness - Resistance to deformation.
**Stirrup** - Reinforcement used to resist shear and torsion stresses in a structural member.

**Strain** - The change in length, per unit of length, in a linear dimension of a body; measured here in micro strains.

**Stress** - Intensity of force per unit area.

**Thick-walled cylinder** – A principle used in modelling cracking of concrete where concrete around corroding steel is treated like a thick cylinder.

**Translation of the cover concrete** – Movement of the cover concrete without change in shape or size.

**Transverse crack** – A crack that develops perpendicular to the length of a member.

**Transverse stiffness** – Resistance of transverse deformation.

**Transverse strain** – Strain that is perpendicular to the length of a member and measured here, on the tensile face.

**Uniform steel corrosion** – Steel corrosion where loss in steel is uniformly distributed around the surface of a bar.

**Unstable corrosion products** – Fundamental corrosion products that are formed when there is limited supply of oxygen such as ferrous hydroxide.

**Vertical strain** – Strain that is perpendicular to the length of a member and measured here, on the side face.

**Visual inspection** - An evaluation procedure in which an investigator observes and classifies deterioration or distress on exposed concrete and steel surfaces.
Wetting cycle – A period during corrosion where concrete surface was wetted with NaCl solution.

1.7 Notation

\[ \Delta = \text{measured deflection} \]
\[ \Delta A_{rp} = \text{maximum radius of steel lost to fill the porous zone} \]
\[ \Delta A_{st,cr} = \text{area of steel lost at the time of cracking of cover concrete} \]
\[ \Delta A_{st,p} = \text{area of steel lost to fill the porous zone} \]
\[ \Delta r_{cr} = \text{maximum radius of steel lost at the time of cracking of cover concrete} \]
\[ \Delta r_p = \text{maximum radius of steel lost to fill the porous zone} \]
\[ \mu_{cr} = \text{maximum radial expansion of concrete that surrounds corroding steel bars at the time of cracking of the cover concrete} \]
\[ a = \text{a distance from centre of corroding bar to a point of consideration on the cover concrete} \]
\[ a_1 = \text{inner radius of a thick concrete cylinder (bar diameter + thickness of the porous zone)} \]
\[ a_2 = \text{outer radius of the thick concrete cylinder (bar diameter + thickness of the porous zone + cover depth)} \]
\[ A_{cor,cr} = \text{volume of corrosion products deposited at the time of cracking of the cover concrete} \]
\[ A_{cor,p} = \text{volume of corrosion products to fill the porous zone} \]
\[ A_{sc} = \text{area of compression reinforcing steel} \]
\[ A_{st} = \text{residual area of tensile reinforcement after corrosion} \]
\[ b = \text{width of concrete section} \]
\[ c = \text{concrete cover depth} \]
\[ d = \text{bar diameter} \]
\[ d' = \text{distance from the extreme compression fibre to the centroid of compression reinforcing steel} \]
\[ d_e = \text{distance from the extreme compression fibre to the centroid of tensile reinforcing steel} \]
\[ E_c = \text{elastic modulus of concrete} \]
\[ E_{eff} = \text{effective modulus of elasticity of concrete} \]
\[ E_f = \text{modulus of elasticity of FRPs} \]
\[ EI = \text{stiffness of corroded beams} \]

\[ E_s = \text{Young’s modulus of steel before yielding} \]

\[ E_{sp} = \text{Young’s modulus of steel reinforcement after yielding} \]

\[ f'_c = \text{characteristic compressive strength of concrete} \]

\[ f_c = \text{stress at the extreme compression fibre of concrete} \]

\[ F_c = \text{internal compression force carried by the concrete in compression} \]

\[ f_{ct} = \text{tensile strength of concrete} \]

\[ f_f = \text{stress applied on FRP plate} \]

\[ F_f = \text{internal tensile force carried by FRP plate} \]

\[ f_{fu} = \text{rupture stress of FRP plate} \]

\[ f_s = \text{stress applied on tensile steel reinforcement} \]

\[ F_{sc} = \text{internal compression force carried by the compression steel reinforcement} \]

\[ F_{st} = \text{internal tensile force carried by the tensile steel reinforcement} \]

\[ f_{su} = \text{ultimate tensile strength of tensile steel reinforcement} \]

\[ f_y = \text{yield stress of tensile steel reinforcement} \]

\[ h = \text{height of the beam} \]

\[ i = \text{corrosion current density} \]

\[ I_g = \text{gross second moment of area of a beam} \]

\[ k = \text{a function of the thickness of the porous zone, bar diameter and cover depth that defines the transverse stiffness of the cover concrete} \]

\[ k_1 = \text{ratio of the average stress of compression stress block to the stress applied at the extreme compression face of the concrete} \]

\[ k_2 = \text{ratio of the location of the neutral axis of the beam to the location of the resultant internal compression force carried by the concrete (all measured from the extreme compression fibre)} \]

\[ k_m = \text{an empirical factor used to reduce the ultimate strain of FRPs to a debonding or usable strain} \]

\[ l = \text{beam span} \]

\[ l_s = \text{shear span} \]

\[ M = \text{external applied moment} \]

\[ M_{cr} = \text{cracking moment of a beam} \]

\[ m_i = \text{mass per length of a corroded coupon} \]
\( m_u \) = average mass per length of an uncorroded steel coupon
\( M_u \) = ultimate capacity of the beam
\( M_{u, measured\_FRP} \) = measured capacity of FRP-repaired RC beams
\( M_{u, repaired} \) = measured capacity of repaired RC beams
\( M_{u, theoretical} \) = theoretical capacity of corroded RC beams based on reduced average cross-sectional area of steel
\( M_{u, theoretical\_crushing\_of\_conc\_in\_comp} \) = theoretical capacity of FRP-repaired RC beams using crushing of concrete in compression as a failure mode
\( M_{u, theoretical\_debonding\_of\_FRP\_plate} \) = theoretical capacity of FRP-repaired RC beams using debonding of FRP plate as a failure mode
\( M_{u, unrepairoed} \) = measured capacity of corroded and non-repaired RC beams
\( n \) = ratio of volume of corrosion products deposited to the volume of steel lost
\( n_f \) = number of FRP plies
\( P \) = internal pressure applied on the cover concrete by expansive corrosion products
\( Q \) = arbitrary mass loss of steel (%)
\( Q_{avg} \) = measured average mass loss of steel (%)
\( Q_{gi} \) = percentage mass loss of steel coupon
\( Q_{max} \) = measured maximum mass loss of steel (%)
\( r \) = radius of uncorroded steel bars
\( t \) = time of testing
\( t_{cr} \) = the time from corrosion initiation to first cracking of the cover concrete
\( t_f \) = thickness of each ply of FRPs
\( t_p \) = thickness of the porous zone
\( w_f \) = width of FRP plate
\( x \) = depth of the neutral axis of the beam as measured from the extreme compression fibre
\( y_g \) = gross depth of the neutral axis of a beam
\( \alpha \) = correction factor for theoretical capacity of corroded RC beams based on average mass loss of steel
\( \varepsilon_c \) = concrete strain at the extreme compression fibre
\( \varepsilon_{df} \) = debonding or usable strain of FRPs
\( \varepsilon_f \) = strain in the FRP plate
\( \varepsilon_{fa} \) = rupture strain of FRP plate
\( \varepsilon_{lc} \) = average longitudinal strains measured 30 mm from the compression face
\( \varepsilon_{lt} \) = average longitudinal strains measured on the tensile face of a beam
\( \varepsilon_o \) = concrete strain corresponding to the concrete compression strength \( f'_c \)
\( \varepsilon_{sc} \) = strain in the compression steel reinforcement
\( \varepsilon_{st} \) = strain in the tensile steel reinforcement
\( \varepsilon_{su} \) = ultimate strain of tensile steel reinforcement
\( \varepsilon_{tt} \) = transverse strains on the surface of concrete
\( \varepsilon_u \) = ultimate strain of concrete in compression
\( \varepsilon_y \) = yield strain of tensile steel reinforcement
\( \sigma_r \) = radial stress on the cover concrete applied by pressure from corrosion products
\( \sigma_t \) = circumferential stress on the cover concrete applied by pressure from corrosion products
\( \nu \) = Poisson’s ratio of concrete
\( \phi \) = curvature
\( \phi_{cr} \) = creep coefficient of concrete

1.8 References

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CHAPTER TWO

BEHAVIOUR OF RC STRUCTURES CORRODED AND REPAIRED UNDER SUSTAINED LOADS- A REVIEW

2.1 Introduction

Corrosion of steel bars embedded in concrete is a worldwide problem that affects numerous reinforced concrete (RC) structures. Whilst corrosion has always been problematic since the beginning of mining and refining of metals, it only gained research recognition in RC structures during the 1960s and 1970s following de-icing salts used in US highways and a construction boom in the Arabian Gulf [1]. Since then, volumes of articles continue to be published worldwide addressing corrosion issues such as mechanisms of corrosion [2-4], behaviour of corroding RC structures [5-15], measures to control further steel corrosion [16], and effectiveness of repair and strengthening to restore the service life of corroded structures [9,17-21]. Theoretical models have also been developed and calibrated by experimental results to predict the behaviour of concrete structures with corroding steel bars as well as their service lives.

In-service structures are expected to corrode under sustained service loads. Surprisingly, the majority of articles on steel corrosion have focussed on laboratory specimens that were corroded in the absence of a sustained load. Following an intensive literature review, only four unique publications on RC beams corroded under sustained loads by Yoon et al. [5], Ballim et al. [6,7], El Maaddawy et al. [8,9] and Vidal et al. [10-15], have been found. These works will be discussed in detail later in the chapter. If the level of applied load on structures corroding in the field affects the rate of steel corrosion; the behaviour of structures; and the effectiveness of repair, it is questionable if the numerous experiments (and hence the models) that were and continue to be conducted in the absence of a sustained load are truly representative of the behaviour of in-service structures.
2.2 Objectives of the chapter

The principal objective of the chapter is to review and provide a critical discussion on the effects of sustained service loads on the behaviour of corroded and repaired RC structures. In line with the objectives of the thesis, the review will be focused on identifying potential measurable parameters on a corroding RC structure that can be used to predict its non-measurable parameters as well as its end-of-service life. Parameters after repairs of corroded structures to measure their effectiveness will also be looked at.

2.3 Behaviour RC beams corroded in the absence of load

It is well researched that corrosion of steel that is embedded in concrete reduces the structural integrity of RC structures 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. Measurable parameters that are used to predict the residual capacity and the end-of-service life of corrosion-affected RC structures are often identified from these main effects of steel corrosion on the structures. To enable an understanding of the effects of load on these parameters, the following section firstly discusses them in the absence of a sustained load.

2.3.1 Time to first cracking of the cover concrete

It has already been discussed in Chapter One that one of the criteria that is often used to define the end-of-service life of corrosion-affected RC structures is the first appearance of visible corrosion cracks. Considerable experimental work has therefore been done on the time from corrosion initiation to cracking of the cover concrete. The main intent of the majority of the research work was to identify factors that influence the resistance of RC structures to cracking due to corrosion of embedded steel. The following section reviews the experimental work and the corresponding results from various researchers.
Liu and Weyers [22] assessed the effect of the amount of chlorides mixed with concrete and the cover depth of corroding steel bars on the time to first cracking of RC specimens. They found that for a RC specimen with a bar diameter of 16 mm, a cover depth of 51 mm and made from concrete that was mixed with chlorides of 5.69 kg/m\(^3\) (1.7% of chlorides by cement weight), the rate of corrosion was 2.41 \(\mu\text{A/cm}^2\). Visible corrosion cracks (to a naked eye) appeared after 1.84 years of testing. With the same chloride content and bar diameter, when the cover depth was increased to 76 mm, the rate of steel corrosion reduced to 1.79 \(\mu\text{A/cm}^2\). In this case, visible corrosion cracks appeared after 3.54 years of testing. In terms of the proportion of the influence of the studied variables on the time to cover cracking, the results indicate that a 49% increase in the cover depth resulted in a 92% increase in the time to cover cracking. This is vital information which shows the resistance of RC members to cracking due to steel corrosion to be greatly increased by a slight increase in the cover depth. For specimens with lower chloride content, no corrosion cracks were observed within the 5 years of the experimental programme. This implies that the rate of steel corrosion is dependent on the amount of chlorides within the corroding area. Further discussion of the variability of experimental procedures to accelerate steel corrosion is in later sections of the chapter.

Liu and Weyers [22] mentioned in their work that similar specimens failed within days of each other. Interestingly, the compressive strength of the concrete after 28 days of water-curing ranged from 30.9 to 39.6 MPa. This indicates that cracking of RC specimens due to steel corrosion was primarily controlled by the cover depth and the rate of steel corrosion but not the compressive strength of the concrete. Whilst this research gave an essential awareness on factors that influence the resistance of concrete to cracking due to steel corrosion, the variables investigated were certainly limited.

In a more thorough research, Alonso et al. [23] measured the time from corrosion initiation to first cracking of the cover concrete of RC specimens with various sizes of steel bars (3, 8, 10, 12 and 16 mm), various concrete cover depths (10, 15,
20, 30, 50 and 70 mm) and various current densities (3, 10 and 100 µA/cm²). The primary objective of their research was to assess the effect of concrete cover depth to bar diameter ratio and the rate of impressed current density on the time to first cracking of RC specimens. They found the time to first cracking of the cover concrete, \( t_{cr} \), to be linearly related to the ratio between the cover depth and the bar diameter, \( c/d \), and inversely related to the current density, \( i \), as shown in equation 2.1.

\[
t_{cr} = \frac{7.53 + 9.32 \frac{c}{d}}{11.6i}
\]

Where; \( t_{cr} \) = the time from corrosion initiation to first cracking of the cover concrete (years); \( c \) = concrete cover depth (mm); \( d \) = bar diameter (mm); and \( i \) = corrosion current density (µA/cm²).

Similar to findings from Liu and Weyers [22], the large coefficient of \( c/d \) in equation 2.1 (=9.32) indicates that a slight increase in the cover depth yields a significant increase in the time to cover cracking. In addition, the equation shows that a slight increase in the current density results in a large reduction in the resistance of concrete to cover cracking. Despite the trend of results from Liu and Weyers [22] and Alonso et al. [23] being similar, equation 2.1 predicts times to cracking of the cover concrete that are about 30% less than those measured by Liu and Weyers [22]. For in-service structures where the time to cover cracking is in the order of tens of years, these differences in the times to first cover cracking can be substantial.

Another research to find factors that influence resistance of RC concrete to cracking due to steel corrosion was conducted by Rasheeduzzafar et al. [24]. Their variables were cover depth to bar diameter ratio and compressive strength of concrete. They found the time to first cover cracking of corrosion-affected RC specimens to be linearly related to the value of the concrete cover depth to the bar diameter ratio. In accordance with findings from other researchers [22,23], more resistance to cracking was found to belong with concrete specimens with larger values of concrete cover depth to bar diameter ratio. They also found that for cover depth to bar diameter ratios, \( c/d \), below 2.5, the ratio between the time to
cover cracking and the cover depth to the bar diameter ratio, \( t_{cr}/(c/d) \), was 8 hours per cover depth to bar diameter ratio. However, for cover depth to bar diameter ratios above 2.5, the corresponding ratio \([t_{cr}/(c/d)]\) increased to 10 hours per cover depth to bar diameter ratio. This indicates that models for time to cover cracking should be different for ranges of values of cover depth to bar diameter ratios below and above 2.5. Probably most importantly, it also shows that to provide more resistance to cracking of the cover concrete due to steel corrosion, concrete cover depth to bar diameter ratios above 2.5 should be used. Since contrary to Rasheeduzzafar et al. [24], Alonso et al. [23] proposed a proportional resistance of concrete to cover cracking for \( c/d \) values up to 7, it is not surprising that equation 2.1 gives poor predictions of actual values measured by Rasheeduzzafar et al. [24]. To be more specific, the equation under-predicts the time to cover cracking measured by Rasheeduzzafar et al. [24] by up to 60%.

Rasheeduzzafar et al. [24] also assessed the effect of the compressive strength of concrete on the time to cover cracking. They generally found concrete with higher strength to be more resistant to corrosion cracking. However, it was most sensitive to the strength of concrete at lower strengths (<35 MPa). For example, for compressive strengths between 22.5 and 35.4 MPa, they found the ratio between the time to cover cracking and the strength of concrete to be 5 hours/MPa of concrete strength. However, for strength of concrete between 35.4 and 46.7 MPa, the corresponding ratio reduced to 3 hours/MPa of concrete strength. This indicates that contrary to the cover depth to bar diameter ratio, the influence of compressive strength of concrete on the resistance of the concrete to cracking due to corrosion of embedded steel is pronounced at low concrete strengths (<35 MPa). It therefore suggests that the concrete cover depth to bar diameter ratio has a dominating effect on the overall resistance of concrete to cracking due to steel corrosion, especially at compressive strengths of concrete above 35 MPa. This helps to explain why Liu and Weyers [22] found specimens with varying compressive strengths but similar bar diameter ratio to exhibit similar resistance to cracking due to steel corrosion.

It is evident from the above discussion that despite various researchers finding common factors that influence the resistance of concrete to cover cracking due to
steel corrosion, the actual values of the resistance of concrete were dissimilar. This is mainly attributed to the difference in the experimental procedures used by the researchers. For example;

1. Time to first cracking of the cover concrete can only be observed when the surface cracks are visible. Since it is difficult to continuously inspect concrete specimens for the appearance of the first visible cracks, the accuracy of the recorded time from corrosion initiation to cover cracking is limited. This error is especially incurred in laboratory specimens were corrosion is accelerated so that the time to cover cracking is in the order of hours. In an attempt to overcome this hurdle, Rasheeduzzafar et al. [24] designed a system to specifically monitor the time to first cracking. The system was designed such that when it registered cracking of concrete, it sent a signal to a clock that would stop, indicating its time. On the other hand, Alonso et al. [23] presumed cracking to have occurred when corrosion surface cracks had widths of 0.05 mm. Clearly, if the system designed in [24] did not measure cracks of 0.05 mm, the times to cover cracking recorded by the researchers will be different. Furthermore, and perhaps most importantly, according to other researchers [25,26], cracking of concrete begins at the steel-concrete interface and propagates outwards. This indicates that the researchers observed visible cover cracks subsequent to internal cracking.

2. The researchers used different procedures to accelerate the corrosion process. Alonso et al. [23] mixed their concrete with 3% CaCl\textsubscript{2} by cement weight and then accelerated the corrosion process by impressing an anodic current to achieve current densities ranging from 3 to 100 µA/cm\textsuperscript{2}. Liu and Weyers [22] however, mixed their concrete with chlorides up to 1.7% by cement weight but did not impress an anodic current to accelerate the corrosion process. The current densities measured by Liu and Weyers [22] were 1.79 and 2.41 µA/cm\textsuperscript{2}. Rasheeduzzafar et al. [24] on the other hand, partially immersed their specimens (which were initially free from chlorides) in a 5% NaCl solution and then applied a direct current to achieve a current density of 3000 µA/cm\textsuperscript{2}. This current density is about 1700 times larger than the current density used by Liu and Weyers [22]. If the relative response of RC structures
to steel corrosion is affected by the procedure of the accelerated corrosion as will be discussed later, the results obtained by the researchers will be different.

Despite the dissimilar results for time to first cracking of corroding RC specimens discussed above, other researchers have attempted to model it [22,25-29]. Moreover, they used the data from the various researchers above to calibrate their models. It is not surprising therefore that the models they developed, even though some are too complex to use by practitioners and engineers, have failed to give accurate predictions. The following section describes the models used to predict the time from corrosion initiation to cracking of the cover concrete. Because of the already-mentioned complexity of some of the models, the review will primarily look at the principles used to develop them.

2.3.1.2 Previous models on time to first cracking of the cover concrete

Similar to findings from Alonso et al. [23] (equation 2.1), Morinaga [27] proposed an empirical equation (equation 2.2) to predict the time from corrosion initiation to cracking of the cover concrete;

$$t_{cr} = \frac{0.602d \left(1 + \frac{c}{d}\right)^{0.85}}{i}$$  \hspace{1cm} (2.2)

Where; $t_{cr}$ = the time from corrosion initiation to cracking of the cover concrete (days); $c$ = concrete cover depth (mm); $d$ = bar diameter (mm); and $i$ = corrosion current density ($10^{-4}$ g/cm$^2$/day).

In accordance with the above-discussed experimental results from various researchers, equation 2.2 shows the time to cracking of the cover concrete due to steel corrosion to be influenced by the bar size ($d$), the concrete cover depth to bar diameter ratio ($c/d$) and the rate of steel corrosion ($i$). However, it gives predictions that are very different from those observed in other experimental tests. For example, according to Rasheeduzzafar et al. [24], a RC specimen with a compressive strength of 35.4 MPa, a corroding bar with a diameter of 12.7 mm, a concrete cover depth to bar diameter ratio of 2.5 and current density of 3000
µA/cm², will crack due to steel corrosion after 20 hours. Morinaga’s model however, predicts the time to cover cracking of a similar specimen to be 70 hours (3.5 times more), which is a significant difference especially for in-service structures.

Other researchers have as such contended that Morinaga’s model is inadequate and have developed alternative analytical models. These models were primarily based on the principle of a thick-walled cylinder under uniform internal pressure caused by the expansive corrosion products. They assumed the thick-walls of the cylinder to be made from the concrete and a uniform internal pressure to be applied by the corrosion products at the interface of the corroding steel and the concrete walls as shown in Figure 2.1.

Figure 2.1 Uniform distribution of internal pressure due to steel corrosion
Bazant [28] was the first to use this assumption when developing an analytical model to predict the time to cover cracking of corroding RC specimens. The researcher’s model was however, later criticised by other researchers for its inability to recognise that concrete is a porous material and contains voids which corrosion products must first diffuse into before applying stresses on the cover concrete [22,25,26,29]. Regrettably, researchers who decided on including this factor in their models found the non-homogeneity of concrete to offer a difficult challenge to quantify the voids accurately. As a simplification of the problem, the voids in concrete were represented by a porous zone around the steel bars that has a uniform thickness, $t_p$, ranging from 10 to 20 micrometers [22,25,26,29] as shown in Figure 2.1.

The assumption of the existence of a porous zone around steel bars necessitates the relation between the expansion of the cover concrete and the loss in the area of steel during the period from the activation of the corrosion process to first cracking of the cover concrete to be modelled in two distinct stages. The first stage corresponds to the time required for corrosion products to completely fill the porous zone. During this stage, corrosion products are assumed to diffuse into the porous zone without applying stresses on the cover concrete. The second stage is when the porous zone has been fully-filled with corrosion products so that continued steel corrosion necessitates the surrounding concrete to expand so as to allow for deposit of new corrosion products.

Perhaps the two simplest models that use the principle of a thick-walled cylinder under uniform internal pressure to model the time to cover cracking of concrete due to steel corrosion and appreciate the concept of the porous zone were developed by El Maaddawy and Soudki [29] and by Liu and Weyers [22]. The primary difference between the models is that Liu and Weyers [22] developed their own relation of the level of steel corrosion, current density and time of electrolysis whilst El Maaddawy and Soudki [29] used Faraday’s Law. Since Faraday’s Law is the more widely used relation and many researchers have shown that it better-predicts the level of steel corrosion than the relation by Liu and Weyers [30], the review will focus on the model developed by El Maaddawy and Soudki [29] (equations 2.3 to 2.7).
\[
t_{cr} = \left[ \frac{7117.5(d + 2t_p)(1 + \nu + \beta)}{iE_{eff}} \right] \left[ \frac{2c f_{ct}}{d} + \frac{2t_pE_{eff}}{(d + 2t_p)(1 + \nu + \beta)} \right]
\]  
(2.3)

\[
f_{ct} = 0.94 \sqrt{f'_{c}}
\]  
(2.4)

\[
E_{c} = 4500 \sqrt{f'_{c}}
\]  
(2.5)

\[
\beta = \frac{(d + 2t_p)^2}{2c(c + d + 2t_p)}
\]  
(2.6)

\[
E_{eff} = \frac{E_{c}}{1 + \phi_{cr}}
\]  
(2.7)

Where; \( t_{cr} \) = time to cover cracking of concrete due to steel corrosion (days); \( d \) = diameter of steel bars (mm); \( t_p \) = thickness of the porous zone (mm); \( \nu \) = Poisson’s ratio of concrete (0.18); \( f_{ct} \) = tensile strength of concrete (MPa); \( c \) = depth of the cover concrete (mm); \( E_{eff} \) = effective modulus of elasticity of concrete (MPa); \( E_c \) = elastic modulus of concrete (MPa); \( \phi_{cr} \) = creep coefficient of concrete (2.5); \( f'_{c} \) = characteristic compressive strength of concrete (MPa); and \( i \) is the current density (µA/cm²).

In consent with experimental results discussed above as well as Morinaga’s model, the model by El Maaddawy and Soudki [29] shows the bar diameter, the current density, and the cover to bar diameter ratio to be important factors of the resistance of concrete to cracking due to corrosion of embedded steel. As already mentioned, in developing their model, El Maaddawy and Soudki [29] criticised the model by Morinaga [27] for its failure to account for mechanical properties of concrete. However, if equations 2.4, 2.5 and 2.7 are substituted into equation 2.3, it can be shown that the mechanical properties of concrete in the model (\( E_{eff} \) and \( f_{ct} \)) cancel out. The model by El Maaddawy and Soudki therefore, considers the Poisson’s ratio and the amount of voids in concrete as the only physical properties of concrete that affect its resistance to cracking due to corrosion of embedded steel. Even then, the function \( 1 + \nu + \beta \) in the model is approximately equal to one.
so that the effect of the Poisson’s ratio on the model is limited. From this discussion, it can be shown that equation 2.3 simplifies to equation 2.8. The resulting equation indicates that the model by El Maaddawy and Soudki [29] considers concrete with large voids, which will normally have a low compressive strength, to be more resistant to cracking due to steel corrosion.

\[
t_{cr} = \frac{5.4}{t} \left(1.9c + 2571.4t_p\right)
\]  
(2.8)

To emphasise the earlier notion that the analytical models give poor predictions of the experimental data, the model by El Maaddawy and Soudki [29] (using an average thickness of the porous zone of 0.015 mm) gives predictions that are up to 75% less that the experimental values obtained by Rasheeduzzafar et al. [24]. Due to this low level of accuracy, in calibrating their model, the authors proposed using a range of values of the thickness of the porous zone from 0.01 to 0.02 mm. Unfortunately, the range in the porous zone used by the researchers may have predictions of times to cover cracking on a corroding structure that are up to 40% apart. As previously mentioned, for in-service structures where the rate of corrosion is slow such that the times to cover cracking are in the order of tens of years, the differences in the predicted values after varying the thickness of the porous zone are very significant.

The aforementioned experimental programmes from various researchers and their corresponding results make it difficult to develop analytical models to predict time to first cracking accurately. The level of impressed current density aside, the pressing matter with the experimental work has been the accuracy as well as the consistency in the measurements of the times to cover cracking. Analytical models have also differed greatly on their assumption of complete-cracking of the cover concrete. For example, the simplified models by Liu and Weyers [22] and by El Maaddawy and Soudki [29] were developed with the assumption that cracking of the cover concrete occurs when the tensile stresses in the circumferential direction at every part of the concrete ring have reached the tensile strength of concrete. In relation to the internal pressure applied by corrosion products, this assumption is represented by equation 2.9. Unless this assumption
corresponds to visible corrosion cracks (>0.05 mm), it does not model the time to
cover cracking found in experimental works.

\[ P = \frac{2cf_{ct}}{d + 2t_p} \]  

(2.9)

Where; \( P \) = internal pressure (MPa); \( d \) = diameter of steel bars (mm); \( t_p \) =
thickness of the porous zone (mm); \( c \) = depth of the cover concrete (mm); \( f_{ct} \) =
tenisle strength of concrete (MPa).

For more complex models such as in [25,26], cracking of the cover concrete is
modelled as a process of tension softening, so that the cracked concrete will have
some residual strength after its tensile capacity is exceeded. The models further
propose that cracking of the cover concrete due to steel corrosion initiates on the
inner surface of the concrete ring (near the corroding steel) and progresses
outwards. The concrete cover is assumed to have fully-cracked when the crack-
front reaches the surface of the concrete member. Since internal cracks are
difficult to measure (and the researchers did not measure them), the time for
internal cracking as well as the progression of the crack-front to the outer surfaces
of a RC member that is calculated from the models is speculative. This explains
the little added-accuracy that is gained by using these complex models. The
following section introduces a simple procedure that could be used to accurately
and consistently measure the time to cover cracking of corrosion-affected RC
structures.

\textit{2.3.1.3 Forward looking on measuring time to first cracking of concrete}

From basic mechanics, a uniform radial pressure, \( P \), applied on the internal
surfaces of a thick-walled cylinder (as shown in Figure 2.1) yields two principal
stresses on the walls of the cylinder; the circumferential stress, \( \sigma_t \), and the radial
stress, \( \sigma_r \), which are given by equations 2.10 and 2.11 [31].

\[ \sigma_t = \frac{Pa_1^2(a^2 + a_2^2)}{a^2(a_2^2 - a_1^2)} \]  

(2.10)
\[ \sigma_r = -P \frac{a_1^2(a_2^2 - a^2)}{a^2(a_2^2 - a_1^2)} \]  

(2.11)

Where; \(a_1\) = inner radius of the thick concrete cylinder; \(a_2\) = outer radius of the thick concrete cylinder; and \(a\) = a distance from centre of the corroding bar to a point of consideration on the cylinder.

In addition to equations 2.10 and 2.11, it can be shown that transverse deformations measured as circumferential strains on the walls of the cylinder or transverse strains on the external surfaces of the cylinder, \(\varepsilon_{tt}\), are given by equation 2.12.

\[ \varepsilon_{tt} = \frac{l}{E_c} \left( \sigma_t - \nu \sigma_r \right) \]  

(2.12)

Where; \(E_c\) = modulus of elasticity of the concrete; and \(\nu\) = Poisson’s ratio of concrete.

It is evident that strains on the surfaces of the concrete which theoretically are given by equation 2.12, can easily be measured using strain gauges. Most importantly, when the voids in concrete are fully-filled with corrosion products, the strains are expected to increase with an increase in the level of steel corrosion. Moreover, it is logical that after cracking of the cover concrete, its expansion due to continued steel corrosion will not follow the principle of a thick-walled cylinder under internal pressure. This is because after cracking, corrosion products are easily dissipated through the cracks and thus relieves the corrosion pressure on the surface of the concrete. In addition, cracking of the cover concrete significantly reduces its stiffness. The variation of strains before and after cracking of the cover concrete is therefore expected to be different. If the strains due to steel corrosion are continuously measured with time, their variation will surely indicate accurately, the time when cracking occurs. This is an important procedure that would overcome the hurdle discussed above. Furthermore, from the variation of strains and knowing the amount of strains that indicate cracking of concrete, better-calibrated analytical models to predict the time to cover cracking
of concrete can be developed. This procedure is extensively used in this research and is discussed in detail in Chapter Three. The corresponding results as well as an analytical model that links the measured strains with the level of steel corrosion are discussed in Chapter Six.

2.3.2 Widening of corrosion cracks

Corrosion products are deposited around the surface of a corroding bar that is embedded in concrete. In fact many researchers assume that they are uniformly distributed around the surface of a bar. The corresponding principal stresses that they apply at the steel-concrete interface are therefore, nearly orthogonal to the longitudinal axis of the bar as shown in Figure 2.1. As a result, corrosion cracks on the cover concrete often propagate parallel to the axis of a corroding bar but widen in the vertical or transverse direction of RC members depending on which face of the member they appear. As mentioned earlier, the main interest of researchers and asset managers is to predict accurately the residual capacity of the structures. Regrettably, load-bearing capacity of structures is difficult-to-measure. The solution has therefore been to identify measurable parameters on corrosion-affected RC structures that can indicate their residual capacity. The length of corrosion cracks along a RC member can only locate the corrosion region but can’t quantify the level of steel corrosion. It is therefore not of direct interest here. The width of corrosion cracks, which constantly widens with the level of steel corrosion and is easy to measure, however, is. The following sections look into previous research on corrosion crack widths in RC structures.

2.3.2.1 Relation between corrosion crack widths and the level of steel corrosion

Probably the most thorough work to relate the width of corrosion cracks with the level of steel corrosion was presented by Torres-Acosta and Martinez-Madrid [32]. In their publication, they compiled data in the literature on the interaction between maximum crack widths and the level of steel corrosion. Since the various researchers from whom they obtained the data used different experimental procedures, it is not surprising that the relation they obtained had a large scatter. However, they found a defined trend that indicated corrosion crack widths to
linearly increase with an increase in the level of steel corrosion. From this trend, it can be shown that a mass loss of steel of 1% corresponded to a maximum crack width of 0.03 mm. Other researchers (not included in Torres-Acosta and Martinez-Madrid [32]) such as Alonso et al. [23] also found the width of corrosion cracks to linearly increase with the level of steel corrosion. They however, contended that a mass loss of steel of 1% corresponds to a maximum crack width between 0.04 and 0.08 mm. El Maaddawy and Soudki [33] found mass loss of steel of 1% to yield maximum corrosion cracks from 0.08 to 0.14 mm. In agreement with the above results, Badawi and Soudki [34] found mass loss of steel of 1% to correspond to crack widths from 0.1 to 0.14 mm.

As will be discussed later, these relations have been used by some researchers to predict the residual capacity of corroding RC structures [35]. Their major drawback however, is that whilst crack widths were measured at a particular location of a corroding RC member so as to identify maximum crack width, mass loss of steel was taken as an average loss within the corroded region. From a structural mechanics viewpoint, and as will be shown later, it is foreseeable that the residual capacity of a corrosion-affected RC structure is related to the maximum loss in cross-sectional area of steel. The interest of researchers should therefore be to relate the maximum crack width with the maximum loss of steel. In fact, Rio et al. [18] showed that a RC structure with a maximum crack width of 0.1 mm may have a maximum mass loss of steel of 42% whilst a similar specimen with a crack width of 1 mm may have a maximum mass loss of 17%. This highlights the difficulty of trying to relate maximum crack widths with mass loss of steel. Further discussion on this is in Chapter Six. The other shortcoming of the above relations between mass loss of steel and crack widths is that various researchers used different procedures to accelerate steel corrosion. As will be shown later, some of the procedures they used do not yield results that are representative of structural damage caused by natural steel corrosion.

2.3.2.2 Corrosion crack patterns

In order to further understand the cracking behaviour of corrosion-affected RC structures, some researchers have studied the pattern of corrosion cracks. Alonso
et al. [23] found that when a corroding steel bar was placed at the corner of a concrete specimen so that it had equal covers on the two near-faces, the first corrosion cracks simultaneously appeared in both faces and propagated parallel to the steel bars. When the cover depths were varied, they found the cracks to appear on the face nearest to the bar. In a similar work, El Maaddawy and Soudki [33] studied the pattern of corrosion cracks in corrosion-affected RC specimens but with two corroding bars. The bars had centre-to-centre spacing of 80 mm and concrete covers of 25 mm on the side and top face of beams. For all samples, they initially observed two corrosion cracks near each corroding bar, and each crack propagated parallel to the bar. The cracks were either on the top face or on the side face. Interestingly, when the level of steel corrosion was increased, a third crack appeared next to a corroding bar but on a face that was uncracked. This indicates that the pattern of corrosion cracks changes with an increase in the level of steel corrosion.

Cabrera [36] also monitored crack patterns but on corroded RC slabs with different cover depths. Their specimens were reinforced with three steel bars with centre-to-centre spacing from 98 to 124 mm. Cover depths of steel bars were also varied. They found crack widths on all slabs to develop parallel to each corroding bar. When cover depths on the side faces and the tensile face were varied, the cracks were only observed on the face nearest to the corroding bar. However, when covers were equal, exterior bars either exhibited a crack on the side face or on the tensile face but not on both faces. Similar crack patterns were observed by other researchers such as Torres-Acosta et al. [37], Rodriguez et al. [38], and Rio et al. [18].

Without understanding the implications of these crack patterns on the rate of widening of corrosion cracks, the rate of steel corrosion, and the behaviour of corroded structures, there is little value in studying them. Unfortunately, in studying corrosion crack patterns, many researchers only meticulously prepared crack maps and measured crack widths at the end of corrosion tests [6,8,9,23,36-38]. For the few who continuously measured widening of corrosion cracks, they limited their measurements to a few points on the cover concrete [33]. Research is therefore needed to assess the interaction between corrosion crack patterns, rate of
widening of corrosion cracks and mass loss of steel. This interaction is discussed in Chapter Six.

2.3.2.3 End-of-service life based on corrosion crack widths

Despite inadequate information on the rate of widening of corrosion cracks, DuraCrete Final Technical Report [39] has proposed two limits of corrosion crack widths that indicate end-of-service life of corrosion-affected RC structures. The lower limit is a crack width of 0.3 mm. This is a crack width (regardless of the cause) that is specified by many design standards such as [40] to be non-aesthetic to users of the structure and hence indicate loss in serviceability of the structure. The upper limit is a crack width of 1 mm. The report contended that it indicates that spalling of cover concrete is likely to occur. From previously-discussed relations between crack widths and mass loss of steel, the lower limit of undesirable corrosion crack widths relates to mass loss of steel from 2 to 10%. The upper limit is caused by mass loss of steel ranging from 7 to 33%. It will be shown later in this chapter as well as in Chapter Four that the range of mass loss of steel to reach the upper crack limit corresponds to a significant variation on residual load-bearing capacity of structures. It is discussed in Chapter Six that this large variation is a result of lack of information on the interaction between corrosion crack widths and the rate of widening of corrosion cracks. Another probable cause of the large variation in mass loss of steel at a chosen crack widths is the variety of procedures used to accelerate steel corrosion.

2.3.2.4 Influence of accelerated corrosion on the rate of structural damage

Despite steel corrosion causing the most damage in in-service RC structures near the marine environment, in laboratory terms, the process of natural steel corrosion is very slow needing tens of years to cause reasonable structural damage. Researchers therefore, have and continue to use various techniques to accelerate the corrosion process 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.
To hurriedly de-passify steel, some researchers opted to mix concrete with chlorides ranging from 1% [19] to 5% [33] by weight of cement. Others immersed their cured samples in tanks with NaCl solution with concentration of 3% [41] to 5% [36] by weight of the solution. Both procedures above were used by some researchers [19,41,42]. Surely these procedures result in uniform distribution of corrosion agents around the steel. Under natural steel corrosion however, limited faces of the structure are often exposed to chloride attack. In addition, chlorides and other deleterious compounds are often excluded from concrete mixes in practice. Regrettably, how the variation of the distribution of corrosion agents around the corroded steel as well as their concentration affects the corrosion process and the behaviour of corroding structures is unclear. Results from Liu and Weyers [22] (discussed in section 2.3.1) however, suggest that if after contamination of specimens with chlorides the specimens are allowed to corrode naturally, higher concentration of chlorides increases the rate of steel corrosion.

To better represent natural steel corrosion, some researchers contaminated selected faces of their specimens with chlorides. This was achieved by either building NaCl ponds on the surfaces of the specimens to be contaminated [5] or by selectively spraying them with salt solution [18]. These procedures of selective contamination of RC specimens with corrosion agents enabled some researchers to simulate cyclic wetting and drying of RC structures with salt solution that is often observed in in-service structures. In addition to simulating in-service conditions, wetting and drying of specimens with contaminants was carried out to promote steel corrosion. Whether this procedure is effective at increasing the rate of steel corrosion has however, not been well researched. Further discussion on it is in Chapter Four.

Probably the most varied factor of steel corrosion in accelerated tests has been the level of impressed current density. It has ranged from 3 µA/cm² [23] to 10400 µA/cm² [43]. Assuming a proportional structural damage due to accelerated tests, level of damage caused by a current density of 3 µA/cm² over a period of one year can be obtained within two hours when a current density of 10400 µA/cm² is used. Should the accelerated tests give proportional damage then there is little value in using low rates of steel corrosion.
To assess the effect of varying current densities on the proportion of structural damage, Mangat and Elgarf [19] measured slopes of load-deflection curves of RC specimens that were corroded with corrosion rates from 1000 to 4000 µA/cm². The concrete mix that they used for RC specimens had 1% NaCl salt by cement weight and during the accelerated test, their specimens were immersed in a 3.5% NaCl solution. At lower levels of steel corrosion (<10% mass loss), Mangat and Elgarf found little influence of corrosion rates on the stiffness of corroded specimens. However, at mass losses of steel above 10%, the slopes of load-deflection curves of corroded RC specimens were very much influenced by the rate of steel corrosion. For example, at a mass loss of steel of 15%, a specimen that was corroded with a current density of 4000 µA/cm² needed a load of 26 kN to cause a deflection of 3 mm. At the same level of steel corrosion, a similar specimen that was corroded with a current density of 1000 µA/cm² exhibited a deflection of 3 mm at a load of 38 kN. This indicates that at the same level of steel corrosion, there was a larger loss in stiffness of specimens that were corroded under a higher current density. Therefore, if the loss in stiffness was to be used to predict the level of steel corrosion, its relation with corrosion level from accelerated tests will result in engineers underestimating the level of steel corrosion in in-service structures.

Mangat and Elgarf [19] asserted that for accelerated corrosion tests in laboratory specimens, especially when the target level of steel corrosion is high, the lowest practical corrosion rate should be used to accelerate reinforcement corrosion. Since they used corrosion rates that ranged from 1000 to 4000 µA/cm², it is reasonable to assume that a corrosion rate that is below 1000 µA/cm² is appropriate for laboratory tests. The major drawback with this work is that it used parameters that were only measured at the end of the corrosion process. It therefore does not provide the influence of the rate of steel corrosion on the much-needed rate of change of the parameters with an increase in the level of steel corrosion.

The following section describes a more refined work by El Maaddawy and Soudki [33]. It was also intended to find an impressed current density that can produce desired structural damage in a short time without excessively altering structural
response under natural steel corrosion. In their work, the researchers used the rate of widening of corrosion cracks and the average mass loss of steel at the end of the corrosion process as parameters that indicate corrosion damage. Crack widths were measured on the side faces of specimens using a demountable mechanical (demec) gauge with a gauge length of 50 mm. De-passivation of steel was accelerated by mixing concrete with 5% NaCl by weight of cement. Current densities assessed ranged from 100 to 500 µA/cm².

They found that at corrosion crack widths below 0.03 mm (which corresponded to a theoretical mass loss of steel from Faraday’s Law of 0.8%), specimens corroded using various current densities exhibited a similar rate of expansion of the cover concrete. Unfortunately, these cracks were too small to see with a naked eye. They therefore provide little information on the earlier-discussed procedures to measure time to first cracking. At larger crack widths (> 0.03 mm), the researchers found specimens subjected to current densities above 350 µA/cm² to exhibit a significantly larger rate of widening of corrosion cracks (up to 4 times) than specimens that were subjected to current densities below 200 µA/cm². Interestingly though, they found that regardless of the level of impressed current density used, the average mass loss of steel at the end of the corrosion process was within 4% of the theoretical mass loss of steel predicted from Faraday’s Law. These results indicate that if crack widths were to be used to predict the level of steel corrosion, relations between crack widths and mass loss of steel from highly accelerated tests will underestimate steel corrosion in in-service structures. To be more precise, under highly accelerated tests (500 µA/cm²), they found a crack width of 1 mm to correspond to mass loss of steel of 7.3%. However, at a reduced corrosion rate (100 µA/cm²), a crack width of 1 mm was found to correspond to a mass loss of steel of 13.3%. Probably under natural steel corrosion, crack widths of 1 mm will correspond to larger levels of steel corrosion. This will be confirmed in Chapter Six.

Large crack widths exhibited in highly accelerated tests probably explain the large reduction in stiffness of specimens observed by Mangat and Elgarf [19] at corresponding corrosion rates. These findings also help to explain the large variations in mass loss of steel required to cause a selected crack width (especially
if it is above 1 mm) discussed earlier. They therefore question the applicability of those relations to in-service structures where the level of steel corrosion is low. El Maaddawy and Soudki [33] concluded that an impressed current density below 200 µA/cm² does not excessively alter the structural performance of corrosion-affected RC specimens that would be observed under natural steel corrosion. Their research however, does not address other earlier-discussed factors of steel corrosion such as partial contamination of specimens with chlorides as well as cyclic wetting and drying of specimens with salt solution. Furthermore, crack widths were measured on one face of the specimens so that the effect of crack patterns on the rate of widening of corrosion cracks was also not assessed. If in addition to the level of impressed current, crack widths and the rate of steel corrosion are related to crack patterns and other procedures of accelerating steel corrosion, then the findings from El Maaddawy and Soudki [33] may not fully represent in-service conditions. Despite these drawbacks, the findings from the researchers have been followed by many other researchers [34,44]. More discussion on this matter is in Chapters Four and Six.

2.3.2.5 Type of corrosion products during steel corrosion

Another parameter that needs discussion when comparing natural steel corrosion with accelerated corrosion tests in laboratories is the types of corrosion products. Researchers have detected various corrosion products in corrosion-affected RC structures, all with different densities and volume expansion as shown in Figure 2.2 [3,45]. The type of each corrosion product was found to be primarily dependent on the pH and the availability of oxygen [1,3,45]. These factors (pH and quantity of oxygen) are extremely variable and difficult to quantify in a corroding RC structure. Many researchers contend that, for corrosion of steel that is embedded in concrete, ferrous hydroxide is the fundamental corrosion product [3,22,25,26,29,46]. With an increase in the supply of oxygen (especially after cracking of the cover concrete), more stable corrosion products such as haematite and magnetite are formed.
Varying the procedure of the accelerated corrosion process is therefore likely to influence the type of corrosion products formed. For example, when specimens are fully immersed in NaCl solution, oxygen is scarce so that more soluble products such as ferrous hydroxide are expected. In addition, when the rate of steel corrosion is high (as in accelerated corrosion tests), the rate of ingress of oxygen into the concrete might not be adequate to produce stable compounds. This helps to explain why in accelerated corrosion tests where specimens are immersed in salt solution (to be discussed in detail in Chapters Four and Six), corrosion products formed are often greenish-black in colour indicating a large presence of ferrous hydroxide. On the other hand, when steel corrosion is slow and concrete is drier, oxygen is abundant so that more stable products such as haematite and magnetite are expected. To confirm this notion, reddish-brown products indicating a large presence of stable corrosion compounds are often
found in in-service structures as well as in laboratory specimens where steel corrosion is natural. Since these products are of different volume densities, the rate of widening of corrosion cracks is expected to be greatly influenced by the procedure used to accelerate steel corrosion. It is important to observe that large densities belong with more soluble products. Therefore, at the same level of steel corrosion, specimens that exhibit unstable corrosion products are expected to be more severely cracked than those with more stable products. This is in agreement with results from El Maaddawy and Soudki et al. [33].

2.3.3 Prediction of mass loss of steel using Faraday’s Law

Section 2.3.2 indicated that if properly measured, cracking of concrete due to steel corrosion is an important parameter that can be used to predict the level of steel corrosion. To confirm the interaction between corrosion crack widths and the level of steel corrosion, some researchers have measured the actual level of steel corrosion at the end of the corrosion process. This was done by removing corroded steel bars from concrete specimens; cleaning them; and measuring the level of steel corrosion as mass loss of steel or as corrosion pit depths. In real structures however, it is uncommon for corroded steel bars to be removed from structures. Faraday’s Law is therefore often used to estimate the level of steel corrosion. It is also extensively used in modelling other parameters of corroding RC structures such as time to first cover cracking [29] and stiffness of corroded structures [47]. To relate measurable parameters of RC structures with the level of steel corrosion accurately, there is need to understand the suitability of Faraday’s Law to estimate the level of steel corrosion.

Figure 2.3 shows a plot of mass loss of steel measured at the end of the corrosion process with predicted mass loss of steel from Faraday’s Law. The data in Figure 2.3 was obtained from various researchers in the literature. The actual values and a summary of conditions of the experiments used by the researchers are in Appendix A. As expected from the variation of conditions for accelerated corrosion from various researchers, the figure shows a large scatter. The difference between mass loss of steel predicted from Faraday’s Law and actual mass loss ranged from -6.7 to 23.9% with a mean of 1.3% and a standard
deviation of 3.6%. There was however, a trend ($R^2 = 0.82$) that measured mass loss was linearly related to predicted loss. It is evident from the figure that at mass losses of steel above 8%, the majority of data points were below the line of equality. This indicates that at large mass losses of steel ($> 8\%$), Faraday’s Law tends to overestimate the level of steel corrosion. The trend-line shows the predicted loss to be around 18% larger than the measured loss. Some researchers believe this to be caused by corrosion products building up around the reinforcing bar surface and thus forming a physical barrier to the ingress of corrosion agents [22,34]. From the previous discussion, it is expected that more soluble products which occupy larger volume will form a large barrier and hence significantly retard the corrosion process. This is discussed in more detail in Chapter Four.

Figure 2.3  Measured versus predicted mass loss of steel from Faraday’s Law

Despite the trend discussed above, it is worth pointing out that the measured mass loss of steel presented in Figure 2.3 is an average mass loss of steel over the entire
corroded length of a bar. If the level of steel corrosion varies along the bar, average mass loss of steel and hence Faraday’s Law, may underestimate the maximum level of steel corrosion. Rather than measuring average mass loss of steel, some researchers opted to measure maximum pit depths [32,35]. Torres-Acosta et al. [32,35] tried to correlate maximum pit depths with average penetration depth (calculated from average mass loss). They found them to be linearly related but, the maximum pit depth was about eight times larger than the average penetration depth. Similar results were found by Rodriguez et al. [38]. This is important information which suggests the need to evaluate the accuracy of Faraday’s Law to predict the maximum mass loss of steel. The more important matter is certainly how the various mass losses of steel relate to the residual capacity of corrosion-affected RC structures.

2.3.4 Relation between residual capacity and level of steel corrosion

As previously mentioned, residual load-bearing capacity of a corrosion-affected RC structure is an important parameter for users of the structure because of the severe consequences of structural collapse. Similar to the level of steel corrosion, load-bearing capacity is difficult to measure. After relating the level of steel corrosion with corrosion crack widths, researchers have found it necessary to relate the level of steel corrosion with residual capacity of the structure. These relations were intended to enable engineers in the field to easily associate residual capacity of corrosion-affected RC structures with corrosion crack widths. In addition, knowing unsafe load-bearing capacities of corroding RC structures, these relations may help redefine widths of corrosion cracks reported in DuraCrete Final Technical Report [39] that indicate end-of-service life of the structures.

Ting and Nowak [48] developed an analytical model to simulate the effect of loss in the area of steel on the load-bearing capacity of structural members. They then demonstrated their model on a typical RC T-beam using arbitrary values of loss in the area of steel. Their model showed a linear relation between residual load-bearing capacity of the beam with level of steel corrosion. They found a 1% loss in the area of steel to correspond to a 0.7% loss in the capacity of a beam. Since
Ting and Nowak’s model was not calibrated with experimental results, its applicability to in-service structures is open to discussion. In fact, Mangat and Elgarf [19] as well as Azad et al. [42] contended that at large mass losses of steel (>10%), calculated values of load-bearing capacity using measured average mass losses of steel had little relations with experimental results. For example, according to Mangat and Elgarf [19], a mass loss of steel of 19% corresponded to a predicted loss in load-bearing capacity of 20%. The measured loss in the load-bearing capacity was however, found to be 78%. Azad et al. [42] found average mass loss of steel of 1% to relate to loss in load-bearing capacity of 1.4%. The corresponding relation between mass loss of steel and theoretical load-bearing capacity varied with the level of steel corrosion. At a corrosion level of 31%, theoretical load-bearing capacity exceeded the measured capacity by 30% but at lower levels of corrosion (around 5%), theoretical capacity was found to be similar to the measured capacity. The researchers 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. According to Azad et al. [42] the residual load-bearing capacity of corroded RC beams should be calculated using equations 2.13 and 2.14. In line with their experimental findings, these equations indicate that the needed-correction-factor, \( \alpha \), reduces with an increase in the level of steel corrosion \((i_t)\).

\[
M_{u, \text{unrepaired}} = \alpha M_{u, \text{theoretical}}
\]  
(2.13)

\[
\alpha = \frac{14.7}{d(i_t)^{0.5}} \leq 1
\]  
(2.14)

Where \( M_{u, \text{unrepaired}} \) = measured capacity of beams (kN-m); \( M_{u, \text{theoretical}} \) = theoretical capacity of beams based on reduced average cross-sectional area of steel (kN-m); \( \alpha \) = correction factor; \( d \) = bar diameter (mm); \( i \) = corrosion current density (mA/cm\(^2\)); and \( t \) = duration of corrosion (days).

Section 2.3.3 mentioned a significant difference between average and maximum mass loss of steel. Therefore there is merit in also attributing the discrepancy
between measured and predicted capacity of structures (using average mass loss of steel) to the difference in the average and maximum mass loss of corroded steel. Furthermore, despite developing bond correction factors (equation 2.14), Azad et al. [42] point out that due to pitting corrosion, the loss in reinforcing bars at some sections was seen to be considerably higher than at other sections. To confirm the influence of maximum loss of steel on the residual capacity of concrete structures, Torres-Acosta et al. [35] found a poor relation between average penetration depth on steel bars (calculated from average mass loss of steel) due to steel corrosion and the residual capacity of RC specimens. A cross-sectional loss of steel of 1% was found to be equivalent to a loss in capacity of 1.6%. This relation is similar to the relation found by Azad et al. [42] where average mass loss of steel was used. Torres-Acosta et al. [35] however, found a good relation ($R^2 \approx 1$) between the load-bearing capacity and maximum pit depths. From this relation, it can be shown that a 1% maximum loss in area of steel yields a 0.6% loss in load-bearing capacity. Note that Torres-Acosta et al. [35] presented their results using radius loss instead of loss in cross-sectional area of steel. They were converted here to allow for them to be compared with those from other researchers [19, 42, 48]. Interestingly, relation found by Torres-Acosta et al. [35] is similar to the relation developed by Ting and Nowak [48]. Even more intriguing, no correction factors as recommended by Mangat and Elgarf [19] and Azad et al. [42] were needed in Ting and Nowak’s model. It therefore suggests that the correction factors are limited to theoretical models of load-bearing capacity that use average mass loss of steel. More importantly, it implies that the loss in bond between steel and concrete may not be the reason for the failure of the theoretical models.

In terms of crack limits stipulated in [39] to define end-of-service life of corrosion-affected RC structures; i) a lower crack width limit of 0.3 mm corresponds to loss in load-bearing capacity of 3 to 15%; and ii) an upper crack width limit of 1 mm corresponds to loss in load-bearing capacity of 11 to 50%. These calculations were based on the relations between average mass loss of steel and measured residual capacity because crack widths were also related with average mass loss. It is evident that using a corrosion crack width limit of 1 mm to indicate end-of-service life may result in some structures being repaired at unsafe
residual load-bearing capacities. Since it was shown to be more accurate to model loss in capacity using maximum loss in area of steel, it is essential to develop relations between crack widths and residual load-bearing capacities that are based on maximum loss in area of steel. These relations are discussed in Chapter Six.

2.3.5 Stiffness of corrosion-affected RC beams

Stiffness is another measurable parameter of corrosion-affected RC structures that can be used to indicate corrosion damage as well as end-of-service life of the structures. It has often been measured from load-deflection curves obtained from subjecting already-corroded specimens to monotonically-increasing static loads. Mangat and Elgarf [19] found slopes of load-deflection curves of corroded beams to decrease with an increase in the level of steel corrosion thereby indicating a reduction in stiffness of beams due to steel corrosion. Similarly, Cabrera [36] found stiffness of corroded beams to be greatly reduced by steel corrosion. They found a mass loss of steel of 1% to result in a deflection increase of about 5%. The drawback with work from Mangat and Elgarf [19] as well as Cabrera [41] is that they only measured stiffness at a chosen level of steel corrosion. The progression of loss in stiffness with increase in the level of steel corrosion was therefore unknown. Torres-Acosta et al. [37] attempted to attain this progression by constantly measuring stiffness of different corroding RC specimens. To achieve this, they used cyclic service loading of specimens to measure stiffness at a chosen level of steel corrosion followed by unloading them to continue steel corrosion. The researchers found that when corrosion was distributed over the entire RC beam span, a 1% mass loss of steel yielded a 1.4% loss in stiffness. For localised corrosion, they found a 1% mass loss of steel to yield a 1.3% loss in stiffness. Bear in mind that relations by Torres-Acosta et al. [37] had to be converted here to loss in cross-sectional area of steel to allow for them to be compared to those provided by other researchers [19, 36]. Their actual published results were that 10% radius loss due to steel corrosion reduced stiffness by 24.6% and 26.9% for localised corrosion and general corrosion, respectively.

It should be pointed out that when developing a model for residual capacity of RC beams due to steel corrosion, Ting and Nowak [48] also demonstrated the effect
of loss in the stiffness of beams. They found a 1% loss in cross-sectional area of steel to correspond to a 0.6% loss in stiffness. Furthermore, according to their model, and in consent with Torres-Acosta et al. [37], stiffness linearly reduces with an increase in the level of steel corrosion. Contrary to the above results, Cairns et al. [41] recorded an increase in stiffness of 90% when average steel corrosion was increased to 10%. They attributed the increase in stiffness to an increase in anchorage capacity due to increased radial stresses (from steel corrosion) at the concrete-steel interface. Their explanation was probably prompted by previous work on the effect of steel corrosion on the bond between steel and concrete.

Perhaps one of the mostly cited works on the effect of steel corrosion on the bond between steel and concrete was carried out by Almusallam et al. [43]. They corroded various concrete specimens by impressing a constant current density of 10400 µA/cm². Their results showed that as mass loss of steel was increased from 0 to 4%, the bond capacity increased by about 16%. They attributed this to an increase in the confinement of the concrete/steel interface due to expansive corrosion products. After the 4% corrosion, a sudden reduction in the bond was recorded so much that at a corrosion level of 10%, the bond strength was reduced by more than 80% of the bond-capacity of uncorroded specimens. This reduction was attributed to cracking of the cover concrete. After the 10% corrosion, very little change in bond strength was measured. This was ascribed to a much-reduced confinement of steel and concrete due to corrosion cracks such that any further corrosion did not reduce this confinement any further. Similar findings were recorded by Cabrera [36]. They however, found the peak in the bond strength to occur at mass loss of steel of about 1.8%.

From the viewpoint of structural mechanics, an increase in the bond between steel and concrete is expected to result in stiffer specimens. Earlier discussions however, indicated that despite steel corrosion increasing bond at low levels of steel corrosion, it does not always increase beam stiffness. This highlights the difficulty of using stiffness of corrosion-affected RC structures to indicate the level of steel corrosion. Certainly, this problem will be compounded when RC
members corrode whilst under a sustained load. More discussion on this is in the following sections as well as in Chapter Five.

2.4 Behaviour of RC beams corroded under load

The following section of the thesis discusses the influence of sustained service loads on the rate of steel corrosion as well as the behaviour of corroded specimens. It also provides a detailed discussion on the load frames used by various researchers. It should be noted that there are some research programmes in the literature where small RC specimens were corroded under load such as in [45,49]. The focus of this section (and the overall thesis) however, is on works where structural performance of RC structures was looked at during the corrosion process, and it was possible to repair and strengthen them.

2.4.1 Yoon et al. [5]

This work involved testing RC specimens of dimensions 100 x 150 x 1170 mm. Their specimens were tested whilst under a sustained load using a mechanical loading frame shown in Figure 2.4. Other specimens were tested in the absence of a sustained load but having been previously loaded. Others were tested in the absence of a sustained load with no previous loading.

Loading on beams that were tested under a sustained load was induced by hanging weights on a lever beam. 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. They were therefore 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. Rather than uniformly distributing corrosion products around the corroding steel as done by many other researchers [8,9,19], they contaminated only the tensile faces of their specimens. As discussed earlier, this process is more representative of in-service conditions. Their testing arrangement also exposed the tensile faces of beams where the severest level of steel 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 certainly made it easy to monitor various other parameters 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 test specimens.

Figure 2.4  Loading frame used by Yoon et al. [5]

The overall configuration of the loading system was such 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 20, 45, 60 and 75% of the ultimate capacity of a virgin beam. For specimens tested under 45 and 75% sustained loads, the testing process was divided into two stages; corrosion initiation stage; and corrosion propagation stage. A similar testing process was exercised on
beams that were preloaded before testing in the absence of a sustained load. Other specimens were only tested under the corrosion propagation stage.

During the corrosion initiation stage, beams were subjected to cycles of four days wetting with 3% solution of NaCl and three days drying in natural air. A half-cell potential was used to monitor corrosion initiation during this stage. The results showed beams that were previously loaded to high loads to have shorter corrosion initiation periods (two days and four days for beams previously loaded to 75 and 45% loads respectively). Similar results were observed on beams under sustained loads. A beam with a sustained load of 75% had corrosion initiation after three days compared to ten days on a beam with a sustained load of 45%. It is interesting to observe that the time to corrosion initiation on a beam that was previously loaded to 45% before being tested in the absence of a sustained load was 2.5 times less than the time to initiation on a beam that was tested under a sustained load of 45%. Wider flexural cracks to facilitate ingress of corrosion agents into the concrete were however, expected on a beam with a sustained load. These results reiterate the complexity of the corrosion process that was observed in specimens that were corroded in the absence of a sustained load. They also offer an interesting research area on the influence of sustained load on the time to corrosion initiation of steel that is embedded in concrete. The scope of this thesis is however, on behaviour of RC structures that are already-corroding. It should be mentioned though that since 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 Yoon et al. [5] may be insignificant.

After corrosion initiation, an impressed current density of 370 µA/cm² was applied to accelerate steel corrosion. This current density is above the upper limit current density recommended by El Maaddawy and Soudki [33]. The results obtained by the researchers are therefore not likely to be representative of in-service conditions. However, they should provide an indication of the effect of load on the rate of steel corrosion as well as the corresponding behaviour of concrete structures. During testing, researchers monitored daily, the galvanic current passing through corroding steel bars and converted it to mass loss of steel using Faraday’s Law. Unlike in the corrosion initiation stage, specimens were
always ponded with NaCl solution during the corrosion propagation stage. At mass losses of steel below 2%, the level of preload was found to have little effect on the rate of steel corrosion. However, after 2% mass loss, specimens with a preload of 75% exhibited corrosion rates that were about 30% larger than rates on specimens with a preload of 45%. Interestingly, for specimens under a sustained load, effect of loading on the rate of steel corrosion was distinct even at the early corrosion stages. Specimens with a 75% sustained load also exhibited corrosion rates that were about 30% larger than rates on specimens with a 45% sustained load. The actual values of steel losses on specimens under sustained loads were however, around 33% larger than on specimens with equivalent preloads. For example, a specimen with a preload of 75% had a mass loss of 4% after 18 days of accelerated corrosion compared to a mass loss of 6% on a specimen that was corroded under an equivalent sustained load. A specimen that was tested in the absence of a sustained load exhibited mass loss of steel that was up to three times less (1.3% after 17 days) than loss on specimens under load. It should be noted that the expected mass loss of steel after 18 days in all specimens was 4.5%. To enable a comparison of these rates with corresponding rates found on specimens that were tested in the absence of a sustained load, results from Yoon et al. [5] are plotted in Figure 2.3. The figure indicates that the differences between predicted and measured mass losses of steel from the researchers are within those for specimens tested in the absence of a sustained load. Results from Yoon et al. [5] are therefore not sufficient to ascertain the effect of load on the rate of steel corrosion.

Central deflections of specimens that were tested under a sustained load were continuously monitored during the testing process. Prior to accelerated corrosion, all specimens illustrated increased deflections with decreasing rates with time due to creep effects. Higher loaded specimens exhibited increases in both initial and long-term deflections. To be more specific, the initial respective central deflections of specimens were 0.8 and 1.4 mm for specimens with 45 and 75% sustained loads. After 50 days of testing without impressed current, specimens with a sustained load of 75% exhibited a central deflection of 2.5 mm compared to 1.4 mm on a specimen with a sustained load of 45%. This implies 80% increase in the deflection on a specimen with a 75% sustained load compared to 40%
increase on a specimen with a 45% load. Typical of creep in RC structures, 90% of these increases were observed within the first 20 days.

After 50 days of testing, an artificial current density of 370 µA/cm² was impressed on the specimens. An increase in deflections was immediately observed. It was however, only ten days later that a much more pronounced increase in deflections was recorded. Interestingly, little additional deflections were recorded after 70 days of testing. This was despite continued accelerated steel corrosion. From 60 to 70 days, respective increases in deflections were 20 and 16% for specimens under a load of 75 and 45%. The authors attributed the sudden increase in deflections after 60 days to cracking of the cover concrete. The deflections reaching a plateau after a certain level of corrosion was ascribed to a reduction in the pressure applied on the concrete by corrosion products due to a free discharge of the products via the corrosion cracks.

The sudden increase in deflections after accelerated corrosion shows that there was no gain in stiffness of specimens at the early corrosion stages. Furthermore, increase in deflections when impressing direct current after creep has stabilised at high sustained loads (up to 75% of the ultimate capacity) suggests that it is the corrosion process that controls deflections of beams corroded whilst under a sustained load and not creep. Most importantly, it implies that regardless of the level of sustained load, at the early corrosion stages, change in stiffness can be used to predict the level of steel corrosion. Deflections remaining constant after 70 days despite a continued increase in the level of steel corrosion shows that after a certain level of steel corrosion, it becomes difficult to use stiffness to indicate the level of steel corrosion. From the rates of steel corrosion discussed earlier, it can be shown that this critical level of corrosion ranged from 4 to 7%. Due to a limited number of tested specimens in Yoon et al. [5], it is important to further assess this critical level of steel corrosion.

It is evident that deflections and hence stiffness measured in Yoon et al. [5] are fundamentally different to those discussed on specimens that were tested in the absence of a sustained load. For specimens that were corroded in the absence of a sustained load, researchers assessed their behaviour at a chosen level of steel
corrosion when they were subjected to monotonically-increasing static loads. However, Yoon et al. [5] assessed structural behaviour when loads on specimens were kept constant but steel corrosion was varied. Their work is certainly more representative of in-service conditions. In addition, it gives the progression of change in structural behaviour with increase in the level of steel corrosion. To ascertain that there is difference in structural response from the testing procedures, Torres-Acosta et al. [37] found stiffness to linearly decrease with increase in the level of steel corrosion. Yoon et al. [5] however, found stiffness to only decrease at the early corrosion stages after which it becomes constant despite continued steel corrosion. This matter is discussed further in Chapter Five.

2.4.1.1 Critical discussion points on Yoon et al. [5]

Results from Yoon et al. [5] indicate that some behaviours of beams corroded under load are different from corresponding behaviours 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. Moreover, their results indicated that deflections were limited to indicating steel corrosion at low levels of corrosion. Other easy-to-measure properties of corroding RC structures such as corrosion crack widths and their propagation were not monitored during the test programme.

2. Yoon et al. [5] indirectly measured mass loss of steel by monitoring current that passed through corroding bars and converted it to mass loss using Faraday’s Law. To ascertain the effect of load on the rate of steel corrosion, it was necessary to verify the calculated corrosion rates with actual mass loss of steel bars.

2.4.2 Ballim et al. [6,7]

The programme involved testing eight RC specimens of dimensions 100 x 160 x 1550 mm. They were reinforced with a single 16 mm-diameter steel bar in tension
with 32 mm cover to the tensile face and 42 mm cover to the side faces. Shear reinforcement was 8 mm-diameter stirrups which were spaced at 60 mm centre-to-centre and was limited to the shear span of beams. Their specimens 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 2.5.

Contrary to many studies discussed earlier where de-passivation of steel was accelerated by adding chlorides to the concrete mix, Ballim et al. [6,7] used carbonation-induced de-passivation of steel. This entailed placing specimens in a carbonation chamber at a pressure of 80 kPa for six weeks. It is however, not clear how this process of steel de-passivation affects the rate of steel corrosion compared to the previous methods. Following de-passivation of steel, they accelerated corrosion propagation by placing the samples in 3% solution of NaCl and by impressing a current density of 400 \( \mu A/cm^2 \). As shown in Figure 2.5, ponding of samples in salt solution meant that the bottom supports of the beams were also immersed in the solution.

![Figure 2.5](image.png)

**Figure 2.5** Loading frame used by Ballim et al [6,7]

Ballim et al. [6,7] divided their work into two sets with equal number of test beams (four each); set 1 beams were tested under a load equivalent to 23% of the
ultimate capacity of an uncorroded beam; and set 2 beams were tested under a load equivalent to 34% of the ultimate capacity. In each set, three beams were corroded whilst one was used as a control. Central deflections were measured on beams during the corrosion process. At the end of corrosion testing, beams were removed from their respective loading frames and corrosion crack maps were prepared. After testing beams to failure, corroded bars were retrieved from the concrete, cleaned and weighed to determine their mass loss.

They found Faraday’s Law to generally overestimate the level of steel corrosion, especially at low levels of sustained loads. For beams under a sustained load of 23%, Faraday’s Law overestimated their findings by 47% compared to an overestimate of 39% on beams corroded under a load of 34%. Despite these large errors, Figure 2.3 shows that their results were still comparable to those found on specimens that were tested in the absence of a sustained load. Therefore, sustained loads had little influence on the rate of steel corrosion. It is however, noted in their work that in addition to corroding steel bars, beam supports were also corroded. The researchers contended that some of the applied current that was intended to corrode steel bars was lost to corrosion of the test rig. If this was avoided, they foresaw obtaining larger rates of steel corrosion.

Regardless of the level of the sustained load, the majority of beams exhibited a single crack on their tensile faces that propagated parallel to the corroding steel bar. In addition to the main crack, discrete cracks on the side faces of beams were observed. It should be mentioned though that one of the beams under a load of 34% exhibited cracks only on the side faces that propagated near and parallel to the tensile steel bar. These crack patterns are similar to those discussed previously on beams that were tested in the absence of a sustained load. It therefore suggests that crack patterns due to steel corrosion are not affected by the level of sustained loads. This is expected as external applied flexural loads primarily induce longitudinal stresses on the specimens compared to transverse and vertical stresses from steel corrosion.

Maximum crack widths on all beams were observed at the beam ends. Since corrosion was accelerated and uniformly distributed through the entire corrosion
region, wider cracks at the ends can be attributed to low confinement of the concrete. The largest confinement from the closely-spaced stirrups was expected within the shear span. Despite not having stirrups, the constant moment region was probably more confined than the far ends because it’s furthest point from a stirrup was 175 mm compared to 200 mm for the beam ends. Beams under a sustained load of 23% had maximum crack widths ranging from 1.21 to 2.28 mm compared to a range of 1.32 to 4 mm on beams under 34% loads. Since the range of maximum crack widths on beams under different sustained loads overlapped, it is not obvious that larger crack widths belonged with beams under larger sustained loads as contented by the researchers. It is interesting to note that similar specimens under the same test regime can have maximum crack widths that are more than two times apart.

Whilst Ballim et al. [6,7] observed maximum crack widths at the beam ends, for in-service structures, steel corrosion is mostly expected near flexural cracks because they allow an easier ingress of corrosion agents. If corrosion cracks around the flexural cracked region of beams are associated with the measured mass losses of steel, for specimens under 23% load, 1% mass loss of steel corresponded to a maximum crack width of 0.15 mm. Similarly, for specimens under a load of 34%, 1% mass loss of steel corresponded to a maximum crack width of 0.13 mm. These values were calculated as an average of the three specimens in each set. Interestingly, these relations are about similar to those obtained when specimens were corroded in the absence of a sustained load. They therefore indicate little influence of load on the rate of steel corrosion as well as on the rate of widening of corrosion cracks.

As expected and consistent with studies by Yoon et al. [5], beams tested under high load levels were found to exhibit larger central deflections. An uncorroded beam with a load of 34% had initial deflection of 1.33 mm which increased to 1.5 mm (13% increase) after 30 days. For a beam with 23% load, initial deflection was 0.76 mm and deflection after 30 days was 1.1 mm (30% increase). Surprisingly, beams that were corroded under a load of 23% exhibited an increase in deflections of 27% within three days of accelerated steel corrosion. For beams under a sustained load of 34%, the increase in deflections after three days of
accelerated steel corrosion was 50%. This confirms results from Yoon et al. [5] that deflections of structural members under a sustained load are mostly controlled by the corrosion process and not creep. Furthermore, mass loss of steel after three days was expected to be around 0.6%. Clearly the mass loss of steel was not proportional to the increase in deflections. The authors therefore attributed increased deflections to the loss in bond between steel and concrete due to cracking of the cover concrete. They however, could not validate this since their specimens were immersed in salt solution which hindered them from observing corrosion cracks. Another interesting phenomenon from their results which was also observed by Yoon et al. [5], is that after a certain level of steel corrosion, deflections ceased to increase despite a continued increase in the level of steel corrosion. Assuming a linear loss of steel during the testing period, this critical level of steel corrosion was about 5%. This work, again in consent with Yoon et al. [5] indicates that change in stiffness of RC beams due to steel corrosion is limited to indicating level of steel corrosion below 5%. Intriguingly, this critical level of steel corrosion was found to be independent of the level of the sustained load.

2.4.2.1 Critical discussion points on Ballim et al [6,7]

This work provided further insight on the behaviour of beams corroded under a sustained load. However, some of the results obtained are questionable due to the nature of the testing programme.
1. As stated in their work, there was corrosion of the test 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 beams 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 series 1 and 2 lost loads of about 0.6 kN and 1 kN respectively. In terms of the spring accuracy (0.05 kN), these loads are significant. It was therefore necessary to constantly adjust the applied loads.
3. Even though deflections were found to increase with an increase in the degree of corrosion and the level of the applied load, other parameters of structural
behaviour such as the variation of strains were not 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.

4. Crack maps were drawn 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. It is therefore difficult to associate the obtained crack maps with the service life of corroding RC structures.

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

2.4.3 El Maaddawy et al. [8,9]

This work involved corroding fifteen quasi-full-scale RC beams with dimensions of 152 x 254 x 3200 mm to four different accelerated corrosion periods of 50, 110, 210 and 310 days. Their specimens were reinforced with two 16 mm-diameter bars in tension. The shear reinforcement was 8 mm-diameter stirrups spaced at 80 mm centre-to-centre in the shear span and 333 mm centre-to-centre in the constant moment region. Eight beams were tested under a sustained load equivalent to 60% of the ultimate load of a virgin beam whilst corresponding seven 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 2.6. 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 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.
To selectively contaminate steel, 2.25% chlorides by weight of cement were added to the concrete mixture used to cast the middle 1400 mm of the specimen and to a height of 100 mm from the tensile face of the beam. The contaminated region is indicated in Figure 2.6. A current density of 150 $\mu$A/cm$^2$ was used to accelerate steel corrosion. During accelerated corrosion, specimens were constantly sprayed with mist to supply water and air necessary for corrosion.

At the early testing stages, beams were constantly monitored for the appearance of the first corrosion cracks. It was found that for beams corroded under load, corrosion cracks appeared after 53 hrs whilst for beams corroded in the absence of a sustained load, cracks were observed after 95 hrs. It is worth noting that the difference between 95 hrs and 53 hrs seem small but beams in the programme were corroded at a corrosion density that is up to a thousand times more than the corrosion density of in-service structures [18,23,50]. Since they used current density below 200 $\mu$A/cm$^2$, results from El Maaddawy and Soudki [33] suggest that the corrosion damage they induced was proportional to damage on in-service structures. Therefore the 50% reduction of the time to cover cracking due to a
sustained load is very significant for in-service structures where the time to cover cracking is in the order of tens of years. However, from section 2.3.1, and particularly studies by El Maaddawy and Soudki [29], the variation in the thickness of the porous zone (amongst many other parameters) was found to be capable of causing a similar range in the time to cover cracking as found above. It is therefore difficult to attribute the range in the time to cover cracking from El Maaddawy et al. [8,9] to the sustained load.

Following the appearance of the first visible corrosion cracks, the rate of widening of cracks at the middle of two beams that were intended to have the highest level of corrosion (310 days testing) was continuously monitored using a demec gauge. They reported a bi-linear rate of widening of corrosion cracks with time. During the first 50 days of accelerated corrosion, beams under a sustained load had a rate of widening of corrosion cracks of 22 µm/day compared to 18 µm/day for beams corroded in the absence of a sustained load. After 50 days, the rate of widening of corrosion cracks was about 7 µm/day for both loading regimens. From the variation of crack widths between similar specimens found by other researchers such as Ballim et al. [6,7], these differences in the rate of corrosion crack widening are expected. In addition, since cracks were only continuously monitored on two beams and at a single point on each beam, it is not clear if other beams exhibited the same rate of crack widening.

At the end of the test, the researchers prepared crack maps of the corroded beams. Each beam had two longitudinal cracks on the tensile face and near each corroding bar. Discrete cracks were observed on the side faces of some beams and they were mostly pronounced on heavily corroded beams. From corrosion crack patterns discussed earlier, and the fact that some specimens were corroded in the absence of a sustained load, it is evident that the sustained load had little influence on the pattern of corrosion cracks. This was also found by Ballim et al. [6,7].

In drawing crack patterns, El Maaddawy et al. [8,9], meticulously measured corrosion crack widths at 100 mm intervals along the beam span. Contrary to Ballim et al. [6,7] where maximum crack widths were found at the beam ends, they observed maximum crack widths within 200 mm of the centre of the
corrosion region. This emphasises the earlier notion that corrosion crack widths are influenced by the confinement of concrete. In this case, more confinement was expected at the ends of the corrosion region from closely-spaced stirrups as well as the mass of the undamaged concrete. Maximum corrosion crack widths not necessarily being at the centre of the corrosion region implies that measuring crack widening only at the middle of the beam as done by the researchers does not always provide the maximum rate of widening of corrosion cracks. In fact the maximum crack widths on the specimens which they reported the rate of widening of corrosion cracks were 100 and 200 mm from the centre of the corrosion region. It was therefore essential for them to monitor the rate of widening of corrosion cracks at different points along the corrosion region. This procedure and the corresponding results are discussed in Chapters Three and Six.

After testing beams to failure, six coupons, three from each bar, each 30 to 50 mm in length were extracted from the corrosion zone. They were then cleaned and weighed to determine the mass loss of steel. Specimens that were corroded in the absence of a sustained load had average mass losses of steel of 8.7, 14.2, 22.2 and 31.6% for 50, 110, 210 and 310 days respectively, of accelerated corrosion. The corresponding mass losses of steel for specimens corroded under load were 9.6, 15.6, 23.3 and 30.5%. Note that the expected respective levels of steel corrosion from Faraday’s Law were 6.1, 13.3, 25.4 and 37.5%. At mass losses of steel below 15%, the mass losses of steel in beams corroded under load were around 10% larger than corresponding losses of steel in beams corroded in the absence of a sustained load. For these mass losses of steel, Faraday’s Law underestimated the measured mass loss of steel by up to 36%. At higher mass losses of steel (>20%) however, the influence of sustained load on the rate of steel corrosion was negligible. To assess the magnitude of the differences in mass losses and hence the effect of sustained load on the rate of steel corrosion, these mass losses were plotted in Figure 2.3. Similar to results from Yoon et al. [5] as well as Ballim et al. [6,7], the figure shows that there was little noticeable effect of sustained load on the rate of steel corrosion. Even the errors from Faraday’s Law (-3.5 to 7%) are within those found by other researchers on specimens that were corroded in the absence of a sustained load (-6.7 to 23.9%).
The relations between mass losses of steel and maximum crack widths from various beams were found to be the same. A 1% mass loss of steel corresponded to a maximum crack width of 0.1 mm. This relation is similar to those discussed earlier on specimens that were corroded in the absence of a sustained load by various researchers. It therefore emphasises the earlier notion that the level of sustained load had little effect of the rate of widening of corrosion cracks as well as the rate of steel corrosion.

After corrosion testing, they tested some of their specimens to failure. Other specimens were repaired prior to ultimate load test. The interest here is on specimens that were not repaired. Results on repaired specimens will be discussed in later sections of the chapter. Even though it was not reported in their work, their results indicated that ultimate load-bearing capacity of the specimens linearly reduced ($R^2 \approx 1$) with an increase in the level of steel corrosion. A 1% mass loss of steel corresponded to 1.1% loss in the load-bearing capacity of beams. These findings are consistent with those found by other researchers where load-bearing capacity was related to average loss in the area of steel. Since the residual capacity of beams were on the same curve regardless of the test conditions, residual capacity of corrosion-affected RC structures was affected by the level of steel corrosion and not the procedure of steel corrosion. This implies that relations between ultimate capacity and actual steel loss from accelerated laboratory tests are representative of relations on in-service structures.

2.4.3.1 Critical discussion points on El Maaddawy et al [8,9]

Clearly this work was more detailed than the previous two works [5-7] on the effects of loading on the behaviour 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 and to extensively monitor corrosion cracks.
2. Whilst the rate of widening of corrosion cracks was continuously monitored at the centre 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 maximum crack widths did not necessarily occur at the centre of the corrosion region. Furthermore, the results showed that some beams had corrosion cracks on the side faces in addition to cracks on the tensile face. Since the variation of widths of corrosion cracks on the side faces of beams was not monitored, it is difficult to relate the cracks on side faces with the cracks on the tensile face. It will however be shown in Chapter Six that this relation is important when designing an intervention scheme that is aimed at reducing the rate of corrosion crack widening.

3. Even though mass loss of steel was measured at different sections 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 of steel 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 of steel. This relation is however, critical for associating ultimate capacity with measurable properties of a corroding structure as was shown by Torres-Acosta et al. [35] and will be further discussed in Chapter Four.

2.4.4 Vidal et al. [10-15]

In this work, 36 quasi-full-scale RC beams (150 x 280 x 3000 mm) were tested under a sustained load. In addition to these beams, 36 more beams were used as controls and were not corroded. Their beams were of two main types; A and B. Beam A had a cover of 48 mm and was reinforced with two 16 mm-diameter bars on the tensile face. Beam B had a cover of 16 mm and was reinforced with two 12 mm-diameter bars on the tensile face. Test specimens were further divided into series 1 and series 2 depending on the level of the sustained load. Beams in Series 1 were loaded to 50% of the failure load of a control beam whilst series 2 beams were loaded to 80% of the failure load. For each beam type, the entire beam was reinforced with shear stirrups at a centre-to-centre spacing of 220 mm. The load on beams was applied as a three-point flexure by placing beams back-to-back horizontally. Similar to works by El Maaddawy et al. [8,9], bottom beams were
tested with the tensile face up whilst top beams were tested with the tensile face down. The beams were attached to each other at the middle as shown in Figure 2.7.

The accelerated corrosion process was achieved by:

- 0 to 6 years: continuous spraying of beams with 35 g/l NaCl solution under laboratory conditions (T ≈ 20 °C)
- 6 to 9 years: cyclic spraying of beams with NaCl solution under laboratory conditions (T ≈ 20 °C), one week of spraying and one week of drying
- 9 to 19 years: cyclic spraying, one week of spraying of beams with NaCl solution and one week of drying. The confined room was however, transferred outside, so the beams were exposed to the temperature of the south-west of France climate, ranging from −5 °C to 35 °C

Figure 2.7  Loading frame used by Vidal et al [10-15]
- 19 years to now: stopped spraying beams with NaCl solution, beams were unloaded from their respective test frames and allowed to corrode naturally in the absence of a sustained load.

During the corrosion process, various measurable parameters were periodically measured on the specimens. This thesis will discuss parameters measured after four years when steel corrosion initiated. Even more relevant parameters to this thesis are after six years when first visible corrosion cracks were observed. These parameters were; detailed crack maps; mechanical tests where beams were removed from their loading frames and loaded on a separate frame to 50% of their ultimate capacity; and ultimate failure load. Reported results from the researchers are mostly on type B beams (with 16 mm cover and reinforced with 12 mm-diameter bars).

As expected, after first cracking of beams (six years from the start of the test), all specimens exhibited an increase in the severity of cracking with time. Furthermore, in all beams, two major corrosion cracks on the tensile face of the beams that propagated parallel to the tensile steel bars were observed. Major cracks on some beams were also observed on the side faces and also parallel to the tensile steel bars. Few corrosion cracks were observed on the compression face. Corrosion crack patterns within the tensile region of beams are similar to those observed on specimens that were corroded in the absence of a sustained load. In agreement with work from Ballim et al. [6,7] as well as El Maaddawy and Soudki [8,9], these results further-confirm that corrosion crack patterns are not influenced by the level of the sustained load.

The rate of widening of corrosion cracks varied extensively along the beams. For example; after 14 years of testing, a beam with 16 mm cover and loaded to 50% (type B/surface 1) exhibited a maximum crack width of 2 mm on the tensile face and about 700 mm from the middle of the beam. Interestingly, the location of the maximum crack width was within the flexural cracked region. After 19 years of testing, the maximum crack width on the specimens was 3 mm still on the tensile face but now at the centre of the beam. Intriguingly, this crack was only 0.7 mm wide after 14 years. The widest crack after 14 years was 2.5 mm wide after 19
years. Therefore, from 6 to 14 years and from 14 to 19 years the beam exhibited a respective maximum rate of widening of corrosion cracks of 0.68 and 1.26 µm/day. After 23 years of testing, the specimen had a maximum crack width of 3.3 mm on the tensile face but at a location that had a crack width of 2.1 mm after 19 years (500 mm from the centre of the beam). This indicates that from 19 to 23 years, the maximum rate of widening of corrosion cracks was 0.82 µm/day. A similar beam exhibited a maximum rate of widening of corrosion cracks of 1.6 µm/day from 14 to 17 years. It was also from a crack that was on the tensile face but near the end support. From 17 to 23 years, the beam had a maximum rate of crack widening of 0.73 µm/day. This was still on the tensile face but near the opposite support to where the maximum crack was previously observed.

Since these results are from a limited number of specimens, it is difficult to explain the various changes in the rate of widening of corrosion cracks. It is however, worth pointing out that after 19 years of testing, the specimens were removed from their respective loading frames. Moreover, when a crack on the tensile face was actively widening, the rate of widening of an adjacent crack on the side face was relatively low. This indicates that the rate of widening of corrosion cracks can be attributed to the level of the sustained load as well as the crack pattern. Further discussion on this is in Chapter Six.

It is important to point out that flexural cracks were observed within the 1400 mm of the middle of the specimens. Under natural conditions, the majority of corrosion cracks would be expected within this region since flexural cracks allow an easier ingress of corrosion agents. The researchers however, constantly and uniformly sprayed their specimens with chlorides. Even though this procedure is not as aggressive as impressing an external current density, it is still likely to have contaminated steel even on areas with no flexural cracks. In corroboration, their specimens exhibited corrosion cracks near compression steel bars. Since confinement of concrete was about the same through the entire beam span due to the arrangement of stirrups, maximum corrosion cracks were to be expected anywhere along the beam.
Whilst the researchers corroded 36 beams, only two beams (both type B) were tested to failure; one was tested after 14 years whilst the other was tested after 23 years. Following the ultimate test, corroded bars were removed from the concrete, cleaned and weighed to determine mass loss of steel. They recorded mass loss of steel of 35% for a beam that was corroded for 14 years. Assuming mass loss of steel linearly increased after corrosion initiation (four years), this mass loss implies a rate of loss of steel of 0.085 g/day/m of bar length. From Faraday’s Law, it converts to a current density of 9 µA/cm². This current density lies within the range of current densities of 0.1 to 100 µA/cm² found in in-service structures by Andrade et al. [50]. The mass loss of steel after 23 years was found to be 45%. Assuming a constant rate of steel loss and that specimens had the same rate of steel loss, this indicates a loss of steel of 0.027 g/day/m of bar length from 14 years to 23 years. Again assuming Faraday’s Law, it converts to a current density of 3.5 µA/cm².

In relation to maximum crack widths, a beam that was tested to failure after 14 years had a mass loss of steel of 1% corresponding to a maximum crack width of 0.04 mm. For a beam that was tested to failure after 23 years, a mass loss of steel of 1% corresponded to a maximum crack width of 0.07 mm. Interestingly, these relations are similar to those found by other researchers where steel corrosion was assessed on RC specimens in the absence of a sustained load. In terms of residual load-bearing capacity, they found a mass loss of steel of 35% to correspond to loss in capacity of 20% whilst a mass loss of steel of 45% corresponded to loss in capacity of 40%. Again, these relations are similar to those discussed previously on specimens that were tested in the absence of a sustained load. This was despite using a much lesser rate of steel corrosion. Their results are therefore in consent with work from El Maaddawy and Soudki [33] that current densities below 200 µA/cm² are representative of natural steel corrosion.

In addition to measuring deflections when beams were tested to failure, the researchers occasionally loaded their specimens to service loads of 50% of ultimate capacity of an uncorroded beam to determine the loss in stiffness due to steel corrosion. A control beam from type B beams had a central deflection of 1.5 mm at a load of 50% of its ultimate capacity. One corroded beam (type B),
exhibited a central deflection of 1.75, 2.6 and 2.7 mm after 14, 19 and 23 years, respectively, of corrosion testing. Similar results were found on other beams. This is an interesting finding which indicates that up to 14 years there was a loss in bending stiffness of about 17% compared to a loss of 73% after 19 years. Intriguingly, from 19 to 23 years, very little loss in bending stiffness was recorded. As discussed earlier, this was despite a continued loss in steel. Assuming a constant loss in area of steel, mass loss at 19 years was 40%. Therefore, they found stiffness to reduce by 17% when mass loss of steel reduced by 35%. From then, stiffness reduced by 48% when mass loss of steel only reduced by 8%. Relations from 14 to 19 years are calculated relative to stiffness and remaining bar diameter at 14 years.

As previously noted, their procedure for measuring stiffness is similar to that used by researchers who tested their specimens in the absence of a sustained load. The large mass loss of steel (40%) at which stiffness remained constant despite continued steel corrosion probably explains why previous researchers failed to recognise this phenomenon. It was also discussed earlier that this procedure is not representative of in-service conditions. The researchers should have continuously monitored deflections during the corrosion process as done by Yoon et al. [5] and Ballim et al. [6,7].

2.4.4.1 Critical discussion points on Vidal et al [10-15]

Of the above-discussed programmes, this experimental programme is closest to conditions of in-service structures primarily because corrosion was natural and over a long period (up to 23 years). Its major drawbacks are;
1. Most of the structural behaviour on this test involved removing the beams from the loading systems and testing them on a different system. It is as such difficult to appreciate the effects of continued steel corrosion at a chosen load on the behaviour of RC beams.
2. The loading system they used is not a mechanism such that it did not self-adjust as the beams deflected.
3. The tensile face of beams was hidden which made it difficult to continuously monitor corrosion damage on the extreme tensile face of the beams or to later use the frame to repair the beams whilst under load.

4. Despite testing beams for 23 years, corrosion crack widths were recorded at discrete times (14, 17, 19 and 23 years). It is therefore difficult to predict accurately, the rate of widening of corrosion cracks.

2.5 Behaviour of corroded RC structures after patch repairs

Patch repairs on corroded RC structures are commonly undertaken to maintain or restore the serviceability of the structures [18]. To avoid aggravation of the public due to pre-mature failure of repairs, even though it is still common, there is extensive research that has been carried out to establish suitable repairs as well as ‘best’ practices to repair damaged concrete. It is widely accepted that the most critical element to consider in repairs is the compatibility of repair materials with the substrate concrete. The more important of these factors are dimensional (modulus of elasticity and shrinkage) and electrochemical compatibilities.

When a RC structure suffering from steel corrosion is repaired, some of the chloride contaminated concrete may be left in place. The hardened repair phase (from modern repair mortars) often offers more resistance to steel corrosion compared to the substrate concrete. It has been shown by many researchers such as Raupach [4] that in this case, part of the bar that is in the repair material becomes cathodic whilst the part in the substrate concrete becomes anodic and may corrode further. According to Vaysburd and Emmons [17], there is a widespread problem of corrosion in concrete repairs of in-service structures. To avoid further steel corrosion after repairs due to electrochemical incompatibility of repairs and the substrate concrete, manufacturers of materials offer various corrosion protective systems. These include cathodic protection, corrosion-inhibiting admixtures and electrochemical/physical barriers between steel bars and repairs. If properly used, these protective systems have been shown to be very effective. The problem of further steel corrosion due to electrochemical incompatibility of repairs and the substrate concrete will therefore not be
discussed further is this thesis, except for a few comments in Chapters Three and Four.

If repairs are stress-carrying, Emberson and Mays [51] have shown that compatibility of the modulus of elasticity between repairs and the substrate concrete becomes vital. Differences in modulus of elasticity causes unequal load sharing between repairs and the substrate concrete resulting in localised interface stresses. This may result in debonding of the repairs. Similar to protective systems, the manufacturing industry offers a wide range of repair mortars. It is therefore relatively easy to choose a repair mortar which when hardened, will have modulus of elasticity that is compatible to that of the substrate concrete. Bonding agents are also available to supplement the bond between repairs to steel bars and repairs to the substrate concrete. If repair mortars and bonding agents (if needed) are carefully selected, and additional stresses are not induced on the structure until repairs have fully-hardened, incompatibilities due to modulus of elasticity and their effects can be minimised. This incompatibility therefore does not require any further discussion.

Incompatibility of repairs and substrate concretes that offers the most problem to structural engineers is drying shrinkage. It is mainly because repairs involve joining an initially semi-liquid substance (which must shrink during hardening) with an already-hardened substance (which is now free from shrinking). Free shrinkage strains of repair mortars up to 1500 micro strains have been reported [52]. Even the ‘low shrinking’ repairs available in the industry also shrink significantly more than the substrate concrete. When joined with concrete, the substrate concrete offers restraint to this shrinkage which often results in cracking of the repairs. If repair was for a corrosion-damaged RC structure, these cracks allow more ingress of corrosion agents which can speed up the corrosion process. Moreover, it is foreseeable and has been shown in experiments [53] that if concrete is to offer restraint to shrinkage of repairs, these shrinkage forces will also distress the substrate concrete. How these distresses affect the behaviour of the resulting composite structural element (substrate concrete and repair mortar) is unclear. Undoubtedly, the effect of stresses applied in the substrate concrete due to shrinkage of repairs will be appreciated the most when repairs are carried out
under load. This is because repairs are often carried out on the tensile region of the substrate concrete (with residual tensile stresses) and shrinkage stresses from repairs are compressive. Unfortunately, there is little data in the literature on this. Chapter Five offers an in-depth discussion on this matter.

Whilst patch repairs on real structures are often carried out on corrosion-damaged RC structures, there is a paucity of research on the structural performance of corroded structures after patch repairs. However, one interesting work on this was carried out by Rio et al. [18]. They corroded RC specimens (200 x 120 x 2000 mm) to various levels of steel corrosion. Steel reinforcement was three 12 mm-diameter bars in tension with a cover of 30 mm and two 8 mm-diameter bars in compression. Shear reinforcement with stirrups was limited to the shear span. Corrosion was on tensile steel bars and was restricted to a length of 300 mm of the middle of the beam by only contaminating that region with chlorides. It was accelerated by impressing a current density of 100 µA/cm². At the end of the corrosion process, they removed the damaged concrete to a depth of 50 to 70 mm and to a length of 65 to 180 mm beyond each end of the corroded region. Penetration of reinforcement corrosion on steel bars was measured at different sections along the corroded bar and used to calculate the smallest remaining bar diameter/maximum loss in cross-sectional area of steel. Concrete was then replaced with a cement based mortar with modulus of elasticity that was almost equal to that of the substrate concrete. To assess the effectiveness of repairs, some specimens were corroded and not repaired whilst one was not corroded to act as a control. All specimens were finally tested to failure.

Flexural cracks on the control beam were first observed at a load of about 33% of the load-bearing capacity of the beam. The absence of transverse reinforcement stirrups on the middle part of the beam caused pre-mature buckling of the compression reinforcement. Expectedly, corroded but non-repaired beams followed the same load-deflection trend as the control beam. However, at the same applied load, they exhibited more deflections than the control beam. In agreement with work from other researchers such as Cabrera [36] and Mangat and Elgarf [19], steel corrosion therefore reduced the stiffness of beams. Interestingly, patch repaired beams were found to be stiffer than the control beam. Furthermore,
first flexural cracks on repaired beams were observed at higher load values than the control beam (40 to 45% of the ultimate load). It is surprising how patch repairs with the same modulus of elasticity as the substrate concrete and carried out on beams with lesser area of steel than the control beam tend to produce stiffer beams than the control beam. It will be shown in Chapter Five that this is an overvalue of performance of repairs that are carried out under load.

It should be pointed out that none of patch repaired beams showed debonding of the repair mortar at failure. This confirms the earlier notion that incompatibility of the modulus of elasticity between repairs and the substrate concrete is avoidable. The ultimate capacity of repaired beams with maximum loss in cross-section of steel of 9% was about the same as the capacity of the control beam. Since design of the beams was such that failure was buckling of compression reinforcement, it is difficult to appreciate the effect of loss in the area of steel on the capacity of beams, let alone the effect of the patch repairs. In corroboration, one beam that was corroded to maximum loss of steel of 15% and not repaired had a load-bearing capacity that was similar to the capacity of the control beam. Previous discussion (section 2.3.4) however, indicated that the load-bearing capacity of the corroded beam was supposed to have reduced by about 10%, especially that steel corrosion was within the region of maximum applied stresses.

Another work on the effect of repairs on the behaviour of corroded specimens was carried out by Mangat and Elgarf [54]. They corroded RC beams and repaired them with two different mortars. The 28-days modulus of elasticity of the substrate concrete, mortar 1 and mortar 2 were 22, 19 and 32 GPa, respectively. Their results that are relevant to this thesis are when they assessed the effect of repairs on the failure load of corroded beams at a permissible deflection (span/250). The control beam (uncorroded) had a failure load of 88 kN. When a beam was corroded to 5% mass loss of steel and repaired with mortar 1, the capacity reduced to 55 kN (38% loss). The corresponding loss in capacity when mortar 2 was used was only 9%. At a mass loss of steel of 10%, reductions in capacities after repairs with mortars 1 and 2 were 45 and 15%, respectively. Similar to findings from Rio et al. [18], they did not observe debonding of patch repairs.
These results indicate that at low levels of steel corrosion, a careful selection of repairs can reduce the loss in the load-bearing capacity of corroded beams. Expectedly, at larger losses of steel, repairs do not enhance the load-bearing capacity of corroded beams. It is therefore necessary to strengthen them. Amongst various strengthening techniques in the literature as well as in practice (bonded steel plates, steel jackets and RC jackets), fibre reinforced polymers (FRPs) bonded to external faces of damaged RC structures has recently emerged as the state-of-the-art strengthening technique [20,21]. This is primarily because FRPs have superior advantages (such as handling, durability and improved mechanical properties) over the previous methods.

It is worth noting that whilst in-service structures are often repaired under load, no literature was found on corroded RC specimens that were patch repaired under load. There is little doubt that patch repairing laboratory RC specimens under load is a demanding experimental procedure. Moreover, test frames discussed earlier to corrode RC beams under load did not allow for an easy repair of the specimens under load. If however, the effect of load has a great influence on the behaviour of repaired structures, the usefulness of results obtained on specimens that were repaired in its absence becomes low. Behaviour of corroded RC specimens that are patch repaired under load is discussed in Chapter Five.

### 2.6 Behaviour of corroded RC structures after FRP repairs

Fibre reinforced polymer (FRP) plates bonded to the tensile face of a RC beam have been shown to be able to increase its flexural capacity by an excess of 100% [55]. This is however, thwarted by additional failure modes that the strengthening process often brings out. In addition to the normal failure modes of RC beams in flexure (crushing of concrete in compression; yielding of tensile steel; buckling of compression steel; and shear), FRP-strengthened RC beams may fail by rupture of FRP plates and by debonding of FRP plates. From experimental observations, the majority of failures are debonding of FRP plates [56]. Many techniques have therefore been developed to prevent this pre-mature debonding of FRP plates. The most common is wrapping them and the RC member with FRP sheets.
Unfortunately, debonding of FRP plates that are wrapped with FRP sheets has also been reported [55].

In design and analysis of FRP-strengthened RC beams, it is essential that the bond capacity of FRPs be known accurately. Many debonding models have as such been developed and are well reviewed amongst many others in Smith and Teng [57], Toutanji et al. [58] and Aram et al. [59]. The majority of them are too complex to use by practicing engineers. The model developed by ACI Committee 440 [20] is probably the easiest to follow. Interestingly, Toutanji et al. [58] showed that in addition to being simple, it gives predictions that are more accurate compared to other models. Therefore, in modeling effectiveness of FRPs to increase the load-bearing capacity of corroded RC beams in this thesis (Chapter Four) only the ACI Committee 440 debonding model will be used.

According to ACI Committee 440 [20] the load-bearing capacity of FRP-strengthened RC beams should be based on a usable or debonding FRP strain rather than the ultimate strain of FRPs provided by manufactures or obtained from laboratory tests. This debonding strain, $\epsilon_{df}$, is a product of the ultimate strain of FRPs, $\epsilon_{fu}$, and an empirical factor, $k_m$, found from extensive laboratory tests. As indicated in equation 2.15, this factor is a function of the stiffness of plates ($n_f t_f E_f$) as well as the rupture strain of the FRPs, and it must be less than 0.9.

$$
k_m = \begin{cases} 
\frac{1}{60 \epsilon_{fu}} \left( \frac{1}{90000} - \frac{n_f t_f E_f}{360000} \right) \leq 0.9 & \text{for } n_f t_f E_f \leq 180000 \\
\frac{1}{60 \epsilon_{fu}} \left( \frac{90000}{n_f t_f E_f} \right) \leq 0.9 & \text{for } n_f t_f E_f > 180000 
\end{cases} \tag{2.15}
$$

$$
\epsilon_{df} = k_m \epsilon_{fu} \tag{2.16}
$$

Where; $\epsilon_{fu} =$ ultimate strain of FRPs; $n_f =$ number of FRP plies; $t_f =$ thickness of each ply (mm); $E_f =$ modulus of elasticity of FRPs (MPa); and $\epsilon_{df} =$ debonding or usable strains of FRPs.
Even with debonding being a problem, strengthening corrosion-damaged RC beams with FRPs has been shown to significantly increase their load-bearing capacity. For example; after corroding RC beams, El Maaddawy et al. [9] (discussed earlier) strengthened some of their specimens with FRPs. Their strengthening scheme for flexure was a FRP sheet bonded to the tensile face of a beam. To preclude debonding of FRPs, strengthened beams were wrapped with U-shaped FRP sheets. They found strengthening an uncorroded beam to increase its load-bearing capacity by 50%. The load-bearing capacity of RC beams that were strengthened after steel corrosion linearly decreased ($R^2 = 0.83$) with an increase in the level of corrosion. From this trend, 1% mass loss of steel corresponded to 0.4% loss in the load-bearing capacity of a strengthened beam. Compared to the relation between load-bearing capacity of corroded but unstrengthened beams (1% mass loss to 1.1% loss in load-bearing capacity), these results confirm that FRPs are very effective at increasing the load-bearing capacity of corroded beams. Interestingly, it can be shown from their results that for a strengthened beam after steel corrosion to have a capacity that is equal to the capacity of an uncorroded beam, the mass loss of steel must be about 80%. This is because in addition to increasing the capacity of an uncorroded beam (50% increase), the rate of loss of load-bearing capacity of strengthened beams in terms of mass loss of steel is much lower (3 times less) than the corresponding rate of unstrengthened beams.

It is important to mention that in the majority of experiments that were used to evaluate the effectiveness of FRPs to increase the load-bearing capacity of corroded RC structures, specimens were immediately tested to failure after strengthening with FRPs. If RC structures were to further corrode, which is a high possibility in in-service structures, expansive forces from corrosion products as well as concrete cover cracking may cause pre-mature debonding of FRPs. The effectiveness of FRPs to increase the capacity of corroded RC structures may therefore not be as high as reported. It is therefore important to understand the potential of FRPs to prevent further steel corrosion.

One such work was conducted by Badawi and Soudki [34]. They used RC specimens as well as an accelerated corrosion process that was identical to that
used by El Maaddawy et al. [8,9] (discussed in section 2.4.3). Unstrengthened specimens were corroded to a target mass loss of steel of 15%. Other specimens were corroded to a target mass loss of 5%, then strengthened with FRPs before corroding them further to the 15% target. The strengthening scheme was FRP sheets that were U-wrapped around the tensile face of the beam and on two sides of the cross-section in an intermittent manner. At the end of the target loss of steel, corroded steel bars were removed from the concrete, cleaned and then weighed to determine the actual mass loss of steel. They found a beam that was corroded without strengthening to have a mass loss of steel of 11.3%. After strengthening, the mass loss of steel was reduced to 7.4%. Another beam exhibited a mass loss of steel of 13.2% when not strengthened compared to 8.8% when strengthened. Note that at the time of strengthening, mass loss of steel was found to be 5.9%. To assess the effectiveness of FRPs to reduce mass loss of steel, mass losses of steel from beams after strengthening with FRPs are shown in Figure 2.3. It is clear from the figure that the differences between the measured losses of steel after strengthening and the target loss of steel of 15% are within those found by other researchers on unstrengthened beams. It is therefore difficult to appreciate the effectiveness of FRPs to reduce the rate of steel corrosion. It is however, obvious that there was continued steel corrosion after strengthening. How this continued steel corrosion affects the behaviour of strengthened beams is not clear and needs further research.

In another research to evaluate the effectiveness of FRPs to reduce the rate of steel corrosion, El Maaddawy et al. [60] corroded various RC cylinders. The height of some cylinders was fully-wrapped with FRP sheets whilst others were not wrapped. Steel corrosion was accelerated by mixing concrete with 3% NaCl by weight of cement and by applying a constant voltage of either 15 or 60 V. At the end of the corrosion process, they assessed the mass loss of corroded steel bars. To compare their results with those found by other researchers where RC specimens were not strengthened, mass loss of steel on FRP-strengthened cylinders is shown in Figure 2.3. Similar to results from Badawi and Soudki [34], the figure does not indicate a significant reduction of steel due to FRPs. In fact it indicates that there are instances where FRP wrapped cylinders had larger mass loss of steel than mass loss predicted from Faraday’s Law. It is therefore evident
that significant steel corrosion occurred even after strengthening. As confirmed in their results as well as in Badawi and Soudki [34], there was evidence of widening of corrosion cracks after FRP strengthening due to continued steel corrosion. Therefore, if the service life of RC structures based on the criterion of load-bearing capacity is not significantly affected by further steel corrosion after FRP strengthening, their service life based on the criterion of corrosion cracks width is likely to be.

Another important matter to discuss is that debonding of FRPs really becomes a problem at loads that are much greater than service loads. Techniques to prevent it such as wrapping with FRP sheets are therefore often employed in laboratory specimens with the intent to measure their full flexural capacity. For in-service structures where applied loads are low, there is little need to wrap strengthened members with FRP sheets, and in fact they are often not [55]. Note that despite corroding RC specimens by El Maaddawy et al. [60] and Badawi and Soudki [34] not showing a significant reduction in the rate of steel corrosion after FRP strengthening, they were wrapped with FRP sheets. It is foreseeable that without confinement from FRP wraps, side faces of corroding RC structures will crack more, and probably also corrode more. Further research is therefore needed to evaluate the effectiveness of FRPs without FRP wraps to control further steel corrosion as well as widening of corrosion cracks, especially when strengthening is carried out under load.

2.7 Conclusions

This chapter discussed various measurable parameters of corroding RC structures that are often used to indicate end-of-service life of the structures as well as to predict the non-measurable parameters. The effect of loading on these parameters was also looked at. Finally, the effectiveness of repairs to extend the service life of corroded RC structures was assessed on the basis of changes on both the measurable and non-measurable parameters.

From experimental work on the time to first cracking of the cover concrete due to corrosion of embedded steel, various factors of the resistance of concrete to cover
cracking were revealed. The controlling factor was the concrete cover depth to bar
diameter ratio. It was shown that a slight increase in this factor (especially at
values above 2.5) significantly increases the time to cover cracking. Porous
concrete was also found to be highly resistant to corrosion cracking as corrosion
products must first diffuse into the voids before applying stresses on the concrete.
It however allows an easier ingress of corrosion agents resulting in the embedded
steel being prone to corrosion. Interestingly, the compressive strength of concrete
was found to have a little effect on the resistance of concrete to cover cracking
due to steel corrosion.

The usefulness of the vast experimental work on the time to cover cracking to real
structures was lessened by the substandard laboratory experimental procedures
that were used by various researchers. One major drawback with the work was a
lack of consistency on the programmes to measure time to cover cracking. For
example, some researchers developed devices to measure the time to cover
cracking whilst most recorded it as when they could see the cracks with a naked
eye. There is no doubt that for cracks to be visible, they need to have first
occurred and then sufficiently widened. The times recorded by researchers as
when first cracks occurred therefore included the time taken for them to widen.
Expectedly, the outcome was that results from various researchers were difficult
to compare.

Despite this difficulty, some researchers developed analytical models to predict
the time to cover cracking which they then tried to calibrate with experimental
results discussed above. Their major hurdle was to comprehensively represent the
inconsistently recorded time to first cracking from various experimental works.
Not surprising, they also used different criteria to indicate first cracking. Some
simply related it to a uniform internal pressure applied by corrosion products and
the tensile strength of concrete. Others assumed that cracking starts at the
concrete/steel interface and propagates outwards. Regrettably, none of them
included the time required for the cracks to widen to a visible size. Obviously they
obtained results that were far apart.
The effect of sustained load on the time to cover cracking was investigated by El Maaddawy and Soudki [8,9]. Unfortunately, the inconsistency of results on RC specimens tested in the absence of a sustained load made it difficult to conclude their findings. In fact they also recorded time to first cracking as when the cracks were visible to a naked eye. Clearly to be able to use the criterion of first cracking of concrete due to steel corrosion to predict the service life of corrosion-affected RC structures, more fundamental understanding on the matter is necessary.

Compared to time to first cracking, corrosion crack widths are much easier to measure. More consistent results on this were therefore found. It was shown that corrosion cracks linearly widen with an increase in the level of steel corrosion. The range of maximum corrosion crack widths that corresponded to a mass loss of steel of 1% were from 0.03 to 0.14 mm. At higher corrosion crack width limits such as a crack width of 1 mm from DuraCrete Final Technical Report [39], this range corresponds to a large variation in mass loss of steel (7 to 33%). This was mainly attributed to differences in the various procedures that were used to accelerate steel corrosion, especially the level of impressed current density. It ranged from 3 to 10400 μA/cm².

El Maaddawy and Soudki [33] used relations between corrosion crack widths and mass loss of steel to recommend that accelerated steel corrosion should be limited to impressed current densities below 200 μA/cm². Unfortunately, they also ignored other factors of the rate of steel corrosion such as cyclic introduction of chlorides into the concrete. If these factors significantly affect the rate of steel corrosion and the rate of widening of corrosion cracks, then the current density recommended by the researchers may not fully represent in-service conditions. Fortunately, the level of sustained load (which they did not consider) was found to have a little influence on the rate of widening of corrosion cracks as well as the rate of steel corrosion.

Prior to relating the level of steel corrosion with residual load-bearing capacity of corroded RC structures, various researchers found it fitting to measure the actual mass loss of steel and relate it with theoretical loss. It was shown that Faraday’s Law often over-predicts average mass loss of steel, especially at levels of steel
corrosion above 8%. The trend showed mass loss predicted using Faraday’s Law to be 18% larger than the measured average mass loss. The data however, had a large scatter with the difference between Faraday’s predicted loss and actual loss of steel ranging from -6.7 to 23.9%. Maximum mass loss of steel was also shown by a few researchers to be significantly larger than average mass loss. The relation between maximum mass loss of steel and mass loss from Faraday’s Law was however, not well researched.

Residual load-bearing capacity of corroded RC beams was found to linearly reduce with an increase in the mass loss of steel. Average mass loss of steel of 1% corresponded to 1.4% loss in the load-bearing capacity of RC beams. On the contrary, 1% maximum mass loss of steel corresponded to 0.7% loss in load-bearing capacity. Based on theoretical relations between mass loss of steel and load-bearing capacity, it was recommended for researchers to measure maximum mass loss of steel rather than average mass loss. This emphasises the need to relate maximum mass loss of steel with Faraday’s Law.

Patch repairs were found not to have much influence on the load-bearing capacity of corroded RC specimens. Strengthening with FRPs was however, found to substantially increase their load-bearing capacity. In addition to increasing the load-bearing capacity of RC beams (by over 50%), the rate of reduction in the capacity of strengthened beams with an increase in the level of steel corrosion was much lower. It was shown that 1% average mass loss of steel corresponded to 0.4% loss in the load-bearing capacity of strengthened beams. The major drawback here was that immediately after strengthening, specimens were loaded to failure. It is therefore difficult to appreciate the effect of further steel corrosion (if any) on the effectiveness of FRPs. The information is however, needed in in-service structures which often have a long-term exposure to corrosion agents after strengthening with FRPs.

If properly executed, patch repairs were shown to effectively prevent further steel corrosion. On the contrary, significant steel corrosion was recorded after strengthening with FRPs. Clearly the two practices complement each other; one prevents further steel corrosion (patch repairs) whilst the other increases load-
bearing capacity (FRPs). Surprisingly, strengthening corroded RC specimens with FRPs ignored patch repairs. Research where both practices are used is needed.

For specimens that were corroded in the absence of a sustained load, the majority of researchers found flexural stiffness to decrease with an increase in the level of steel corrosion. Zhang et al. [13] however, found the increase in stiffness with the level of steel corrosion to be limited to levels of mass loss of steel below 40%. Stiffness was mostly measured from deflections at the end of corrosion tests when specimens were subjected to monotonically-increasing static loads.

For specimens that were corroded under load, the corrosion process was found to significantly increase (up to 50%) deflections/reduce stiffness. This was despite some specimens being corroded at sustained loads that were equivalent to 80% of the load-bearing capacity of an uncorroded beam. Furthermore, this influence in deflections was even observed on specimens that were corroded after creep had stabilised. Unfortunately, the effect of steel corrosion on deflections was limited to mass losses of steel below 7%. Despite deflections indicating changes in stiffness, more understanding on the behaviour of RC members requires an in-depth assessment of variation of longitudinal strains along the section of the member. Furthermore, models to predict deflections are based on the variation of longitudinal strains. More research on this is necessary.

Note that for specimens that were corroded in the absence of a sustained load, researchers assessed their behaviour at a chosen level of steel corrosion when they were subjected to monotonically-increasing static loads. However, for specimens that were corroded under sustained loads [5-7], stiffness was assessed when applied loads were kept constant but the level of steel corrosion was increased. The later is more representative of in-service conditions. Even more informative results will be obtained if during corrosion under load, the level of the sustained load is changed as is often the case in in-service structures. This work is discussed in Chapter Five.

The majority of test frames reviewed were such that it was difficult to monitor corrosion crack propagation or even use the frames to repair structures whilst
under load. This probably explains why none of the reviewed test programmes attempted to repair the corroded structures whilst under load. In corroboration, some of the corroded beams by El Maaddawy et al. [8,9] were later strengthened with FRPs but the strengthening process was carried out after removing the specimens from their respective test frames. Considering that large amounts of resources continue to be used worldwide in an attempt to increase the service life of corroded RC structures, it is important that an improved test frame needs to be designed to better assess the effectiveness of these repairs to restore the functionalities of the structures.

2.8 Needed further research

This chapter clearly indicated that a holistic research where the effects of steel corrosion and repairs on performance of corroded RC structures under conditions that are representative of in-service conditions is needed. The primary factor, which has not been dealt with sufficiently in the past, is corroding and repairing RC structures under sustained service loads. As already stated, the absence of a sustained load in previous works (which is present in in-service structures) might have resulted in researchers overvaluing some of the repair techniques.

Another important factor is that repair techniques used in the research should be similar to those used on in-service structures. Some of the repair techniques used in previous laboratory tests over-strengthened specimens and yet they are not often used in in-service structures. They therefore provided information that is of little use to practicing engineers. Rather, the information they provided might be deceiving.

A special test frame, different to those discussed in the chapter, must certainly be designed to allow for the above-needed research. The test frame should also permit continuous measurement of parameters that can be used to indicate the level of steel corrosion.

If the corrosion process is accelerated to achieve needed structural damage in a reasonable time frame, the current density used must be within the limits that are
representative of in-service conditions. Probably most importantly, after repair, rather than immediately assessing the load-bearing capacity of specimens, it is essential that the effectiveness of repairs, under load, to control further steel corrosion be assessed. The behaviour of the RC specimens during this time must also be fully investigated. Following is a summary of specific issues that the research must permit or look into;

1. Contamination of selected faces and regions of RC specimens with corrosion agents. Chlorides should not be added to the concrete mix;
2. Cyclic introduction of chlorides into the concrete as well as drying of the concrete. Specimens should not be fully immersed in salt solution or continuously wetted with water;
3. Accurate measurement of time to cover cracking that can be represented with an analytical model. The research should not rely on first corrosion cracks that are visible to a naked eye;
4. Accurate continuous measurement of the maximum rate of corrosion crack widening on different faces of RC members before, during and after repairs;
5. Accurate continuous measurement of longitudinal strains across various sections of RC members before, during and after repairs. The research should not be limited to measurement of deflections;
6. Accurate measurement of deflections;
7. Measurement of the variation of steel loss along a corroded region. The research should not be limited to average mass loss of steel;

2.9 References


3.1 Introduction

Corrosion of steel embedded in concrete is the principal cause of damage of RC structures that are within the marine environment. Chapter Two provided a review of previous experimental and analytical work carried out on the behaviour of corroding RC specimens. The intent of most studies was to relate measurable parameters of corroding RC structures such as corrosion crack widths, with non-measurable parameters of the structures such as the level of steel corrosion. Regrettably, the majority of previous work has been on RC specimens that were corroded in the absence of a sustained load. A review of the few previous programmes where RC specimens were corroded under load was also given in the chapter.

It was shown in the review that previous researchers failed to accurately and consistently monitor the time to first cracking of the cover concrete due to corrosion of embedded steel [1-3]. Expectedly, analytical models also failed to accurately predict it [4]. It is however, one of the criteria that is often used to indicate end-of-service life of corrosion-affected RC structures. It was therefore recommended in the chapter that an alternative research method is needed to measure as well as predict it accurately.

Chapter Two indicated that there is vast research on corrosion crack widths and their relation with mass loss of steel. Some researchers also monitored corrosion crack patterns [5-8]. However, in the majority of research programmes, corrosion crack widths and crack patterns were measured at the end of the corrosion process [5-10]. In many researches, this was because the experimental programmes did not allow for continuous monitoring of corrosion crack widths. An example of this is when specimens were immersed in salt solution during accelerated
corrosion tests [5-6]. The outcome was that the rate of widening of corrosion cracks was unknown. To predict service life of a structure on the basis of a chosen crack width limit (such as a crack width of 1 mm), this information is essential. Fortunately, sustained loads were found to have a little influence on the rate of widening of corrosion crack widths [5-8].

Similar to crack widths, stiffness of RC specimens that were corroded in the absence of a sustained load was often recorded at the end of the corrosion process. It was deduced from load-deflection curves as the specimens were subjected to monotonically-increasing static loads. Most researchers found stiffness to always reduce with an increase in the level of steel corrosion [9,11]. Zhang et al. [12] however, showed that the reduction in stiffness was limited to mass losses of steel below 40%.

A few researchers, Ballim et al. [5,6] and Yoon et al. [13], continuously measured deflections as specimens were corroded under load. Note that contrary to previous work, deflections here were caused by an increase in the level of steel corrosion at a chosen level of a sustained load as opposed to an increase in the level of applied loads at a chosen level of steel corrosion. Interestingly, they found stiffness to remain constant despite continued increase in the level of steel corrosion at mass losses of steel above 7% compared to 40% found by Zhang et al. [12]. Work by Ballim et al. [5,6] and Yoon et al. [13] is certainly more representative of in-service conditions. Therefore, if engineers are to apply findings from Zhang et al. [12] and many other researchers to corroding RC structures in the field that have mass losses of steel above 7%, they may incorrectly conclude that the structures do not experience further steel corrosion. This emphasises the need to assess performance of corroding RC structures under load. It should be pointed out though that beam deflection is not sufficient to comprehensively represent performance of a RC specimen. Variation of longitudinal strains along sections of specimens is necessary.

At the end of corrosion tests, many researchers tested their specimens to failure. They then measured actual loss of steel and related it with load-bearing capacity. Most relations involved average mass loss of steel [7,8,11,14] whilst a few used
maximum pit depths [15,16]. Theoretical models however, indicated that it was best to relate load-bearing capacity with maximum mass loss of steel [17].

Another matter discussed in Chapter Two was the effectiveness of repairs to increase the service life of corrosion-affected RC structures. Patch repairs were found to control further steel corrosion and FRPs to increase their load-bearing capacity. No research was found where both practices were combined, especially when carried out under load. It was therefore recommended that a holistic research where the effects of steel corrosion and repairs on behaviour of corroded RC structures under conditions that are representative of in-service conditions is needed. The research is introduced in this chapter.

### 3.2 Objectives of the chapter

This chapter outlines a research that was designed to extensively assess the performance of RC beams that were corroded under various levels of sustained service loads. Probably most importantly, it describes a test programme that was specifically designed to allow for patch repairs and strengthening with FRPs to be carried out on corroded beams under load. Following a discussion on the limitations of corrosion procedures that were previously used, a procedure that is more representative to in-service condition is presented. In addition to experimental tests carried out, the chapter discusses a detailed monitoring system that would provide in-depth information on the performance of RC specimens during corrosion as well as after repairs.

### 3.3 Test programme

To achieve the research objectives of the thesis that were discussed in Chapters One and Two, 20 RC beams were tested in this research. Three beams were not corroded to act as controls. The performance of one of the control beams under a sustained load was assessed. Another seventeen beams were initially corroded using an impressed anodic current under various levels of sustained service loads. Following the completion of the accelerated corrosion phase, selected beams were
patch repaired and strengthened with FRPs also under sustained loads. Repaired beams were then allowed to further corrode without impressing anodic current.

The major hurdle with the design of the experimental programme was the level of steel corrosion at which repairs should be carried out. Unfortunately, Chapter Two indicated little information on this, except for corrosion crack width limits specified by DuraCrete Final Technical Report [18]. It was however, shown in the chapter that those limits may prescribe that some specimens be repaired whilst having adequate residual load-bearing capacities and others with unbearable residual load-bearing capacities.

It is worth noting that the review in Chapter Two indicated the lower limit of undesirable corrosion crack widths (0.3 mm) from DuraCrete Final Technical Report [18] to be related to mass losses of steel between 2 and 10%. The upper limit (1 mm) corresponded to mass losses of steel between 7 and 33%. In terms of load-bearing capacity of corroded RC beams, the lower crack width limit corresponded to loss in capacity of 3 to 15%. The upper limit corresponded to loss in load-bearing capacity from 11 to 50%. It was also pointed out in Chapter Two that Mangat and Elgarf [11] showed that at mass losses of steel below 10%, varying the level of impressed current density has little influence on the stiffness of beams. There was however, a recognisable influence of current density on stiffness at mass losses of steel above 10%. Further discussion in the chapter indicated Faraday’s Law to significantly over-predict actual mass loss of steel at levels of steel corrosion above 8%. Perhaps the most interesting finding in Chapter Two was that mass losses of steel above 7% had little effect on the stiffness of beams that were corroded under load.

Note from the above discussion that a critical mass loss of steel that defines the lower limit or at times the upper limit of a range of mass losses of steel for a selected parameter is about 10%. This implies that up to a mass loss of steel of 10%, significant changes in the behaviour of corroding RC beams are likely to be recorded. This critical mass loss of steel was therefore selected in this research to define the time for repair of corroded beams. For the purposes of verification of the variation of mass loss of steel with the load-bearing capacity of beams (to be
discussed further in Chapter Four), the level of corrosion on three beams (beams 8, 11 and 12) before repair was limited to the time of first appearance of a visible corrosion crack. This also represents a conservative approach where the time of first appearance of visible corrosion cracks is used as a criterion of end-of-service life of corrosion-affected RC structures.

During testing, performance of corroded and repaired beams was assessed by continuously monitoring strains on the exterior surfaces of beams. Central deflections were also continuously measured on beams that were tested under load. Eighteen beams in the research were finally tested to failure. The other two are still under long-term corrosion testing. After testing beams to failure, corroded steel bars were then retrieved, cleaned and weighed to determine the mass loss of steel. A summary of the test programme is shown in Table 3.1 and a schematic of the experimental programme is shown in Figure 3.1.

It is evident from Figure 3.1 that in most beams where two-day drying cycles were used, corrosion test lasted for 64 days. This corresponded to 44 days of impressed current. To achieve the 44 days of impressed current in beams where four-day drying cycles were used, the corrosion test lasted for 80 days. From the current density that was used, 189 µA/cm², (to be discussed later), it can be shown that after 44 days, the theoretical mass loss of steel from Faraday’s Law was 10%. It is also clear from the figure that various applied loads were used during the corrosion process. After the target corrosion, some specimens (beams 4,5,8,9,17) were immediately loaded to failure. They were intended to find the effect of steel corrosion on the load-bearing capacity of RC beams. Others were patch repaired (beams 6,7,10,11,13,14,18,19) and amongst those, some were then strengthened with FRPs (beams 11,14). Here, the intention was to evaluate the effectiveness of patch repairs, FRP repairs and combined patch and FRP repairs to increase the load-bearing capacity of corroded RC beams. There are some specimens which after the target level of steel corrosion, were immediately strengthened with FRPs (beams 12,15,16,20) and then allowed to corrode naturally. It should be mentioned that for a few of these specimens (beams 16,20), after the accelerated corrosion process, they were firstly allowed to further corrode naturally and later strengthened with FRPs. Also note that after repairs, the level of sustained loads
<table>
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<th>Beam no.</th>
<th>Target corrosion before repair (%)</th>
<th>Sustained load as % of ultimate capacity during accelerated corrosion phase</th>
<th>Repair</th>
<th>$f'_c$, MPa (s.d.)</th>
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<td></td>
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<td>10%</td>
<td>FRP Plate and FRP wraps</td>
<td></td>
<td>40.1 (1.3)</td>
</tr>
<tr>
<td>16</td>
<td></td>
<td>FRP Plate</td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td></td>
<td>None</td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td></td>
<td>12%</td>
<td>Patch repair</td>
<td>40.0 (1.1)</td>
</tr>
<tr>
<td>19</td>
<td></td>
<td>15%</td>
<td>FRP Plate</td>
<td>40.2 (1.2)</td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* four days drying cycles

$f'_c = \text{compressive strength of concrete (MPa)}$
Designation: UC = uncorroded; AC* = accelerated corrosion with 4-day drying cycles; AC = accelerated corrosion with 2-day drying cycles; NC = natural steel corrosion; PR = patch repair; FP = repair with FRP plate; FW = wrap beam and FRP plate with FRP sheets; and a number after the two letters indicates the level of sustained load as a percentage of the load-bearing capacity of a control beam.

Figure 3.1   Schematic of the experimental programme
on beams that were repaired under loads above 8% (on test frames) was increased. This is common in in-service structures where loads permitted on structures are increased after repairs. Specific reasons for each step taken in the experimental programme of the research are discussed in later sections of the chapter.

3.4 Specimen configuration

Quasi-full-scale RC beams were tested in this research each with a width of 153 mm, a depth of 254 mm and a length of 3000 mm. The reinforcement details of the beams are shown in Figure 3.2. Each beam was reinforced with 3 x 12 mm deformed bars in tension with a cover of 40 mm, and 2 x 8 mm plain bars in compression also with a cover of 40 mm. Stirrups with a diameter of 8 mm were used as shear reinforcement and were spaced at 100 mm centre-to-centre within the shear span. No stirrups were placed in the middle span; instead compression reinforcement bars in the middle span were tied together by 8 mm diameter hooks at 200 mm spacing. These hooks were meant to prevent failure due to buckling of compression steel bars during the test of beams to failure. The overall configuration of specimens and the strength of materials used was such that under flexure, an uncorroded beam failed by yielding of steel in tension followed by crushing of concrete in compression.

3.5 Material properties

All beams were cast using concrete with the same mix proportions and two beams were cast per mix. Maximum aggregate size of the concrete was 13.2 mm and w/c ratio was 0.63. Cement, fine sand and coarse aggregate contents were 300 kg, 909 kg and 950 kg/m$^3$ respectively. Due to the limited number of loading frames to test beams under a sustained load, it was not possible to test beams at the same age. The compressive strength of each concrete (recorded in Table 3.1) was therefore determined at the time of testing using three 100 mm cubes that were cured in fresh water at a constant temperature of 24 °C. From Table 3.1, the measured compressive strength ranged from a minimum of 35.0 MPa (s.d. =1.2) to a maximum of 46.6 MPa (s.d. =1.1). To help with the selection of repair mortar, modulus of elasticity of concrete in selected beams was tested at the time
of patch repair using 100 mm-diameter cylinders of 200 mm height. Similar to the cubes, the cylinders were cured in fresh water. The modulus of elasticity of concrete was found to be 27.8 GPa (s.d. = 7).

Figure 3.2 Reinforcement configuration

NB! All dimensions are in mm

Figure 3.2 Reinforcement configuration
A locally available commercial repair mortar that is often used in the South African construction industry was selected for this research. It is a free-flowing cement based mortar with maximum aggregate size of 9 mm. It is also rapid hardening; the manufactures contend that it can reach a compressive strength of 35 MPa within 24 hours. The 28-days compressive strength and tensile strength of the mortar were tested using 100 mm cubes that were cured in fresh water at a temperature of 24 °C. Similar to the test of the substrate concrete, the 28-days modulus of elasticity of the repair mortar was tested in compression using 100 mm-diameter cylinders with a height of 200 mm. The tensile strength was determined using the Brazilian tensile test. The respective 28-days compressive strength, tensile strength and modulus of elasticity was 66.8 MPa (s.d. = 2.3), 3.8 MPa (s.d. = 0.2) and 30.8 GPa (s.d. = 4.0). Properties of FRPs and epoxy resins (as reported by the manufacturer) used in the research are also shown in Table 3.2.

Table 3.2 Properties of repair materials

<table>
<thead>
<tr>
<th>Property</th>
<th>Patch repairs (at 28 days)</th>
<th>FRP Plates*</th>
<th>FRP wraps*</th>
<th>Epoxy Resin for FRP Plates (at 7 days)</th>
<th>Epoxy resin for FRP wraps (at 7 days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength MPa, (s.d.)</td>
<td>66.8 (2.3)</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Tensile strength MPa, (s.d.)</td>
<td>3.8 (0.2)</td>
<td>2500</td>
<td>3800</td>
<td>12</td>
<td>30</td>
</tr>
<tr>
<td>Modulus of elasticity GPa, (s.d.)</td>
<td>30.8 (4.0)</td>
<td>200</td>
<td>240</td>
<td>0.58</td>
<td>3.8</td>
</tr>
<tr>
<td>Thickness (mm)</td>
<td>N/A</td>
<td>1.4</td>
<td>0.117</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

*Property of FRPs and epoxy resins are as given by the manufacturer

N/A = not available

Tensile pull tests were carried out on the reinforcing steel bars used in this research. The 12 mm deformed bars used for tensile reinforcement had yield strength of 549 MPa (s.d. = 3 MPa) and ultimate strength of 698 MPa (s.d. = 4 MPa). The 8 mm plain bars used for compression reinforcement, shear
reinforcement and hooks had yield strength of 385 MPa (s.d. = 1 MPa) and ultimate strength of 451 MPa (s.d. = 2 MPa).

3.6 Sustained service loads

The levels of sustained service loads used in the research were equivalent to 0, 1, 8, 12 and 16% of the ultimate load-bearing capacity of a control beam. These levels of sustained loads were much lower than the levels of loads used by other researchers [5-8,12,13], discussed in Chapter Two. Their loads ranged from 20 to 80% of the ultimate capacity of equivalent uncorroded beams. The load levels in this research were purposely selected to test beams under the flexural cracked and the uncracked stage. This is because it is well known that structural behaviour of RC beams can be divided into the two distinct cracking stages but it is not known how steel corrosion affects the performance of structures under the different stages. In the previous work [5-8,12,13], specimens were only tested in the flexural cracked stage. For beams tested under the flexural cracked stage, low loads were also chosen because aged structures which are often more susceptible to corrosion due to their extended time of exposure to aggressive environments in addition to poor concrete that was used in the past, were mostly overdesigned such that they continue to experience low applied stresses.

It will be shown in Chapter Four that the average ultimate load-bearing capacity of the three control RC beams was 39 kN-m. The minimum applied moment that induces flexural cracks on a beam (cracking moment), $M_{cr}$, calculated from equation 3.1, can be shown to be 3.64 kN-m.

$$M_{cr} = \frac{I_g f_{ct}}{y_g}$$  \hspace{1cm} (3.1)

Where; $M_{cr}$ = cracking moment of a beam (N-mm); $f_{ct}$ = tensile strength of concrete (2 MPa); $I_g$ = gross second moment of area of a beam ($2.28 \times 10^8 \text{ mm}^4$); and $y_g$ = gross depth of the neutral axis of a beam measured from its tensile face (124.23 mm).
The cracking moment is therefore 9.3% of the ultimate capacity of a control beam. As will be discussed later, the test frame was a mechanism and relied on the applied loads and the test specimens for stability. For safety, it was important to have adequate contact between the loading system of the test frame and the test specimen. A load close to the flexural cracking load of beams (8%) was therefore selected.

A load of 1% was induced on some test specimens because they had to be raised above the concrete floor to allow for a collection pond of corrosion products to be placed underneath the beams. This set-up prevented corrosion products from spilling onto the concrete floor which would stain the floor and possibly damage testing machines near the test area. This load is however, negligible to cause any significant structural damage. Despite that, it is important to specify it since one beam was corroded under a load of 0%. Beams tested under the 1% load were supported on two concrete blocks of 100 x 100 x 200 mm placed at the third points of the beam span. As shown in Figure 3.3, a collection pond for corrosion products was then placed underneath the beams and between the supports. The beams were tested with the tensile face up.

Sustained loads of 8% and above were achieved by testing RC beams using a loading frame shown in Figures 3.4 to 3.7. The frame was designed and built at the University of Cape Town. Steel columns to support the test beams and the loading beam were bolted to a strong floor to provide reaction. Weights were hung at one end of the loading beam and transferred to the load distribution beam using a frictionless bearing support and pinned struts. The loading beam had effective lever ratios of 1:6 between the hung weights and the pinned struts to magnify the load from the weights that was transferred to the load distribution beam. Bearing supports and pinned struts were used to allow for free vertical movement of the loading system (shown in Figure 3.5). Similar to test frames used by Yoon et al. [13] as well as El Maaddawy et al. [7,8], the loading system of the test frame was purposely designed to be a mechanism (when under load) that depended on the test beams for its stability and hence followed them as they deflected. This arrangement was chosen to ensure that the test specimens were always under a constant sustained load.
The load from the hung weights was transferred to the test specimen by the load distribution beam, to produce four-points bending with a constant moment in the middle-third of the beam, using rollers. Ball joints were used at the end supports of the test specimens to allow for free rotations of the specimens (Figure 3.6). The overall configuration of the loading system was such that beams were tested over a span of 2600 mm and had a constant moment region of 1000 mm at the mid-span. A sketch of the side view of the frame showing these dimensions is shown in Figure 3.7.

Figure 3.3 Test set-up for beams under a load of 1% (inverted beams)
Figure 3.4 Loading rig
Figure 3.5 Loading system (end view)

Figure 3.6 End support of test beams
This frame was designed with the intent to resolve some of the major drawbacks from previous test frames discussed in Chapter Two. To achieve this, it was designed to apply tensile stresses on the top face of beams so that they could be tested whilst inverted. This allowed for full access to the tensile face of the beams. It therefore enabled; a simple and yet extensive monitoring of strains and corrosion crack widths on the tensile face of beams during the corrosion process; an easy construction of a NaCl pond for the corrosion process on the tensile face of the beams; and probably most importantly, patch repairs and FRP strengthening to easily be carried out on the corroded beams whilst under load.

Another important consideration in the design of the frame was to have the loading system directly underneath the test beams and with the same longitudinal axis as the beams so as to minimise the laboratory space occupied by the frame. The disadvantage of the design is that test specimens were about 2 m above the ground (Figure 3.7). A platform was therefore required to access the corroding
region of test beams. In addition, the position of the beams made it difficult and unsafe to break open the beams for patch repairs.

The frame was kept free from corrosion attack by protecting it from contact with salt solution from ponds. A collection pond was placed between the load distribution beam and the test specimens to collect any leakage of salt solution from the top pond and corrosion products discharged on the sides of the beams through corrosion cracks. The pond was also intended to prevent corrosion products from contaminating the strong floor.

3.7 Steel corrosion

Accelerated steel corrosion was limited to the tensile steel bars over a length of 700 mm that corresponded to the middle section of the beams as shown in Figures 3.2 and 3.7. The performance of RC beams due to partial steel corrosion has also been assessed by other researchers [7,8,10,19]. Partial steel corrosion was used as opposed to corrosion of the entire length of the steel bars because in in-service structures (especially when due to chloride attack), steel corrosion is mostly expected on the tensile steel bars and within the flexural cracked region where cracks provide channels for ingress of corrosion agents.

Corrosion agents in this research were allowed to reach the steel from one face (the tensile face) of the beams. This is contrary to many other previous research programmes discussed in Chapter Two where the entire surface of the bar within the desired corrosion region was contaminated with chlorides. This involved either complete immersion of specimens in NaCl tanks [5,6,14] or casting concrete mixed with chlorides [7-9]. As discussed in Chapter Two, the set-up in this research is more representative of in-service conditions where in addition to removing chlorides from concrete mixes, one exterior face of the structure is often exposed to chloride attack.

Partial contamination of one face of the beams (tensile face) with chlorides was achieved by building a NaCl pond on the desired section and face of the beam using PVC sheets as shown in Figures 3.2, 3.3, 3.7 and 3.8. The pond was built
such that it did not offer any restraint to the deformation of the concrete. Silicon, which is very flexible when hardened, was used to attach PVC sheets to the concrete. The ponds were then partially filled with 5% by weight, NaCl solution. A 12 mm stainless steel bar of length 250 mm was then placed in the NaCl solution to act as a cathode. The tensile steel bars inside the concrete and the stainless steel bar were connected to a power supply to induce a constant current of 150 mA to be equally shared between the steel bars. Assuming corrosion was limited to the tensile steel bars and to the desired length of 700 mm (which will be confirmed in Chapter Four), this current corresponded to a current density of 189 $\mu$A/cm$^2$. This current density is much larger than current densities in real structures which normally range between 0.1 and 100 $\mu$A/cm$^2$ [2,19,20]. The intention of the accelerated corrosion test was to produce desired structural damage within a reasonable time frame but without excessively altering the structural response that would be obtained under natural steel corrosion. In the absence of a testing standard for accelerated corrosion of RC laboratory specimens, a guide from El Maaddawy and Soudki [21], discussed in Chapter Two, that impressed current densities should be limited to 200 $\mu$A/cm$^2$ was followed.

For the majority of beams, the accelerated corrosion process consisted of a ponding cycle of four days wetting and two days drying under natural air to promote corrosion. For beams 4-7 however, drying cycles also lasted four days. As discussed in Chapter Two, longer drying days are expected to produce corrosion products that are more stable and with less volume expansion. The rate of widening of corrosion cracks is therefore expected to differ between drying cycles. Anodic current was only applied during the wetting period.

As already mentioned, following the completion of the accelerated corrosion phase, selected beams were repaired using mortars and FRPs. They were then let to corrode further under natural laboratory conditions and under various levels of sustained service loads. Contrary to experimental steps in other repaired beams, following the end of the accelerated corrosion phase; beams 16 and 20 were firstly allowed to corrode naturally under different levels of sustained loads before strengthening with FRPs. During this natural corrosion phase, the load on beam
16, which was initially at 8%, was increased to 12% whilst the load on beam 20, which was initially at 12%, was reduced to 8%. The changes in the load levels were intended to simulate real situations where in the worst case, no action is taken after steel corrosion and yet there is a corresponding increase in the level of the load that is imposed on the structure (beam 16), or the load on a structure is reduced after noticing corrosion damage (beam 20). They were also meant to assess the effects of loading on the performance of RC specimens under natural corrosion attack. At the end of the first natural corrosion phase, beams 16 and 20 were strengthened with FRPs and then allowed to corrode further, still under natural corrosion, but at a load of 12% (Figure 3.1). After 73 days from the time of first load increment following repairs (209 days for beam 16 and 157 days for beam 20), the load on beams was further increased to a load equivalent to 16% of the ultimate capacity of a control beam. At the time of writing (after 450 days for beam 16), these beams are still under natural corrosion testing.

3.8 Strain measurements

Transverse and vertical deformations of corroding RC beams were assessed by monitoring transverse and vertical strains occurring on the concrete at various potential cracking regions. This is in contrast with the majority of previous studies where corrosion damage on concrete due to expansive corrosion products was presented as corrosion crack widths that were directly measured using various devices such as magnification lenses, microscopes and crack compactors [2,5-10]. The main advantage of measuring strains as opposed to corrosion crack widths is that strains are able to reveal deformation on beams even before cover cracking. They can therefore indicate accurately, time to first cover cracking. Despite not being fully exploited, the advantages of using strains over crack widths to assess the behaviour of corroding RC specimens have also been reported elsewhere. For example, Andrade and Alonso [22] found strain gauges to be informative at the early corrosion stages when cracks were not visible. Badawi and Soudki [23] and Soudki and Sherwood [24] found strain gauges to be useful after strengthening specimens by wrapping them with FRP sheets when the cracks were covered.
In this research, transverse and vertical strains were measured using a 100 mm-gauge length demountable mechanical (demec) strain gauge with a range of ±10 to ±5000 micro strains. A minimum tensile strain of 10 micro strains recorded from the gauge is therefore equivalent to expansion of concrete or a crack widening of 0.001 mm. From corrosion crack patterns discussed in Chapter Two, corrosion cracks on the tensile face of beams in this research were expected to occur between the corroding bars and to propagate in the longitudinal direction of the bars. Transverse strains at a section on the tensile face of a beam were therefore measured by gluing two sets of stainless steel targets for the demec gauge on the concrete surface, perpendicular to the longitudinal axis of the beam such that they overlapped all the three longitudinal tensile steel bars. The reinforcement configuration shown in Figure 3.2, was designed such that this was possible. Seven sections at spacing of 100 mm were monitored for transverse strains on the tensile face of a corroding beam as shown in Figure 3.9a. This was meant to obtain the maximum rate of widening of corrosion cracks. As discussed in Chapter Two, this information is necessary to predict accurately, the service life of corrosion-affected RC structures on the basis of the criterion of corrosion crack widths. The strains were only monitored within the corrosion region.

For the first set of beams tested in the research, the design of the ponds (shown in Figure 3.3) was such that vertical strains near corroding steel bars could not be measured. For the subsequent set of beams, ponds were re-designed (shown in Figure 3.8) to allow vertical strains on side faces of beams to also be measured on different potential cracking faces of the beams. The change of the set-up was primarily carried out to assess the nature of strains on uncracked side faces of beams. As will be discussed in detail in Chapter Six, this change was equally important in assessing the effectiveness of FRP strengthening schemes used in the research to control the rate of corrosion crack widening.

Following the improved ponding system, to monitor vertical strains at a section on the side face of a beam, steel targets were placed across the exterior bars and perpendicular to the longitudinal axis of the beam as shown in Figure 3.9b. Seven sections were selected such that they were directly opposite the monitoring sections on the tensile face. Despite the experimental changes made on points of
measurements of strains, mechanical frames used for testing beams were such that only two faces of each beam (tensile face and one side face) were accessible. To counter for the missing data on the other side of the beams, a number of beams were tested under the same conditions in order to characterize all the various modes of deformations (Figure 3.1).

Figure 3.8 Measuring strains on side faces of beams

To measure strains in the longitudinal direction of beams, stainless steel targets for the demec gauge were glued to the concrete surface at a spacing of 100 mm at four different levels on the concrete beam across the corrosion region as shown in Figure 3.9b. At each level, seven targets were placed at a spacing of 100 mm in the longitudinal direction of the beam. The targets between levels were such that they were directly above or beneath each other. Similar to the vertical strains, it was not possible to measure longitudinal strains on the side faces of beams at levels 2 and 3 for the first set of beams. The change in the set-up was
Designation: lt = longitudinal strains on the tensile face; tt = transverse strains on the tensile face; ts = vertical strains on the side face; l2 = longitudinal strains on the side face measured 30 mm from the tensile face; l3 = longitudinal strains on the side face measured 100 mm from the compression face; lc = longitudinal strains on the side face measured 30 mm from the compression face.

Figure 3.9 Location of targets for strain measurements
carried out to assess the compatibility of strains during the corrosion process and after interventions (to be discussed in Chapter Five). This information is necessary for models on flexural behaviour of corroding RC structures. Previous models assumed compatibility of strains along sections of beams [25,26]. Cracking of beams is however, expected to disturb this compatibility. Unfortunately, deflection which previous researchers relied on, cannot provide this information.

In general, this form of measuring strains together with the mechanical test frame used to load the beams allowed for an easy replacement of targets when lost and to easily reset the targets when applied strains exceeded the measuring capacity of the demec gauges.

### 3.9 Patch repairs

After the accelerated corrosion phase, beams to be patch repaired (Figure 3.1) were removed from their respective loading frames and placed on a flat floor for removal of the corrosion damaged concrete. Placing the beams on the flat floor was done because as a safety measure, it was decided not to remove the damaged concrete with the beams on the test frames. This is one limitation from previous loading frames that could not be resolved by the frame used in this research.

Concrete surrounding corroded steel bars was removed to a depth of about 40 mm below the corroded steel bars and to a length of about 150 mm beyond each end of the corrosion region (where there was no visual sign of corrosion of steel bars) as shown in Figure 3.10. Exposed corroded steel bars were then cleaned with a motorised wire brush to remove corrosion products and loose concrete particles. High pressurised air, water and sand were used for surface preparation of the substrate concrete before repair. Beams that were initially corroded under the loading frames were then mounted on their respective test frames with exposed reinforcements as shown in Figure 3.11. They were all subjected to a load equivalent to 8% of the ultimate load capacity of a control beam. Patch repair to replace the removed concrete was then carried out on the loaded beams using locally available cement based repair mortar with properties shown in Table 3.2.
Beams that were initially corroded under a sustained load of 1% were repaired still under the load of 1%.

Prior to application of the repair mortar, substrate concrete was saturated with plain water for six hours. An epoxy bonding agent was used to improve the bond between the repair mortar, the substrate concrete and the reinforcing steel bars. It was sufficiently coated on steel bars to form an electrochemical/physical barrier for chlorides to reach the steel. It therefore also acted as a protection for further steel corrosion. The repair mortar was then cured for 28 days under moist conditions. Targets for the demec gauge on the face of beams that were removed with the damaged concrete were replaced after the patch mortar had set. They were intended to measure the structural response of repaired beams due to drying shrinkage of mortars during the curing stage. After the repair mortar had fully cured, the sustained load on beams that were tested under the load rigs and were patch repaired but not strengthened (beams 13,18,19), was increased to a load equivalent to 12% of the ultimate capacity of a control beam, as indicated in Figure 3.1.
3.10 FRP strengthening

The tensile faces of beams to be strengthened with FRPs were pre-treated prior to strengthening. Pre-treatment involved grinding the tensile face of a beam to uncover aggregates and to level the concrete surface. It also involved using high pressurized air to remove loose dust and aggregates. A carbon fibre reinforced polymer (CFRP) plate with a width of 80 mm, a thickness of 1.4 mm and a length of 2400 mm was bonded to the tensile face of each strengthened beam using an epoxy adhesive. A beam under load after strengthening with a FRP plate is shown in Figure 3.12. For beams that were tested under the load rigs, the sustained load was kept at 8% during the strengthening and curing period (14 days) after which, it was increased to 12% of the ultimate capacity of a control beam (Figure 3.1).
Since steel targets for the demec gauge to measure corrosion crack widths on the tensile faces of beams were removed prior to the strengthening process, the targets were replaced in beams 16 and 20 to assess the rate of corrosion crack widening after strengthening. It was unfortunately not possible to measure transverse strains on the tensile face of beam 15 after strengthening. Note that at service loads, debonding of FRP plates is not an issue. There was therefore no need to wrap the FRP plates with FRP wraps. Similarly, strengthening in-service structures in flexure using FRP plates often does not include FRP wraps [27].

Finally, all beams except beams 16 and 20, were removed from their respective loading frames and tested to failure on a different set-up. Prior to testing FRP strengthened beams (beams 11, 12, 14 and 15) to failure, they were wrapped with 300 mm wide FRP sheets at the ends of the FRP plate and also with 150 mm wide FRP sheets in the middle in an intermittent manner. The spacing between the...
sheets was 300 mm centre-to-centre as shown in Figure 3.13. The wraps were provided to act as anchorages for the longitudinal CFRP plate and to preclude any premature shear failure caused by an increase in the beam flexural capacity after strengthening.

Figure 3.13  Strengthening scheme using FRP wraps

3.11  Ultimate load test

As previously mentioned, specimens were eventually tested to failure in four-points bending in a universal testing machine as shown in Figure 3.14. As shown in Figure 3.1, the ultimate test of the repaired and strengthened beams was carried out at least 60 days after the end of the accelerated corrosion phase to allow for a full test of the serviceability state of the patch repaired and strengthened beams. It will be discussed in detail in Chapters Four to Six. The set-up for the ultimate test was such that the test beam had a span of 2754 mm, a shear span of 877 mm and a constant moment region of 1000 mm at the mid span. Failure load was read directly from the analogue on the loading machine, and was assumed to occur when the load applied on the beam began to drop, with increasing mid-span deflections. The loading rate used was about 1 kN/minute.
3.12 Mass loss of steel

Following the ultimate load test, beams were broken open to remove corroded steel bars and measure their mass loss due to corrosion. Steel bars in the concrete were cut at least 150 mm beyond each end of the corrosion region where there was no visual sign of corrosion. Each of the three bars in a beam was marked at the ends with a permanent ink marker to indicate the direction of current flow and its position in the beam during the corrosion process. The bars were firstly mechanically cleaned using a motorised wire brush. For the first set of beams, steel bars were then cut to coupons of about 100 mm in length. After noticing that there was a significant range in the mass loss of bars along the corrosion region, steel bars in subsequent beams were cut to coupons of about 50 mm in length.
Steel coupons were then dipped in a solution of hydrochloric acid, hexamethylenetetramine and reagent water in accordance with ASTM G1-03 [28]. Figure 3.15 shows cleaned coupons ready for weighing. To correct for any possible loss in the base metal due to the cleaning process, replicate uncorroded control bars from beam 1 were cleaned using the same procedure as for corroded steel bars.

The coupons were then weighed and their actual length was measured to determine their mass per length. The percentage mass loss of a steel coupon, $Q_{gi}$, was calculated from equation 3.2.

$$Q_{gi} = \frac{m_u - m_i}{m_u} \cdot 100$$  \hspace{1cm} (3.2)

Where; $m_u$ is the average mass per length of an uncorroded steel coupon (=0.888 g/mm of length for the 12 mm bars used in the research); and $m_i$ is the mass per length of a corroded coupon.

This procedure of measuring mass loss of steel is different from procedures specified by the ASTM G1-03 [28] and ASTM G46-94 standard [29]. They are therefore different from those used by other researchers who measured the average mass loss of the entire corroded steel bar [5-8,14,23] and pit depths [2,15,16,19]. The disadvantages of other methods are that: the average mass loss does not indicate the variation of mass loss along the bar; Chapter Two indicated that average mass loss of steel does not relate well with load-bearing capacity of RC beams; the actual diameter of an uncorroded deformed bar often varies along the bar due to varying rib heights which makes it difficult to later relate the reduced bar diameters due to corrosion with the loss in area of steel bars; and loss in section of the steel due to corrosion is not uniform as will be shown in Chapter Four. These disadvantages are avoided in the present procedure.
3.13 Conclusions

This chapter described a holistic experimental research programme where the performance of corroded and repaired RC beams was assessed under conditions that are close to in-service conditions. It gave details of a test frame that was purposely designed to allow for corrosion and repairs to be carried out under load. The loads applied were selected such that RC beams could be tested under flexural uncracked and flexural cracked stages. This was intended to allow for more understanding of the performance of RC beams that are corroded under the unique stages. These conditions exist in many aged structures. However, previous researchers only focused on the flexural cracked stage [5-8,12,13].

Figure 3.15 Measurement of mass loss of steel

a) Cleaning of steel coupons  
b) Arranged steel coupons before weighing
One of the weaknesses of previous research that was raised in Chapter Two was the corrosion procedures used by various researchers. It was shown that corrosion products should not be added to the concrete mix and during accelerated corrosion, specimens should not be immersed in salt solution. The reason was that these procedures do not represent in-service conditions. In addition, if corrosion is accelerated to produce needed structural damage in a reasonable time frame, the current density used must be such that damage is proportional to damage caused by natural steel corrosion. Despite El Maaddawy and Soudki [21] showing that current densities must be below 200 µA/cm², densities up to 3000 µA/cm² have been recorded recently [14].

These problems were addressed in this research by building a NaCl pond on a selected face (tensile face) of a beam. The set-up allowed cyclic contamination of concrete with NaCl salt solution as well as drying of the concrete. This process is common in in-service structures but was rarely used in previous research. To further simulate in-service conditions, after the target level of steel corrosion was reached, some specimens were allowed to corrode naturally. The level of sustained load during all stages of steel corrosion was varied.

Time to first cover cracking was shown to be difficult-to-measure in previous research. The researchers had to wait for cracks on the cover concrete to occur and widen to visible cracks before they could make observations of corrosion damage. The inadequacy of the programmes made it difficult for other researchers to develop analytical models to predict the time to cover cracking. In this research, this matter was resolved by measuring transverse and vertical strains on potential cracking sections of beams using a demec gauge. The set-up for the corrosion process (not immersing specimens in NaCl tanks) also allowed for an easy access of targets for the demec gauges during the corrosion process.

The monitoring system of strains was not only limited to indicating the time to cover cracking; it also accurately monitored the rate of widening of corrosion cracks. It even monitored deformations on other faces of beams that remained uncracked. To provide more understanding on the cracking behaviour of corroded beams, cracks were monitored on various sections of beams. The intention was
also to obtain the maximum rate of widening of corrosion cracks. Continuously monitoring the width of corrosion cracks on different faces of beams brings out the influence of corrosion crack patterns on the rate of widening of corrosion cracks. This matter was introduced in Chapter Two when discussing results from Zhang et al. [12]. However, it could not be mined further due to limited information.

To supplement limited information that previous researchers obtained from beam deflections due to steel corrosion, longitudinal strains along various depths of beams were assessed in this research. Deflections of RC specimens during the corrosion process were also measured. In addition to providing stiffness, longitudinal strains were measured at various depths of corroding RC beams to assess compatibility of strains. Note that this compatibility is essential for models of flexural response of RC beams such as one developed by El Maaddawy et al. [25].

Chapter Two indicated that most information on performance of corroded RC structures after repairs was limited to repairs that were carried out in the absence of a sustained load. However, in-service structures normally corrode and are repaired under load. Information from previous researchers on performance of repairs might therefore not be applicable to in-service structures. In fact, it might deceive asset managers and engineers who have great interest on the service life of the structures. This chapter outlined an experimental programme which will provide this information. In this research, patch repairs and FRPs were carried out under load. Furthermore, combined patch repairs and strengthening with FRPs was introduced. To provide for more understanding of the performance of repair systems, repaired beams were allowed to further corrode naturally and at varying levels of sustained loads. The monitoring system of strains discussed earlier was utilised well during this assessment.

Finally, an improved procedure to measure mass loss of steel was provided. Contrary to previous research where average mass loss of steel was used (despite not relating well with theoretical load-bearing capacity), this procedure gave the variation of mass loss of steel of bars along the beam. The information it provides
is also needed to relate maximum crack widths with maximum mass loss of steel, a matter that was discussed in Chapter Two but could not be exploited further due to limited information.

### 3.14 References


CHAPTER FOUR

LEVEL OF STEEL CORROSION AND LOAD-BEARING CAPACITY OF CORRODED AND REPAIRED RC BEAMS

4.1 Introduction

Substantial resources continue to be used worldwide on repairs of corroding RC structures with the intent to meet or extend their design service lives. It was discussed in Chapter Two that the effectiveness of repairs to add service life to a corroded RC structure depends primarily on the residual load-bearing capacity of the structure as well as the material properties of repairs relative to material properties of the substrate concrete. Therefore, when designing for repairs, engineers must know the residual load-bearing capacities of corroded RC structures. When repairs should be carried out is decided from various criteria. The most widely used are the first visible corrosion cracks and corrosion crack widths. It is therefore critical that these criteria be associated with residual load-bearing capacities of structures.

It was pointed out in Chapters One and Two that for already-corroded RC structures, the time to appearance of first corrosion cracks is of little value to engineers for an obvious reason that the structures are already cracked. Moreover, it occurs at mass losses of steel that are too low to significantly impair the load-bearing capacity of structures. Corrosion crack widths have therefore been extensively used to indicate the capacity of corroded structures as well as when repairs should be carried out. For example, DuraCrete Final Technical Report [1] has specified that repairs should be carried out at crack widths between 0.3 and 1 mm. To associate these critical crack widths with the corresponding residual load-bearing capacities of structures, Chapter Two indicated that they must firstly be related with the level of steel corrosion. This is because it is a common parameter between them.
Unfortunately and similar to the load-bearing capacities of structures, the level of steel corrosion is not a measurable parameter of in-service structures. For laboratory specimens, previous researchers found relations between the level of steel corrosion and corrosion crack widths by firstly monitoring crack widths during the corrosion process, followed by removing corroded steel bars from the specimens to measure the actual loss of steel bars using either their mass loss or corrosion pit depths. They found corrosion crack widths to be linearly related with mass loss of steel. A mass loss of steel of 1% corresponded to corrosion crack widths from 0.03 to 0.14 mm. In most researches, mass loss of steel was measured as average mass loss of steel along the corroded bar. A 1% average mass loss of steel was found to correspond to 1.6% loss in load-bearing capacity of a RC specimen. In terms of corrosion crack width limits from DuraCrete Final Technical Report [1], this implies that repairs will be carried out when losses in load-bearing capacities of structures are between 2 and 50%. Certainly this range is too large and therefore speaks to the need to validate these relations with relations found from real structures. Regrettably, it is rare for real structures to be loaded to failure or for steel bars to be removed from the structures. It is therefore important that improved relations be found from laboratory tests that are conducted at near-in-service conditions.

Chapter Two outlined various procedures that were used by previous researchers to obtain needed damage in RC structures due to steel corrosion in a short time. Most of them were clearly far from in-service conditions. Examples are when chlorides were added to concrete mixes and when specimens were immersed in salt solution with various concentrations during accelerated tests. The most critical factor was the level of impressed current density. Densities up to 10400 µA/cm² were reported [2] despite El Maaddawy and Soudki [3] later recommending that they should be below 200 µA/cm². Even then, El Maaddawy and Soudki [3] also used other procedures that do not represent in-service conditions. Chapter Three described an experimental set-up in this research where these shortcomings were avoided.

Another important matter that was discussed in Chapter Two was the ability of repairs to increase the load-bearing capacity of corroded RC structures as well as
control further steel corrosion. Patch repairs were shown to be effective at preventing further steel corrosion but did not increase the load-bearing capacity of corroded structures. FRPs increased their load-bearing capacities but failed to control further steel corrosion. This was despite over-strengthening specimens by wrapping them with FRP sheets, a procedure that is rarely used in in-service structures. Note that most of these tests were under accelerated steel corrosion where steel bars were forced to corrode. In addition, the specimens were corroded and repaired in the absence of a sustained load. Chapter Three outlined a set-up in this research where the ability of repairs to control further steel corrosion and under load was assessed.

4.2 Objectives of the chapter

This chapter discusses improved relations between the level of steel corrosion and load-bearing capacity of RC beams from procedures of steel corrosion that are close to in-service conditions. It also looks into the ability of patch repairs and FRPs under load, to prevent further steel corrosion as well as increase the load-bearing capacity of corroded RC beams. Note that relations discussed here are for non-measurable parameters of corroding RC structures. They will be associated with measurable parameters that are useful to asset managers and engineers in subsequent chapters.

4.3 Visual assessment of corroded steel bars

Figure 4.1a shows a picture of the surface of corroded steel bars that faced the direction of ingress of corrosion agents (top surface) and Figures 4.1b shows the other side of the same bars (bottom surface). The bottom surface here was the furthest surface of bars from the tensile face of beams. Corrosion pits were evident on the top surface of bars whilst near-uniform steel corrosion was observed on the bottom surface. It is also interesting to note that there were no ribs left on the top surface of steel bars. Despite being much smaller, ribs were still visible on the bottom surface of bars. Furthermore, Figure 4.1b shows that there was little corrosion on the core section on the bottom surface of bars.
a) Top surface of steel coupons
b) Bottom surface of steel coupons

Figure 4.1  Visual assessment of corroded steel bars
Loss of steel being mostly experienced on the top surface of bars can be attributed to the direction of ingress of corrosion agents into the concrete. If they were to be uniformly distributed around the steel bars by mixing concrete with chlorides (as used by previous researchers), probably uniform steel corrosion would be observed. The set-up in this research is however, more representative of in-service conditions. It therefore questions the validity of results from previous researches.

Logically, if steel corrosion was limited to the top surface of bars, then the pressure applied on the cover concrete by the expansive corrosion products was also most likely concentrated on the internal surfaces of the cover concrete that were adjacent to the top surface of corroding bars as shown in Figure 4.2. It was however, discussed in Chapter Two that the majority of analytical models for concrete cover cracking due to steel corrosion often assume corrosion to be uniformly distributed around the surface of bars as shown in Figure 2.1 [4-6]. Figures 2.1 and 4.2 suggest that these models are unlikely to apply to real situations where corrosion agents often reach the steel from one face of the structure. It will be shown in Chapter Six that the distribution of vertical and transverse strains on the exterior surfaces of corroded beams also confirms the non-uniformity of the corrosion process around corroding bars.

Figure 4.2  Internal pressure applied by partial corrosion of steel surface
It is also worth pointing out that with the loss in steel limited the top surface of a bar, the remaining shape of the corroded bar is not circular. Therefore, measuring pit depths will most probably underestimate its residual cross-sectional area. It is however, the cross-sectional area of the bar that relates to the stresses on the steel due to applied loads. Pit depth may therefore underestimate the residual capacities of corroded RC structures. Considering the amount of resources that continue to be used to repair corroded RC structures, it is important to predict accurately, the residual load-bearing capacities of the structures.

As already mentioned, the severest corrosion attack on steel bars was on the steel ribs rather than the core section of the bar. Since nominal diameter of deformed bars is calculated considering the rib heights [7], their loss should undoubtedly also be considered when assessing the level of steel corrosion. Figure 4.1 therefore agrees well with the experimental procedure used in this research which specified measuring mass loss of steel coupons as opposed to average mass loss or pit depths. It is also evident from Figure 4.1a that the severest level of steel corrosion was towards the centre of the corrosion region. The next section of the chapter quantifies this level of corrosion.

**4.4 Mass loss of steel under accelerated corrosion**

From equation 3.2, the average mass per length of uncorroded steel coupons, $m_{ur}$, was found to be 888 g/m of bar length. This mass per length is identical to the mass per length of the steel bars that was provided by the supplier and the South African Code of Practice [7]. This indicates that the cleaning process used in the experimental programme effectively removed concrete particles around the steel but did not remove the base metal.

**4.4.1 Loss of steel per bar along the corrosion region**

Figure 4.3 shows the variation of mass loss of each of the reinforcing bars along the corrosion region for selected beams in the research with target level of steel corrosion of 10%. Other beams had similar variation of steel loss. The zero point on the horizontal axis of the graphs represents the edge of the corrosion region on
the current input side, and 700 mm represents the edge of the corrosion region on the current output side. The figure shows that there was a significant variation of mass loss of steel along the corrosion region, with the maximum mass loss of steel occurring around the centre of the region. Very little loss in the area of steel was observed beyond the ends of the corrosion region. This indicates that the experimental set-up used was able to limit the length of the corrosion area as required. It is also evident from the figure that for the majority of beams, the maximum mass loss of steel bars occurred on the centre bar and close to the middle of the corrosion region. Towards the ends of the corrosion region however, the mass loss on the centre bar was not necessarily larger than the mass loss on the exterior bars. It was thus necessary to determine the total loss of steel on each corroded bar. Instead of simply presenting the total mass loss of each bar, cumulative mass loss of each bar along the corroded region was calculated by accumulating individual mass losses of steel coupons from the current input side to the output side. In this way, a section along the beam where mass losses for each bar varied the most could be established.

Figure 4.4 shows that for the majority of beams, the centre bar had a larger rate of steel corrosion than the exterior bars. The difference between the cumulative mass losses of different steel bars along the corroded length was mostly between 50 and 600 mm from the current input edge of the corrosion region. At the edges of the corrosion region, the graphs are almost flat indicating that all steel bars corroded at about the same low rate. However, they were expected to corrode at the same rate over the entire corrosion region because they had the same dimensions and should have equally shared the applied current. The differences in the rates of corrosion of steel can be attributed to corrosion crack patterns. It will be shown in Chapter Six that even though a number of crack patterns were observed on the surfaces of beams during the corrosion process, for the majority of beams, the widest cracks were exhibited on the tensile face of beams. Due to the location of the dominant cracks, in this research the corrosion agents probably reached the centre bar much easier than the exterior bars which resulted in the centre bars having larger corrosion rates. Further assessment of the differences in the rates of
Designation: AC* = accelerated corrosion with 4-day drying cycles; AC = accelerated corrosion with 2-day drying cycles; PR = patch repair; and a number after the two letters indicates the level of sustained load as a percentage of the load-bearing capacity of a control beam.

Figure 4.3  Variation of mass loss per bar along the beam
Designation: AC* = accelerated corrosion with 4-day drying cycles; AC = accelerated corrosion with 2-day drying cycles; PR = patch repair; and a number after the two letters indicates the level of sustained load as a percentage of the load-bearing capacity of a control beam.

Figure 4.4 Cumulative mass loss per bar along the beam
corrosion of steel bars is necessary for corrosion under a natural rate. This is because the natural anodic protection of bars under natural corrosion on the basis of which bar corroded first does not exist when anodic current is applied in an accelerated test. With that, the structural performance of a corroded structure is mostly related to the total loss in the area of steel at every section along the structure rather than the loss per bar.

It should be mentioned that there were differences in mass losses of steel depending on the procedure of accelerated corrosion used. To avoid repetition, they will be discussed under the section on total loss of steel.

4.4.2 Total loss of steel

The mass loss of steel for every 50 or 100 mm section of the corroded length (depending on the length of steel coupons) was calculated as the average percentage mass loss of the three steel coupons corresponding to that section and its variation along the beam is shown in Figure 4.5. The figure shows that there was clear variation of mass losses of steel between beams that were corroded using two-day drying cycles and those where four-day drying cycles were used. The discussion of Figure 4.5 will therefore be separated between the procedures. To help the reader follow the discussion on the effect of the history of beams on the rate of steel corrosion without needing to refer to Chapter Three, the schematic of the experimental programme (Figure 3.1) is shown after Figure 4.5.

4.4.2.1 Total loss of steel in beams under two-day drying cycles

It is evident from Figure 4.5 that the variation of the mass losses of steel along the corrosion region was such that the maximum mass loss of steel was close to the centre of the region. As already mentioned, little losses of steel were observed at both ends of the corrosion region. Interestingly, the figure indicates that the influence of the level of the sustained load on the rate of steel corrosion was not clear. For example, beams 17 to 19 (corroded under a load of 12%) had average mass losses of steel that ranged from 7.8 to 8.6%. Respective ranges of average
mass losses of steel for beams under a load of 8% and under a load of 1% were 7.1 to 7.8% and 6.5 to 7.8%. Average mass loss was calculated only within the corrosion region because predictions from Faraday’s Law used a corroded length of bars of 700 mm. If the entire area under the mass loss curve was to be used, these values will be slightly lower. Even then, the average mass losses of steel were less than the 10% target mass loss of steel that was estimated from Faraday’s Law. The non-influence of load on the rate of accelerated steel corrosion and over-predictions of Faraday’s Law were also observed by El Maaddawy et al. [8-9] and Ballim et al.[10-11], discussed in Chapter Two. To confirm the non-
Designation: UC = uncorroded; AC* = accelerated corrosion with 4-day drying cycles; AC = accelerated corrosion with 2-day drying cycles; NC = natural steel corrosion; PR = patch repair; FP = repair with FRP plate; FW = wrap beam and FRP plate with FRP sheets; and a number after the two letters indicates the level of sustained load as a percentage of the load-bearing capacity of a control beam.

Figure 3.1 Schematic of the experimental programme (from Chapter Three)
influence of load on the rate of steel corrosion, these mass losses of steel were plotted against Faraday’s predictions and compared with previous mass losses of steel found by other researchers discussed in Chapter Two and outlined in Appendix A (Figure 4.6a). Certainly, average mass losses of steel on beams that were corroded using two-day drying cycles were within the range of results found by previous researchers. The variation of mass losses of steel in Figure 4.5 however, questions the relation between maximum mass loss of steel and predictions from Faraday’s Law.

Figure 4.6b shows the relation between maximum mass loss of steel and predicted loss from Faraday’s Law. Except for beam 9, which had a maximum mass loss of steel of 9.5%, other beams had maximum mass losses of steel that were greater than predicted losses. Beam 19 had a maximum mass loss of steel of 12.1% compared to 10% from Faraday’s Law. Despite these larger mass losses of steel, Figure 4.6b shows that they are still within the range of values that were observed by previous researchers. Their consistency however, points to the need to reassess predictions of mass loss of steel using Faraday’s Law. Also note that maximum mass losses of steel were up to two times larger than average mass losses of steel. This probably explains the differences in relations between load-bearing capacity and the various mass losses of steel that were discussed in Chapter Two. These relations will be discussed again later in the chapter.

4.4.2.2 Total loss of steel in beams under four-day drying cycles

As previously pointed out, mass losses of steel coupons in beams with four-day drying cycles were consistently larger than corresponding losses in beams with two-day drying cycles. The most obvious reason that can be attributed to beams with longer drying cycles having larger mass losses of steel is that 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. It was mentioned in Chapter Three that for beams to have the target level of steel corrosion of 10%, beams corroded using two-day drying cycles were tested for 64 days (44 wetting days + 20 drying days). Beams tested with four-day drying cycles were however, tested for 80 days (44 wetting days + 36 drying days). This implies that beams
Figure 4.6  Measured mass loss of steel versus predicted mass loss

(a) Average mass loss of steel versus predicted loss

(b) Maximum mass loss of steel versus predicted mass loss
under the four-day drying cycles had 16 days additional natural corrosion compared to beams under the two-day drying cycles. It will however, be shown later that the natural corrosion rate in beams was too low to have resulted in the recorded large mass losses of steel in beams with four-day drying cycles. The large differences in mass losses here could be ascribed to the set-up with two-day drying cycles not allowing the complete dryness of the concrete cover at the corroded rebar depth. Therefore, after the drying period, less stable products such as ferrous hydroxide (which according to Figure 2.2 occupy a larger volume than dryer products) would still be available within the corrosion region. On the other hand, more stable products such as haematite and magnetite (which occupy less volume) could have formed during the four-day drying periods. The formation of these lesser volumetric compounds could have allowed for more access of corrosion agents to the rebar, which could have led to larger corrosion rates. It will be interesting to see how this affected the rate of widening of corrosion cracks (Chapter Six).

Average mass losses of steel on beams with four-day drying cycles were also plotted in Figure 4.6a. Clearly, these mass losses of steel were consistently larger (up to 1.4 times) than mass loss of steel that was predicted using Faraday’s Law. Compared to beams that were exposed to two-day drying cycles, mass loss on beams under four-day drying cycles were up to two times larger. In terms of maximum mass loss of steel, Figure 4.6b shows that mass losses on beams under four-day drying cycles were significantly larger (up to 2.3 times) than mass loss of steel predicted from Faraday’s Law. It is important to note that the set-up of four-day drying cycles is closer to in-service conditions than when shorter drying cycles were used. It therefore confirms earlier discussions that accelerated procedures used by previous researchers, such as completely immersing samples in salt solution, are not representative of in-service conditions. To aggravate the problem, these results indicate that they may slow down the rate of steel corrosion. To avoid unexpected failures of structures which have been assessed using those relations, more research is needed where conditions similar to those used here are followed.
4.4.2.3 Rate of steel corrosion during accelerated corrosion phase

Overall, it is evident from Figure 4.5 that the mass loss of steel for beams with the same duration of drying cycles should be represented as an envelope as opposed to an average (Figure 4.7). From this figure and with the duration of the accelerated corrosion period, it can be shown that beams with two-day drying cycles had a lower envelope maximum mass loss of steel of 1.10 g/day/m of bar length and an upper envelope maximum mass loss of 1.68 g/day/m of bar length. Theoretical loss from Faraday’s Law was 1.39 g/day/m of bar length. The corresponding rates of loss of steel for beams with four-day drying cycles were 1.68 g/day/m of bar length and 2.60 g/day/m of bar length respectively. These rates emphasise the earlier notion that the rate of corrosion was larger for beams with longer drying cycles. They also confirm that there was another phenomenon (natural corrosion aside) that caused the large rates of corrosion in beams with longer drying cycles. The implication of the large differences in mass losses of steel between the drying cycles with respect to the residual load-bearing capacity and corrosion crack widths of the beams will be discussed later and in Chapter Six, respectively.

4.5 Natural corrosion of steel after repairs

An interesting feature about the variation of mass loss of steel shown in Figure 4.5 is that whilst Figure 3.1 shows that patch repaired beams were tested 63 to 115 days from the end of the accelerated corrosion phase depending on the type of repairs selected for the beam, the level of corrosion in patch repaired beams was about the same as that obtained in corroded but non-repaired beams. For example, beam 14 which was tested 115 days after patch repairs, had a maximum mass loss of steel of 10.6% compared to 12.6% on beam 17 which was tested immediately after accelerated corrosion and 10.5% on beam 13 which was tested 63 days after patch repairs. As expected from discussions in Chapter Two, this indicates that there was little further steel corrosion after patch repairing beams. It is also important to reiterate that similar to patch repairs on real structures, corroded bars as well as sections of the substrate concrete to be bonded with patch mortars, were coated with an epoxy bonding agent. Intentions were to increase the bond strength
between repairs and the substrate concrete and also to form a barrier that prevents chlorides from reaching corroded steel bars. It is therefore evident that if effectiveness of repairs to increase the service life of a corroding structure is measured in terms of its ability to reduce further steel corrosion, the procedure for patch repairs used in this research (and also in the field) was effective at increasing the service life of corroded structures.

Figure 4.7 Envelope for variation of mass loss of bars along the beam
Also indicated in Figure 4.7 is the mass loss of steel in beam 15 which was firstly corroded by impressing an anodic current to a target corrosion of 10% and then strengthened with FRPs without patch repairs (Figure 3.1). Following strengthening with FRPs, the beam was allowed to corrode naturally. Steel bars were retrieved from the beam after 249 days of testing (44 days of impressed current + 20 drying days during the accelerated test + 185 days of natural steel corrosion). It is clear from Figure 4.7 that the mass loss of steel of the beam exceeded the upper envelope of mass losses of steel from beams that were only corroded to 10% corrosion under two-day drying cycles or later patch repaired. It is also evident from the figure that this mass loss was largely within the envelopes of mass losses of beams that were exposed to four-day drying cycles. The area that is bound by mass loss of steel from beam 15 and envelopes of mass losses of steel from beams with two-day drying cycles certainly belongs with natural steel corrosion. If drying days during accelerated tests are taken to belong with natural steel corrosion, then beam 15 had 205 days of natural steel corrosion compared to 36 days on beams with four-day drying cycles. The large difference in days of natural steel corrosion between beam 15 and beams 4 to 7 and their near-equal mass losses of steel confirms the earlier notion that the large mass loss of steel in beams with four-day drying cycles was not necessarily a result of extended natural corrosion.

If during accelerated steel corrosion, the rate of loss of steel of beam 15 is assumed to have followed the lower envelope of mass losses of steel from other beams that were exposed to two-day drying cycles, it can be shown that the natural rate of corrosion in beam 15 was 0.603 g/day/m of bar length. If however, the upper envelope mass loss of steel is assumed, the natural corrosion rate in beam 15 was 0.403 g/day/m of bar length. As expected, the natural corrosion rate was much lower than the rates of steel during the accelerated corrosion period which ranged from 1.10 to 1.68 g/day/m of bar length. For the size of bars used in this research (12 mm) and following Faraday’s Law, the maximum rate of mass loss of steel during the natural corrosion process in beam 15 corresponds to current densities between 45.5 to 66.5 µA/cm² if the upper envelope and the lower envelope from beams with two-day drying cycles is assumed, respectively. These current densities are within the range of current values of 0.1 to 100 µA/cm².
measured in real structures under natural corrosion [12-14]. Contrary to patch repairs, these results indicate that significant corrosion occurred in beams that were only repaired with FRPs. This is expected (Chapter Two) as repairs with FRPs do not entail removal of corrosion agents from the substrate concrete. Compared to results from Badawi and Soudki [15] and from El Maaddawy et al. [16] (discussed in Chapter Two), these results are closer to in-service conditions because after repairs, beams were allowed to corrode naturally.

4.6 Discussion of results on mass loss of steel

The variation of mass losses of steel shown in Figure 4.5 can be attributed to corrosion crack widths and reinforcement configuration of beams (Figure 3.2). It will be shown in Chapter Six that corrosion crack widths were larger at the centre of the corrosion region most probably because concrete at the ends of the region was confined by stirrups placed on the shear span. These cracks could have allowed for more corrosion agents to reach the steel bars at the centre of the region than at the ends of the region. Note that the reinforcement configuration in this research (Figure 3.2) follows normal design procedures for real structures where few shear stirrups are placed in constant moment regions. These results show that in those regions, measuring average mass loss of steel as done by previous researchers, will underestimate the maximum mass loss of steel. Certainly, more harm with measuring average mass loss of steel will be experienced when the constant moment region coincides with the location of maximum stresses from external loads. This research presents this worst scenario.

The variation of mass loss of steel is expected to be different if steel corrosion was to occur in a section of a RC member that is reinforced with shear stirrups. The uniform confinement of concrete from the stirrups will most likely result in maximum corrosion cracks randomly occurring around the corrosion region as found by Zhang et al. [17] (discussed in Chapter Two). If maximum mass loss of steel is to occur at locations of maximum crack widths (because they allow more ingress of corrosion agents), the location of maximum mass loss of steel will also be random. It should also be mentioned that stirrups, by virtue of having a smaller cover depth compared to the main reinforcement, are likely to corrode first. In
those situations, less corrosion is expected on the main reinforcement. Further research is needed to confirm this notion.

Another important matter worth discussing is that in beams with two-day drying cycles, mass losses of steel varied smoothly along the corroded region with maximum mass loss occurring around the centre of the corrosion region. In beam 15 and beams 4 to 7 however, there was more localised loss of steel bars. For beams with four-day drying cycles, localised corrosion can be attributed to variations in the rate of drying of the corroded region. Sections that dried the most are expected to have had a large composition of more stable compounds which occupy less volume compared to unstable compounds. They are therefore likely to have corroded more. Expectedly, these will be areas with maximum crack widths which Figure 4.5 and Chapter Six agree to have been the middle of the corrosion region. Localised corrosion in beam 15 is due to a different mechanism. Note that during accelerated tests, the cathode was artificially selected so that steel bars acted at anodes and corroded. When the anodic current was stopped and the cathode removed, corrosion continued by naturally sacrificing other sections of steel bars and having others act as cathodes. Despite the differences in the mechanisms that cause localised corrosion in beam 15 and beams 4 to 7, they exhibited near-similar variation of mass losses of steel. Later sections of this chapter, in agreement with discussions in Chapter Two, will show that more harm to the load-bearing capacity was caused by localised steel corrosion. These results suggest that to simulate this localised corrosion, which surely exists in real structures, an accelerated corrosion set-up should entail long drying cycles. As previously mentioned, completely immersing samples in salt solution during accelerated tests will not give this localised corrosion. It is therefore likely to underestimate the effects of corrosion on real structures.

Despite steel corrosion in beam 15 being much closer to natural steel corrosion, it wasn’t purely natural. When impressed current was stopped following completion of the accelerated corrosion phase, corrosion agents had already been forced into the concrete. Propagation of steel corrosion was therefore much easier. This most probably explains the large current densities (45.5 to 66.5 \( \mu A/cm^2 \)) that were
found. More research where natural steel corrosion is used is therefore recommended.

Similar to results from Ballim et al. [10-11] and from El Maaddawy et al. [8-9], the level of sustained load was found to have little effect on the rate of steel corrosion. Prior to cracking of the cover concrete due to steel corrosion, certainly flexural cracks in RC structures would allow an easier ingress of corrosion agents into the concrete. At this stage, the level of sustained load is expected to significantly influence the rate of steel corrosion. However, after cracking of the cover concrete due to steel corrosion, corrosion cracks which are often much wider than flexural cracks, would form the primary channel for ingress of corrosion agents. The influence of the level of sustained load on the rate of steel corrosion on corrosion-cracked RC structures is expected to be insignificant. As discussed in Chapter One, this research is limited to the behaviour of RC structures during this corrosion propagation stage.

The set-up of limiting the ingress of corrosion agents into the concrete used in this research resulted in partial surface steel corrosion. Surfaces of bars that were opposite the direction of ingress of corrosion agents were barely corroded. Figure 4.6a shows that the average mass loss of steel was however, comparable to those found by previous researchers where steel corrosion was uniform. This indicates that the amount of corrosion products between the different set-ups was the same. Logically, by limiting these products to part of the surface of a corroding bar implies that stresses that they applied on the cover concrete were also localised around the corroding surface. They are therefore expected to have been larger and hence caused more expansion of concrete than when uniform steel corrosion occurred. This will be confirmed in Chapter Six. As discussed in Chapter Three, the set-up used in this research is closer to in-service conditions. This again means that previous experimental set-ups could have underestimated the damage that steel corrosion causes on in-service structures.
4.7 Ultimate capacity of corroded and repaired RC beams

To provide more understanding on relations between residual load-bearing capacities of corroded and repaired RC beams with the level of steel corrosion, a general theoretical model will firstly be presented. It will later be compared with experimental results.

4.7.1 Model for ultimate capacity of corroded and repaired RC beams

Figures 4.8 and 4.9 show the distribution of internal forces in a beam cross-section and strain-stress relations of various materials that are necessary for the prediction of the load-bearing capacity of RC beams corroded and repaired with FRPs. To avoid constantly referring to Chapter One, notations necessary for the model are presented again here;

\[ A_{sc} = \text{area of compression reinforcing steel} \]
\[ A_{st} = \text{residual area of tensile reinforcement after corrosion} \]
\[ b = \text{width of concrete section} \]
\[ x = \text{depth of the neutral axis of the beam as measured from the extreme compression fibre} \]
\[ d_c = \text{distance from the extreme compression fibre to the centroid of tensile reinforcing steel} \]
\[ d' = \text{distance from the extreme compression fibre to the centroid of compression reinforcing steel} \]
\[ E_f = \text{Young’s modulus of FRP plate} \]
\[ E_s = \text{Young’s modulus of steel before yielding} \]
\[ E_{sp} = \text{Young’s modulus of steel reinforcement after yielding} \]
\[ f' = \text{concrete compressive strength} \]
\[ f_c = \text{stress at the extreme compression fibre of concrete} \]
\[ F_c = \text{internal compression force carried by the concrete in compression} \]
\[ f_f = \text{stress applied on FRP plate} \]
\[ F_f = \text{internal tensile force carried by FRP plate} \]
\[ f_{fu} = \text{rupture stress of FRP plate} \]
\[ f_s = \text{stress applied on tensile steel reinforcement} \]
\[ F_{sc} = \text{internal compression force carried by the compression steel reinforcement} \]
Figure 4.8  Strains and internal forces on a FRP-repaired RC beam at failure

Figure 4.9  Stress-strain relations of various materials
\( F_{st} \) = internal tensile force carried by the tensile steel reinforcement
\( f_{su} \) = ultimate tensile strength of tensile steel reinforcement
\( f_y \) = yield stress of tensile steel reinforcement
\( h \) = height of the beam
\( k_1 \) = ratio of the average stress of compression stress block to the stress applied at the extreme compression face of the concrete
\( k_2 \) = ratio of the location of the neutral axis of the beam to the location of the resultant internal compression force carried by the concrete (all measured from the extreme compression fibre)
\( M_u \) = ultimate capacity of the beam
\( t_f \) = thickness of FRP plate
\( w_f \) = width of FRP plate
\( \varepsilon_c \) = concrete strain at the extreme compression fibre
\( \varepsilon_f \) = strain in the FRP plate
\( \varepsilon_{df} \) = debonding or usable strain of FRPs
\( \varepsilon_{fu} \) = rupture strain of FRP plate
\( \varepsilon_o \) = concrete strain corresponding to the concrete compression strength \( f'_c \)
\( \varepsilon_{sc} \) = strain in the compression steel reinforcement
\( \varepsilon_{st} \) = strain in the tensile steel reinforcement
\( \varepsilon_{su} \) = ultimate strain of tensile steel reinforcement
\( \varepsilon_u \) = ultimate strain of concrete in compression
\( \varepsilon_y \) = yield strain of tensile steel reinforcement

It was mentioned in Chapter Three that following guidelines from various codes of practice such as the South African National Standard [18], the design of RC beams in this research was such that the mode of failure of an uncorroded RC beam under flexure was yielding of steel in tension followed by crushing of concrete in compression. As discussed in Chapter Three, hooks were provided on the compression reinforcement in the centre region (which had no stirrups) to prevent buckling of the steel. Since steel corrosion was limited to the tensile reinforcement, it is logical that failure of a corroded RC beam that was not repaired with FRPs also occurred after yielding of tensile steel reinforcement.

Design for repairs with FRPs was such that the above failure mode was also attained even for an uncorroded RC beam. Note that there is little practical benefit
of repairing undamaged RC structures. Their behaviour has however, been widely studied [19-22]. As indicated in Chapter Three, uncorroded beams in this research were therefore not repaired. Another important design consideration with repairs was that theoretically, a FRP-repaired beam that did not have tensile steel bars (assuming steel corrosion of 100%), failed by crushing of concrete in compression before rupture of FRP plates. This mode of failure is considered by Bakis et al. [20] and ACI Committee 440 [19] to be more favourable compared to rupture of FRP plates prior to crushing of concrete in compression. It is worth noting that in predicting the failure mode of repaired beams during design, ultimate strains of materials shown in Figure 4.9 were used with the assumption of no delamination of FRPs. How delamination of FRPs was considered will be presented later.

From Figure 4.8, the equilibrium of the external applied moment, $M_u$, with internal couples due to various internal forces in the beam, about the location of the tensile reinforcement, requires that;

$$M_u = F_c (d_e - k_2 x) + F_{sc} (d_e - d') + F_f (h - d_e + \frac{t_f}{2})$$  \hspace{1cm} (4.1)

Where;

$$F_c = k_1 f_{f's} bx$$  \hspace{1cm} (4.2)

$$k_1 = \frac{\varepsilon_c}{\varepsilon_o} \left(1 - \frac{1}{3} \frac{\varepsilon_c}{\varepsilon_o}\right)$$  \hspace{1cm} (4.3)

$$k_2 = 1 - \frac{2}{3} \left[ \frac{1 - \frac{3}{8} \frac{\varepsilon_c}{\varepsilon_o}}{1 - \frac{1}{3} \frac{\varepsilon_c}{\varepsilon_o}} \right]$$  \hspace{1cm} (4.4)

$$F_{sc} = A_{sc} \varepsilon_{sc} E_s$$  \hspace{1cm} (4.5)

$$F_f = t_f w_f \varepsilon_f E_f$$  \hspace{1cm} (4.6)
The equilibrium of resultant internal forces (Figure 4.8c) in the beam requires that;

\[ F_{st} + F_f = F_c + F_{sc} \]  \hspace{1cm} (4.7)

Since the design of the beams was such that failure occurred by yielding of tensile reinforcing steel followed by crushing of concrete in compression, the internal tensile force that was resisted by the steel at the time of failure, \( F_{st} \), must incorporate the post yield properties of steel shown in Figure 4.9a.

\[ F_{st} = A_{st} f_y + A_d E_y \left( \varepsilon_{ut} - \varepsilon_y \right) \]  \hspace{1cm} (4.8)

Assuming compatibility of strains across the depth of the beams, if the strain at the compression face of concrete (\( \varepsilon_c \)) is known, corresponding strains at different fibres of the section of the beam can be calculated from equations 4.9 to 4.11.

\[ \varepsilon_{sc} = \frac{\varepsilon_c (x-d')}{x} \]  \hspace{1cm} (4.9)

\[ \varepsilon_{st} = \frac{\varepsilon_c (d_c - x)}{x} \]  \hspace{1cm} (4.10)

\[ \varepsilon_f = \frac{\varepsilon_c \left( h-x+\frac{t_f}{2} \right)}{x} \]  \hspace{1cm} (4.11)

According to the SANS Standard [18], El Tawil et al. [23] and Hsu [24], concrete is considered to crush in compression at a strain of 0.003. The corresponding ultimate capacity (crushing of concrete in compression) can be calculated from equations 4.1 to 4.11 by iteratively estimating a value of the depth of the neutral axis, \( x \), that satisfies equation 4.7. In this research, a Matlab script was written and used to run the iteration. For beams that were not repaired with FRPs, there is no contribution of the FRP plates to internal couples as well as internal forces in equations 4.1 and 4.7 (\( F_f = 0 \)).
It is interesting to note that since corrosion reduced the area of tensile steel and that RC beams were double reinforced, there is a possibility from a structural mechanics viewpoint that for corroded beams, failure was fracture of tensile reinforcing steel followed by crushing of concrete in compression. Note that the fracture strain of tensile steel bars that were used in the research ($\varepsilon_{su} = 0.07$) was larger than the rupture strain of FRP plates ($\varepsilon_{fu} = 0.0125$) and the design for a FRP-repaired beam without tensile steel reinforcement was such that it failed by crushing of concrete in compression. Fracture of tensile reinforcing steel before crushing of concrete in compression was therefore only possible on beams that were not repaired with FRPs.

If tensile steel reinforcement fractures in a RC beam with no FRPs, the beam is only able to resist load if the depth of the neutral axis reduces drastically so that the (original) compression reinforcement carries internal tensile forces. The location, the area and the material properties of the compression steel reinforcement used in this research were however, such that this additional load would be insignificant. Fracture of tensile reinforcement can therefore be taken to imply complete failure of a beam. Theoretical capacity of RC beams where failure was controlled by fracture of steel in tension as opposed to crushing of concrete in compression can also be calculated using equations 4.1 to 4.11 but only limiting the strain in the tensile steel ($\varepsilon_{st}$) to 0.07 (equation 4.10).

It should also be noted that from a practical viewpoint, design of RC beams for flexure (without FRPs such as in this research) is often such that after yielding of steel in tension, there is insignificant additional load that the beam can handle. Ultimate capacity of beams without FRPs in this research can therefore, also be calculated by limiting the strain in the tensile steel in equation 4.10 to its yield strain ($\varepsilon_y = 0.0026$). In calculating the capacity of beams, the strain in the compression steel reinforcement must be checked for possible yielding of the steel. If at failure the compression steel has yielded, equation 4.5 must be expressed similarly to equation 4.8 but using the material properties of compression steel from Chapter Three.
4.7.2 Failure mode of corroded and patch-repaired RC beams

As expected from the design of RC beams, an uncorroded RC beam failed by yielding of steel in tension followed by crushing of concrete in compression. No buckling of compression reinforcement was observed. A large crack on the tensile region of the beam, locating the point of yielding of tensile steel was 50 to 100 mm from either end of the constant moment region as shown in Figure 4.10. Few (two to three) large flexural cracks were observed around the failure point. Expectedly, crushing of concrete on the compression face occurred opposite the point of yielding of tensile steel bars.

Figure 4.10 Failure mode of a non-corroded RC beam
Similar to control beams, all corroded beams that were not repaired failed by yielding of steel in tension followed by crushing of concrete in compression. The point of failure was however, at most 200 mm from the centre of the corrosion region. Again similar to the control beam, only two to three large flexural cracks were observed in corroded beams as shown in Figure 4.11. Generally, the flexural cracks ran directly across longitudinal corrosion cracks (Figure 4.11). In beams 9 and 17, combined flexural cracks due to applied loads and longitudinal cracks from steel corrosion detached the entire cover concrete within the corroded region from the parent concrete at loads between 60 and 70% of its failure load as shown in Figure 4.12. As will be described in Chapter Six, these beams had dominant corrosion cracks on the side faces. In other beams, such as beam 4, shown in Figure 4.11, delamination of the cover concrete was only observed on sections of the corroded region where the beam exhibited side cracks. For the cover concrete to delaminate due to flexural cracks and corrosion cracks, then certainly it wasn’t attached to the parent concrete beam. This will be shown in Chapters Five and Six to have a significant influence on the magnitude of longitudinal strains and the rate of widening of corrosion cracks, respectively.

Clearly, the failure point of corroded beams was where Figure 4.5 indicated as the location of the maximum mass loss of steel. Interestingly, in the heavily corroded beams (beams 5 and 6) one of the exterior steel bars fractured at failure. A loud bang indicating fracture of the steel during testing occurred simultaneously with crushing of concrete in compression.

Corroded and patch-repaired beams also failed by yielding of steel in tension followed by crushing of concrete in compression. Similar to corroded but non-repaired beams, the point of failure in patch-repaired beams was within the corroded region and close to the centre of the region. Contrary to non-repaired beams, more flexural cracks (despite being smaller) were observed on repaired beams. They were mostly within the corroded region. Those closer to the failure point ran directly across the interface of the patch-substrate concrete. Others however, ended at the interface and then propagated along the interface (in the longitudinal direction) towards the failure point. They then joined flexural cracks at the point of failure. Longitudinal cracks at the interface are a sign of debonding.
Figure 4.11  Failure mode of a corroded RC beam

a)  Side view of a corroded RC beam at failure

b)  Tensile face of a corroded RC beam at failure (beam inverted)

Figure 4.11  Failure mode of a corroded RC beam
Figure 4.12 Delamination of cover concrete of a corroded RC beam

a) Side view of a corroded RC beam at failure

b) Tensile face of a corroded RC beam (lying on the side face)

Figure 4.12  Delamination of cover concrete of a corroded RC beam
a) Side view of a patch-repaired RC beam at failure (inverted)

b) Close view of a patch-repaired RC beam at failure (inverted)

Figure 4.13  Failure mode of a corroded and patch-repaired RC beam
of the patch repairs. Intriguingly, no longitudinal cracks at the patch-concrete interface were observed away from the corroded region. The ends of the repairs where they joined the parent concrete beams were free from cracking. As found by other researchers [14,25] (discussed in Chapter Two), this implies that patch repairs did not debond from the substrate concrete. A typical failure mode of a corroded and patch-repaired beam is shown in Figure 4.13. The location of the interface is not clear because the beam was painted in white to show its cracking behaviour.

4.7.3 Results on ultimate capacity of corroded and patch-repaired RC beams

All the failure modes above imply that equations 4.1 to 4.11 can be used to predict load-bearing capacities of corroded and patch-repaired RC beams using the limit of the ultimate strain of concrete in compression. In confirmation, at theoretical failure of the beams (including beams 5 and 6) by crushing of concrete in compression, the tensile strain in the steel bars (using the maximum loss of steel) was below the fracture strain of tensile steel used in the research. Fracture of steel in beams 5 and 6 can therefore be attributed to the redistribution of stresses along the failure-section of the beam due to crushing of concrete in compression.

Figure 4.14 shows the variation of ultimate load-bearing capacities of corroded and patch-repaired RC beams with measured maximum as well as average mass losses of steel. Also presented in the figure is the variation of the theoretical load-bearing capacity of beams calculated using equations 4.1 to 4.11, a target compressive strength of concrete of 40 MPa (from Chapter Three) and arbitrary mass losses of steel. Trends (best represented by equations 4.12 to 4.16) can be observed in each plot that the ultimate moment capacity of beams reduced linearly with an increase in the level of steel corrosion. These trends were also reported by other researchers [26,27].

\[ M_{u\_unrepaired} = -0.33Q_{\text{max}} + 39.1 \quad R^2 = 0.96 \]  
\[ M_{u\_repaired} = -0.32Q_{\text{max}} + 40.2 \quad R^2 = 0.96 \]
The theoretical v’s Arbitrary mass loss

Unrepaired v’s Max. mass loss

Repaired v’s Max. mass loss

Unrepaired v’s Avg. mass loss

Repaired v’s Avg. mass loss

Trendlines

\[
M_{u,\text{unrepaired}} = -0.68 Q_{\text{avg}} + 39.4 \quad R^2 = 0.94 \quad (4.14)
\]

\[
M_{u,\text{repaired}} = -0.82 Q_{\text{avg}} + 41.5 \quad R^2 = 0.82 \quad (4.15)
\]

\[
M_{u,\text{theoretical}} = -0.33 Q + 37.7 \quad R^2 = 1.00 \quad (4.16)
\]

Where; \(Q_{\text{max}}\) = measured maximum mass loss of steel (%); \(Q_{\text{avg}}\) = measured average mass loss of steel (%); and \(Q\) = arbitrary mass loss of steel (%).

Figure 4.14 Ultimate capacities of corroded and patch-repaired RC beams

It is clear from the figure that at a chosen level of steel corrosion, the load-bearing capacity of a patch-repaired RC beam was larger than the corresponding load-
bearing of a non-repaired RC beam. This can be attributed to the mode of flexural cracking of the beams. Multiple cracks on the patch-repaired beams were an indication of transfer of stresses from the tensile steel to the patch concrete (tension-stiffening effect). In addition, longitudinal cracks along the patch-substrate concrete interface which did not cover the entire patch region indicate that the interface bond also resisted some applied stresses. The result was that compared to non-repaired RC beams, in patch-repaired beams, lesser stresses were applied on the tensile steel which then increased their load-bearing capacities. However, from an engineering viewpoint, the increase in the capacity (2.5%) is small and can be ignored.

Another interesting feature from Figure 4.14 is the rate of loss of load-bearing capacities of beams with an increase in the level of steel corrosion. This is demonstrated by the slopes of the trends (equations 4.12 to 4.16). Note that ultimate capacities of repaired and non-repaired beams plotted against maximum mass loss of steel had near-equal gradients as the theoretical capacity of beams from arbitrary mass losses of steel. This implies that the amount of stresses that caused multiple cracking of repairs was independent of the level of steel corrosion. Equations 4.12 and 4.13 indicate that the cracking required an internal couple moment of 1.1 kN-m. These similar gradients also suggest that there was a common error, probably in the theoretical model, for both corroded and non-corroded beams. For instance, if this error is adjusted by adding 1.4 kN-m to the theoretical function in the figure, equations 4.12 and 4.16 become identical. Very different gradients were observed when load-bearing capacities of repaired and non-repaired beams were plotted against average mass losses of steel. This implies that it is difficult to model theoretical capacity of beams using average mass loss of steel. A factor, as suggested by Azad et al [28], is needed to adjust the calculated capacities of beams. No factor is however, needed when maximum mass loss of steel is used. Therefore, compared to the average mass loss of steel, the maximum mass loss of steel had a better relation with the theoretical capacity of corroded RC beams. This is quite logical considering that ultimate capacity is a localised property of a beam and hence should relate to localised loss of steel. It helps explain the location of the failure point of corroded beams (where maximum mass loss of steel occurred). Since it was shown earlier that steel corrosion was
mostly localised in RC beams under natural steel corrosion or under long drying cycles, the relation between ultimate capacity and mass loss of steel in Figure 4.14 emphasises the need to predict accurately the maximum mass loss of steel in corroding in-service structures.

In summary, for every 1% maximum mass loss of steel, there was a corresponding 0.33 kN-m or 0.8% reduction in the measured ultimate capacity, but for the same loss in the average mass loss of steel, there was a 1.7% reduction in the measured ultimate capacity. Similar results from previous researchers were discussed in Chapter Two [8,26-28]. Since the theoretical model related well with the maximum mass loss of steel, as previously discussed, the use of average mass loss to predict ultimate capacity at high mass losses of steel will surely overestimate it. For example, at an average mass loss of steel of 20%, the actual loss in capacity is expected to be 34% whilst the corresponding theoretical loss in capacity is 18%. Therefore, using average mass loss of steel in theoretical models of load-bearing capacity of RC beams might lead to engineers incorrectly concluding that a corroding structural member is safe.

4.7.4 Results on load-bearing capacity of FRP-repaired RC beams

Contrary to beams that were not repaired with FRPs which simply failed by yielding of tensile reinforcing steel followed by crushing of concrete in compression, FRP-repaired beams sequentially failed by; i) yielding of steel in tension ii) simultaneous rupture of FRP wraps at one end of the beam and debonding of the FRP plate adjacent to the ruptured wraps iii) rupture of other FRP wraps along the shear span with the debonded end of FRP plate iv) debonding of FRP plate at ruptured FRP wraps v) and finally, crushing of concrete in compression (Figure 4.15). For all beams, crushing of concrete in compression occurred in sections of beams that were not wrapped with FRP sheets. For non-patch-repaired beams, delamination of FRPs ended at the failure point. At that point, the cover concrete along the longitudinal corrosion cracks on the side faces of beams was detached from the parent concrete beam up to the next FRP wrap. A similar mode of failure was observed in patch-repaired beams. Here,
a) Debonding of FRP plate and cover concrete along corrosion cracks

b) Debonding of FRP plate and patch repair
c) Debonding of FRP plate limited to one shear span (inverted)

Figure 4.15 Failure mode of FRP-repaired RC beams

it was the patch repair that was delaminated. These different failure modes are shown in Figure 4.15. The variation of the load-bearing capacity of FRP-repaired RC beams with maximum level of steel corrosion is shown in Figure 4.16. Also shown in the figure are theoretical capacities of beams assuming a limiting strain of crushing of concrete in compression (as designed) and assuming debonding strain of FRP laminate. The debonding strain (4500 micro strains) was calculated using equations 2.15 and 2.16 from ACI Committee 440 [19] and material properties of FRPs used in this research (Table 3.2). The trends from various capacities are given by equations 4.17 to 4.19.

\[ M_{u,\text{measured-FRP}} = -0.09Q_{\text{max}} + 58.4 \quad R^2 = 0.16 \quad (4.17) \]

\[ M_{u,\text{theoretical-crushing of conc in comp.}} = -0.14Q + 71.7 \quad R^2 = 1.00 \quad (4.18) \]

\[ M_{u,\text{theoretical-debonding of FRP plate}} = -0.32Q + 61.9 \quad R^2 = 1.00 \quad (4.19) \]

Where; \( Q_{\text{max}} \) = measured maximum mass los of steel (%); \( Q_{\text{avg}} \) = measured average mass loss of steel (%); and \( Q \) = arbitrary mass loss of steel (%).
Expectedly, Figure 4.16 shows that FRP-repaired RC beams had much larger capacities compared to non-FRP-repaired beams (Figure 4.14). Also anticipated from the failure mode of the FRP-repaired beams, theoretical capacity of beams that was calculated using the limit strain of crushing of concrete in compression is much larger than the measured capacity. On the other hand, it can be shown that the measured capacity was larger than the theoretical capacity assuming yielding of tensile reinforcing steel as the governing mode of failure. This suggests that failure of the beams was controlled by the delamination of the FRP plates. The crushing of concrete in compression shown in Figure 4.15 can therefore be ascribed to redistribution of stresses after the debonding of the FRP plates.

It is evident from Figure 4.16 that when the strain of debonding of FRPs was used as the governing failure strain for the ultimate capacity of beams, the theoretical

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**Figure 4.16** Ultimate capacity of FRP-repaired RC beams
capacity of FRP-repaired beams was much closer to the measured capacity. Contrary to non-FRP-repaired beams where high trends ($R^2 \approx 1$) in measured load-bearing capacities were found, in FRP-repaired beams, a low trend ($R^2 = 0.16$) was found. In accord with the failure mode of beams, this implies that the load-bearing capacity of FRP-repaired beams was not dependent on the level of steel corrosion. Furthermore, patch-repaired beams and those that were not patch-repaired had equal load-bearing capacities. Theoretical models however, show that the load-bearing capacity of corroded and FRP-repaired beams should linearly reduce with an increase in the level of steel corrosion. As expected from discussion in Chapter Two, the theoretical capacity of FRP-repaired beams was less sensitive to the variations in the level of steel corrosion [9]. For example, a model based on debonding of FRPs (equation 4.19) indicates that a 1% maximum loss in the area of steel resulted in a 0.32 kN-m or 0.5% loss in the capacity of FRP-repaired beams as opposed to 0.8% loss in those that were not repaired with FRPs.

### 4.8 Discussion on load-bearing capacity of repaired RC beams

Failure on non-corroded beams occurred near the ends of the constant moment regions. However, for corroded beams (even after patch repairs), failure occurred near the middle of the corrosion region. Figure 4.5 indicates that the failure point of corroded and patch-repaired beams coincided with the location of maximum mass loss of steel. This suggests that failure was controlled by maximum mass loss of steel. Note that previous discussion indicated the variation of steel to have been due to the centre region not being reinforced with stirrups (a design step that is often taken in real-structures). These results imply that in such cases, it is important to know accurately, the maximum mass loss of steel and not average mass loss of steel. In corroboration, theoretical load-bearing capacities of corroded as well as patch-repaired beams related well with maximum mass loss of steel but poorly with average mass loss of steel.

Figure 4.14 shows that patch-repaired beams had load-bearing capacities that were slightly larger than capacities on non-repaired beams. This was attributed to multiple cracking that was exhibited by the patch repairs. In real structures, these
cracks will certainly allow deleterious compounds such as corrosion agents to easily reach the steel bars. This point to the need to protect steel from further corrosion prior to patch repairs as done in this research.

As expected, significant increase in load-bearing capacities of RC beams were found when they were repaired with FRPs. Theoretically (equations 4.16 and 4.19), a mass loss of steel of 78% is required for a FRP-repaired beam to have a load-bearing capacity that is equal to the capacity of a non-corroded RC beam. It will be shown in Chapter Six that beam 15 also exhibited an increased rate of widening of corrosion cracks on the side faces after repairs. In addition, it will be shown that for beams 16 and 20, FRP repairs indirectly induced new corrosion cracks on the side faces. From the failure mode of beams 9 and 17 (which included delamination of the cover concrete), it was pointed out that corrosion cracks on the side faces indicate that the cover concrete is not fully-bonded to the parent beam. A FRP plate would therefore be bonded to cover concrete that is detached from the parent beam. It is expected to debond at a lesser strain. As indicated in Figure 3.1, beams 16 and 20 are under continued steel corrosion after FRP repairs to verify this failure mode. Therefore, despite the capacity of FRP-repaired beams being much larger than the capacity of non-FRP-repaired beams as well as not being very sensitive to changes in the level of steel corrosion, it is recommended that FRP-repairs of corroded RC structures should be carried out subsequent to patch repairs.

4.9 Conclusions

This chapter discussed the variation of steel loss along corroded RC beams and its effect on the load-bearing capacity of the beams. The effects of sustained loads and duration of drying cycles on the rates of corrosion were assessed. The chapter also looked at the effectiveness of patch and FRP repairs to control further steel corrosion and to increase the capacity of corroded RC structures. General discussions on the level of steel corrosion as well as load-bearing capacities of repaired RC structures were presented. The following are specific conclusions to this chapter.
1. It was found that for RC structures where the ingress of corrosion agents is limited to one external face of the structure, corrosion of steel bars is likely to be localised at the surface of the steel bars that faces the direction of ingress of the corrosion agents. This finding disputes assumptions of uniform steel corrosion around the surface of bars as used by previous researchers in various analytical models. They also contest using pitting as a measure of corrosion (even for smooth bars) because pitting assumes that the remaining bar diameter is uniform. More importantly, they suggest that uniform steel corrosion may underestimate the effects of steel corrosion in real structures. This is confirmed in Chapter Six.

2. The variation of the level of steel corrosion along the corrosion region was such that maximum mass loss of steel occurred around the centre of the corrosion region. This was attributed to the absence of stirrups within the corrosion region. It was found that the theoretical capacity of corroded beams was closely related to this maximum mass loss compared to the average mass loss of steel. For every 1% maximum mass loss of steel, there was a corresponding 0.8% reduction in the load-bearing capacity of beams. For the same loss in the average mass loss of steel, there was a 1.7% reduction in the capacity of beams. This finding challenges the use of average mass loss of steel as a measure of ultimate capacity and contends that it is likely to overestimate the residual capacity of corroding RC structures. Since there was a good linear relation ($R^2 = 0.96$) between the load-bearing capacity and maximum mass loss of steel, this finding also shows that measurable parameters of corroded RC structures (Chapters Five and Six) that are related to the maximum mass loss of steel can easily be used to predict residual load-bearing capacities of RC beams.

3. RC beams that were corroded under four-day drying cycles were found to have larger rates of corrosion compared to those where two-day drying cycles were used. This was attributed to longer drying cycles allowing for more drying of the corrosion area, therefore resulting in lesser volumetric products which subsequently allowed for more corrosion agents to reach the steel bars. These results indicate that for a given bar diameter, the level of steel corrosion...
during an accelerated corrosion test is not only dependent on the current density and the time of electrolysis (as stipulated by Faraday’s Law) but also on the conditions of the test programme. Considering the large variety of ways in which laboratory concrete specimens are corroded in the literature, it is important that a standardised testing programme be established. Mass loss of steel for RC beams that were allowed to corrode naturally following the accelerated corrosion process in this research suggest that accelerated corrosion with long drying cycles (> four-day drying) best relates with natural corrosion.

4. Patch repairs were found to be effective at controlling further steel corrosion. Repairs with FRPs were however, found not to control further steel corrosion. Natural rates of corrosion after these repairs were found to have a maximum of 66.5 µA/cm². As expected, FRPs were found to significantly increase the capacity of corroded RC beams. Since FRP-repaired beams failed by delamination of FRPs and it will be shown in Chapter Six that repairs with FRPs resulted in an increased rate of widening of corrosion cracks, it is foreseeable that these corrosion cracks might result in premature delamination of the FRPs. It was therefore recommended that FRP repairs of corroded RC structures should be carried out subsequent to patch repairs.

4.10 References

CHAPTER FIVE

LONGITUDINAL STRAINS AND STIFFNESS OF CORRODED AND REPAIRED RC BEAMS

5.1 Introduction

The principal cause of degradation of RC structures is corrosion of steel bars that are embedded in the concrete. As corrosion of steel bars occurs, there is a corresponding reduction in the area of steel and corrosion products deposited around the steel occupy a larger volume than the volume of the steel lost. The expansive corrosion products apply tensile stresses on the concrete surrounding the corroding steel bar, which can cause cracking and spalling of the cover concrete. The structural integrity of a RC structure undergoing corrosion damage is therefore reduced by the loss of bond between the corroding steel and the surrounding concrete, the loss of area of reinforcing steel and cracking of the cover concrete. Chapter Two indicated that measurable parameters of a corroded RC structure that are often used to indicate the level of steel corrosion and its residual load-bearing capacity are the time to first cracking, corrosion crack widths and stiffness. This chapter focuses on stiffness.

As discussed in Chapter Two, most work on behaviour of corroded RC structures was carried out in the absence of a sustained load. Stiffness at a selected level of steel corrosion was therefore obtained from slopes of load-deflection curves when corroded specimens were subjected to monotonically-increasing static loads. Most researchers [1-4] found stiffness to decrease with an increase in the level of steel corrosion. For example, Torres-Acosta et al. [2] found a mass loss of steel of 1% to cause a 1.4% loss in stiffness. Ting and Nowak [4] contended that 1% loss in cross-sectional area of steel corresponds to a 0.6% loss in stiffness. Contrary, Zhang et al. [5] found stiffness to only decrease with an increase in the level of steel corrosion at mass losses of steel that were below 40%. Above 40% mass loss of steel, they contended that stiffness remains constant despite an increase in the level of steel corrosion. Procedures they used to measure stiffness were criticised.
in Chapter Two for not representing in-service conditions, particularly where loads do not excessively vary. Stiffness changes under those conditions are primarily caused by a change in the level of steel corrosion. These changes in stiffness can only be simulated by corroding RC specimens under load.

Yoon et al. [6] and Ballim et al. [7,8] monitored deflections of RC specimens that were corroded under constant sustained loads. They found the corrosion process to significantly increase deflections (by up to 50%) or reduce stiffness at mass losses of steel between 4 and 7%. At larger mass losses of steel, deflections remained constant despite a continued increase in the level of steel corrosion. Interestingly, the influence of steel corrosion on stiffness was found to be independent of the level of the sustained load. However, it was pointed out in Chapters Two and Three that more understanding on the behaviour of RC members that are corroded under load requires an in-depth assessment of the variation of longitudinal strains along as well as across member sections.

If stiffness is reduced by corrosion of steel then repairs (which if properly carried out, stop further steel corrosion, replace damaged concrete and compensate the lost steel) are expected to increase the stiffness of a corroded structural member. Changes in stiffness can therefore be used to measure effectiveness of repairs. Unfortunately, a review of previous work in Chapter Two did not find literature on RC specimens that were corroded and repaired under load. An improved test set-up to allow for this was outlined in Chapter Three.

5.2 Objectives of the chapter

This chapter presents results and a detailed discussion on the variation of longitudinal strains on RC beams that were corroded and repaired with patch mortars and FRPs under sustained service loads. It is firstly aimed at examining the potential of using the variation of longitudinal strains and their derivatives such as stiffness and the depth of the neutral axis of corroding RC structures as measurable parameters to indicate the level of steel corrosion. The chapter then discusses the potential of using them to measure the effectiveness of patch and
FRP repairs under sustained service loads to increase the service life of corroded RC structures.

5.3 Results on longitudinal strains during the accelerated corrosion phase

Before discussing results on longitudinal strains measured during the corrosion process, it is important to recall the measuring points of longitudinal strains on beams and the symbols that were used for their identification that were discussed in Chapter Three. To help the reader, Figure 3.9 is shown again here, but only with positions on a beam where longitudinal strains were measured. As shown in the figure, longitudinal strains were measured at four different positions across the depth of a beam that corresponded to the corroded region or were directly beneath it (bearing in mind that beams were inverted with the tensile corroding face on top). Strain measurements were limited to within 50 mm of the ends of the corrosion region. Each monitoring position was partitioned into six different 100 mm long sections.

5.3.1 Longitudinal strains on a non-corroded beam

Figure 5.1 shows the variation of longitudinal strains at various positions on beam 3. It was not corroded but tested under a sustained load equivalent to 12% of its load-bearing capacity. From Chapter Three, a load of 12% was such that a beam had flexural cracks. From Figure 5.1a, longitudinal strains on the tensile face of the beam (lt) had two distinct groups. The first group composed of strains which were measured 100, 400 and 600 mm (lt-1, lt-4 and lt-6, respectively) from the current input side of the corrosion region. They exhibited tensile strains of about 400 micro strains. As expected from the applied load, their magnitude (> 300 micro strains) indicates that the beam was cracked. Longitudinal strains in the second group were nearly zero. These strains are an indication of variation of curvatures along the beam; at the location of a flexural crack (group 1) and where the beam was uncracked (group 2). As expected from creep of RC members under load, longitudinal strains at cracked sections increased by about 100 micro strains (up to 33%) over the 70-day testing period.
Designation: \( \text{lt} = \) longitudinal strains at the tensile face; \( \text{l2} = \) longitudinal strains on the side face measured 30 mm from the tensile face; \( \text{l3} = \) longitudinal strains on the side face measured 100 mm from the compression face; and \( \text{lc} = \) longitudinal strains on the side face measured 30 mm from the compression face.

**Figure 3.9** Location of targets for strain measurements
Figure 5.1 Variation of longitudinal strains with time in beam 3

Strains measured 30 mm from the tensile face of the beam (Figure 5.1b) exhibited a similar variation of longitudinal strains to those measured on the tensile face. As expected from the normal variation of strains in a RC beam in bending, strains that were measured 30 mm from the tensile face were lower than those on the tensile face. This indicates uniformity in the variation of strains. Longitudinal strains measured 30 mm from the compression face (lc) were clearly in compression. However, despite being largely in tension, some of the strains...
measured 100 mm from the compression face (l3) were in compression. Those clearly in tension (l3-4 and l3-6) were opposite sections where large tensile strains were recorded on the tensile face. This implies that at cracked sections, the depth of the neutral axis was between the position of lc-strains and l3-strains (30 to 100 mm from the compression face). At uncracked sections, the depth of the neutral axis was between the position of l3-strains and l2-strains (above 100 mm from the compression face). In consent with these results, the theoretical depth of the neutral axis at uncracked sections was 129.77 mm from the compression face. At cracked sections it was 54.53 mm from the compression face.

5.3.2 Longitudinal strains on a corroding RC beam

Figure 5.2 shows longitudinal strains on beam 15 during the accelerated corrosion phase. From Figure 3.1, accelerated corrosion in beam 15 was carried out (for 64 days) under a sustained load equivalent to 8% of the load-bearing capacity of a non-corroded beam. As pointed out in Chapter Three, this load was too small to induce flexural cracks on the beam. During the first eight days of testing, the figure shows expected variation of longitudinal strains. Strains on the tensile face where largely in tension but less than corresponding strains recorded in beam 3, which was tested under a larger sustained load. After eight days of testing, very different strains to those observed in beam 3 were recorded. A large tensile strain (reaching 1600 micro strains) which monotonically increased at a decreasing rate was recorded near the end of the corrosion region (lt-6) on beam 15. Corresponding large compression strains near the compression face (up to five times larger than those in beam 3) were also found. Similar to strains on the tensile face, strains near the tensile face (l2) also had selected sections that exhibited unexpectedly large tensile strains (l2-5). Contrary to a non-corroded beam, a section that exhibited large tensile strains on the tensile face (lt-6) exhibited low strains on a corresponding position that was 30 mm below the tensile face (l2-6). From a structural mechanics viewpoint, one is likely to attribute the large tensile strains at lt-6 in beam 15 to a corresponding large reduction in the area of steel at that section. In contrast, it was found in Chapter Four that the largest level of steel corrosion was at the centre of the corrosion region of beams. The largest tensile strains in beam 15 (lt-6 and l2-5) were located
Figure 5.2 Variation of longitudinal strains with time in beam 15 at the end of the corrosion region where the level of steel corrosion was minimal. This shows that even though some longitudinal strains recorded on the tensile face in beam 15 steadily increased with the level of steel corrosion, it is difficult to relate the rate of increase of the strains with the level of steel corrosion.

Another phenomenon that could be ascribed to the large strains in beam 15 compared to beam 3 is the stresses from the corrosion process. Regrettably, corrosion stresses are nearly orthogonal to the longitudinal axis of a bar. Strains

NB: Results on beam 15 here are during accelerated corrosion at a load of 8 %
due to them are therefore expected to be vertical or parallel to RC members depending on where they are measured.

The large strains in corroded beams can be attributed to cracking of the cover concrete. Cracking of the cover concrete due to steel corrosion will be discussed in detail in Chapter Six. However, to help explain the unexpected distribution of longitudinal strains in corroded beams, a typical pattern of corrosion cracks is shown in Figure 5.3. Note that near the end of the corrosion region (where large longitudinal tensile strains were recorded), the main longitudinal corrosion crack divided into two and the new cracks diverged to the side faces of the beam. It was pointed out in Chapter Four that delamination of the cover concrete during loading was because between two adjacent corrosion cracks, the cover concrete was detached from the parent concrete beam. This implies that section A-B-C around the corroded region and bound by the crack splits in Figure 5.3 was not attached to the parent concrete. As expected, stresses from the corrosion of embedded steel bars pushed it upwards along the planes of the crack splits. If the section of the cover concrete A-B-C was fixed along line B-C then rather that deforming in the transverse direction (which will result in widening of corrosion cracks) it certainly deformed upwards about line B-C. It will be shown in Chapter Six that corroded beams exhibited crack widths of up to 0.5 mm at the ends of the corrosion region. If at point A, the cover concrete ABC deflects upwards by 0.5 mm, it can be shown that measuring longitudinal strains across A will give strains of about 1250 micro strains. These are near-equal to those recorded in beam 15.

A side view of the longitudinal section of the beam along line D-D can be represented schematically using Figure 5.4. The fixity of the cover concrete in the figure suggests that at line B-C, stresses from expansive corrosion products induced a hogging longitudinal moment on the parent beam. From structural mechanics, this would result in large longitudinal compression strains on the compression face. It will be shown in Chapter Six that the fixity on other cracked sections of the cover concrete (such as D-A-C) resulted in transverse moments being induced on the parent beam. Therefore, they had little effect on the longitudinal strains but significantly widened corrosion cracks. It is logical that during the development stage of the crack pattern, corrosion products were not
Figure 5.3 Typical corrosion crack pattern on tensile face of RC beam

Figure 5.4 Side view of section D-D
freely discharged so that there was significant pressure build-up around the corroding area. A large rate of increase of longitudinal strains was therefore expected. When well-cracked, corrosion products were freely exuded to the exterior surfaces of the concrete. In agreement with Figure 5.2, a reduced rate of increase of longitudinal tensile strains was observed.

It is worth mentioning that the failure mode of beam 4 (shown in Figure 4.11) indicated that in some corroded beams, only part of the cover concrete was detached from the parent beam due to corrosion cracks. Those beams exhibited discrete corrosion cracks on the side faces as opposed to a continuous longitudinal crack as on the tensile faces of beams shown in Figure 5.3. In addition to cracks in Figure 5.3, in sections where the cover concrete was detached by a corrosion crack on the tensile face and another on the side face, strains from expansive corrosion products deformed the cover concrete mainly in the longitudinal direction. At the ends of the discrete cracks on the side faces, the cover concrete was attached to the parent beam. Deformation on those sections was expected in the transverse direction. This will be confirmed in Chapter Six using the rate of widening of corrosion cracks. In consent with Figures 5.2a and b, a change in the variation of the fixity of the cover concrete was expected to result in a non-uniform deformation of the tensile face of beams. This is shown schematically in Figure 5.5 using a side view of a corroding beam. Figure 5.6 shows a photograph of the profile of the tensile face of a corroded beam indicating clearly, depressions and rises on the tensile face. Note from the figure that rises on the tensile face were opposite sections on the side faces of beams that were stained with corrosion products hence indicating the location of discrete corrosion cracks.

From the above-discussion, the variation of longitudinal strains on the cover concrete of a corroding RC beam is primarily dependent on crack patterns. It was shown in Chapter Two, using results from Ballim et al. [7,8], El Maaddawy et al. [9,10] and Zhang et al. [5] that crack patterns are independent of the level of the sustained load. Longitudinal strains on the cover concrete are therefore also expected to be independent of the level of the sustained load. This was confirmed using longitudinal strains in beam 4 which was corroded in the absence of a sustained load (Figure 5.7).
Figure 5.5  Side view of a corroding beam with discrete corrosion cracks

Figure 5.6  Change of profile on the tensile face of a corroding beam
It is evident that despite being corroded in the absence of a sustained load, the beam exhibited significant longitudinal tensile strains on the cover concrete. Interestingly, these strains were very similar to the majority of longitudinal tensile strains on beam 15. They were also significantly larger (up to three times) than corresponding longitudinal strains on the control beam (beam 3) that was non-corroded but tested under a sustained load of 12%. Contrary to beams 3 and 15, in beam 4, longitudinal strains near the compression face (lc) varied around zero. Here the beam was uniformly supported at the compression face so that little curvature, especially around the compression region was expected.

The variation of strains at each position across the depth of a beam may help understand mechanics of deformation of cover concrete due to steel corrosion. However, in real structures it is not always possible to measure strains across the entire depth of a RC structural member. They are measured at limited positions and an assumption of uniformity of strains across the depth of a member is used to calculate strains at other positions.

5.3.3 Uniformity of longitudinal strains across the depth of a corroding beam

It was pointed out earlier that longitudinal strains on the tensile face of beam 15 (and other corroded beams) were generally larger than corresponding strains on other positions across the beam depth. However, there were sections of the beam (such as l2-5) that exhibited tensile strains that were much larger than corresponding strains measured on the tensile face (lt-5). This implies that some cross-sections of corroded beams did not show uniformity of longitudinal strains across the beam depth. The majority of models on the flexural behaviour of RC beams such as in [11,12] as well as other forms of interpreting experimental results of RC beams (such as stiffness) are however, principally based on this uniformity. It was therefore essential to assess it at the various measuring positions shown in Figure 3.9. This was done using Figure 5.8 which connects strains at each section along the tensile face with corresponding strains at other positions across the depth of a beam. For example, lt-1 strains were connected to l2-1, l3-1 and lc-1 strains and they are referred to as target 1. Beams shown in the
**Figure 5.7** Variation of longitudinal strains with time in beam 4

The graphs show the variation of longitudinal strains (in micro-strains) with time (in days) for three different positions:

- **a)** Tensile face
- **b)** 30 mm from the tensile face
- **c)** 30 mm from the compression face

**NB:** Beam 4 was corroded in the absence of a sustained load.

The graphs indicate the longitudinal strains over time for different locations within beam 4. The control beam (beam 3) and one corroded beam under load (beam 15) are also shown, highlighting the strain variations after 30 and 50 days of testing.
Also included in the figure is a connection of average strains from the six sections at a position (such as an average of \( lt \)-1 to \( lt \)-6 strains) with average strains at other positions across the depth of a beam (such as an average of \( lc \)-1 to \( lc \)-6 strains). Evidently, strains recorded at positions that were far from the corrosion region (\( l3 \) and near the compression face) showed good consistency for either beam. Expectedly, \( l3 \)-strains were smaller than \( lc \)-strains. In a non-corroded beam (beam 3), the majority of sections near the tensile face showed good consistency as well. There was however, a clear non-uniform variation of longitudinal strains near the corroded region in beam 15, especially after 50 days of testing (Figure 5.8d).

Figure 5.8 Longitudinal strains across beam depths in beams 3 and 15
A plot of average longitudinal strains at a position against other corresponding average strains showed good consistency. Since Figure 5.8 shows strains at discrete times during the testing period, it was important to assess if this consistency was satisfied through the entire testing period. This was done using Figure 5.9. It entailed calculating strains 30 mm from the tensile face (calc_l2) and 100 mm from the compression face (calc_l3) using measured average strains on the tensile face (lt) and average strains near the compression face (lc) on the beams. The assumption used was that average strains at each position linearly varied across the depth of beams. It is evident from the figure that during the accelerated corrosion process, calculated strains were close to measured strains at the various positions of the depth of beams. Therefore and in consent with Figure 5.8, Figure 5.9 suggests that average strains at each position were consistent with average strains at other positions. Most importantly, it indicates that average strains at two positions across the depth of a corroding beam can be used to determine other needed-derivatives of longitudinal strains such as curvatures, the depth of the neutral axis and stiffness.

5.4 Results on depth of the neutral axis of a corroding RC beam

The depth of the neutral axis of beams during testing, calculated using average longitudinal strains on the tensile face and longitudinal strains measured 30 mm from the compression face (equation 5.1) is shown in Figure 5.10. In the figure, the depth of the neutral axis was measured from the compression face which is represented by an ordinate of zero. Note that equation 5.1 uses absolute values of compression strains as opposed to negative values shown in Figure 5.9.

\[ x = \frac{|\varepsilon_{lc}| h + 30|\varepsilon_{lc}|}{|\varepsilon_{lc}| + \varepsilon_{lt}} \]  

(5.1)

Where; \( x \) = depth of the neutral axis measured from the compression face; \( \varepsilon_{lc} \) = average longitudinal strains measured 30 mm from the compression face; \( \varepsilon_{lt} \) = average longitudinal strains measured on the tensile face; and \( h \) = height of the beam (254 mm).
The figure shows that the depth of the neutral axis of the control beam (beam 3) was near-constant during the testing period. For corroded beams, there was a wide variation of the depth of the neutral axis but with a little noticeable net change. The majority of beams that were corroded under a load of 12%, except beam 20, had a depth of the neutral axis that was largely smaller than the depth of the
neutral axis on beams that were corroded under lower loads. Surprisingly, even during the early testing periods (when the influence of corrosion was expected to be insignificant), beam 20 exhibited a depth of the neutral axis that was similar to those in beams corroded under a load of 8%. This is anomalous behaviour which can be attributed to the non-homogeneity of concrete.

Designation: UC = non-corroded; AC = accelerated corrosion with 2-day drying cycles; and a number after the two letters indicates the level of sustained load as a percentage of the load-bearing capacity of a control beam.

Figure 5.10 Variation of the depth of the neutral axis with time for various beams
It is important to mention that theoretically, the depth of the neutral axis, especially for a gross section, is not sensitive to changes in the level of steel corrosion. For example, at a mass loss of steel of 10% (as used in this research), theoretical depth of the neutral axis for a gross section measured from the compression face was 129.38 mm compared to 129.77 mm for a non-corroded beam. For a cracked section, a mass loss of steel of 10% was expected to reduce the depth of the neutral axis from 54.53 to 52.28 mm. Certainly these changes are too low to accurately measure. These values were calculated assuming external loads equivalent to 8% (uncracked) and 12% (cracked) of a load-bearing capacity of a non-corroded beam are applied to already-corroded beams.

The large longitudinal strains recorded on the tensile faces of corroded beams (Figures 5.2 and 5.7 to 5.9) were however, expected to have had some influence on the depth of the neutral axis. Regrettably, the figures also indicate that an increase in the longitudinal tensile strains was accompanied by a corresponding increase in the longitudinal compressive strains. From equation 5.1, a small change in the depth of the neutral axis was expected.

Another important feature to consider with corroded beams here is that the increase in longitudinal strains during testing was due to the corrosion process whilst the level of sustained load could only contribute with effects of creep. Increase of strains in beam 3 (100 micro strains) compared to 1600 micro strains in beam 15 (under a load of 8%) and 1200 micro strains in beam 4 (in the absence of a sustained load) indicate that the influence of creep was insignificant. Longitudinal tensile strains from the corrosion process were shown to be caused by a lack of full-fixity of the cover concrete to the parent concrete beam due to corrosion crack widths and patterns. However, Chapter Two indicated corrosion crack widths and patterns to be independent of the level of the sustained load. The result was that corroded beams, including a beam that was corroded in the absence of a sustained load, had largely identical longitudinal tensile strains. As previously mentioned, stresses induced on a concrete member by corrosion of embedded steel when an external load is kept constant are very different to those when the level of steel corrosion is kept constant and the load is increased. The former are localised and internal whilst the later are global. Furthermore, internal corrosion
stresses are directly applied on the concrete as opposed to external loads which are transferred to the concrete via the bond between the steel and the concrete. The area of steel (regardless of the level of the sustained load) therefore does not affect the transfer of corrosion stresses. The result is that continued loss in steel has little effect on the depth of the neutral axis.

This finding is fundamental to researchers and structural engineers who may need to assess accurately the structural integrity of a corroding structure so as to prescribe an appropriate type of intervention if needed. Without a detailed understanding of the effects of steel corrosion on the variation of the depth of the neutral axis, they are likely to conclude from the measurements of the depth of the neutral axis that the structure is sound. This is despite Chapter Four revealing levels of mass loss of steel up to 23% at the end of the accelerated corrosion phase. This research therefore discourages the measurement of the depth of the neutral axis as a technique that can be used to indicate the level of steel corrosion or indicate the structural integrity of corroded RC structures. Furthermore, it does not fulfil one of the primary objectives of this research which was to find measurable parameters of a corroding RC structural member that can be used by engineers to indicate its residual load-bearing capacity.

5.5 Results on stiffness of a corroding RC beam

Contrary to the depth of the neutral axis which is equally dependent on the variation of both the longitudinal strains on the tensile face and the corresponding strains on the compression face of a beam, curvatures are more dependent on the variation of longitudinal tensile strains [11,12]. If strains uniformly vary across the depth of a beam, as was shown for beams in this research using Figures 5.8 and 5.9, curvatures can be calculated using equation 5.2.

\[ \varphi = \frac{E_h}{h - x} \]  

(5.2)
Where; \( \phi \) = curvature; \( \varepsilon_{lt} \) = average longitudinal strains measured on the tensile face; \( h \) = height of the beam (254 mm); and \( x \) = depth of the neutral axis measured from the compression face.

Even though curvatures are an important parameter to measure in a structure, the stiffness of the structure \( (EI) \), which is inversely related to its curvature and incorporates the external applied moments, is a more practical parameter of the structure [12]. This section of the chapter therefore discusses the effects of steel corrosion on the variation of the stiffness of RC beams that were corroded under sustained service loads. They were calculated using equation 5.3.

\[
EI = \frac{M(h-x)}{\varepsilon_{lt}}
\] (5.3)

Where; \( EI \) = stiffness of a beam; \( M \) = external applied moment; \( x \) = depth of the neutral axis measured from the compression face; \( \varepsilon_{lt} \) = average longitudinal strains measured on the tensile face; and \( h \) = height of the beam (254 mm).

Figure 5.11 shows the variation of stiffness (from equation 5.3) of various beams that were tested under load with time. It also shows theoretical stiffness (fully-cracked and gross stiffness) calculated from the geometric properties of the beam and the applied moment. The stiffness for the fully-cracked concrete was calculated by incorporating the maximum mass loss of reinforcing steel bars of 12.1% for beams that were corroded under 2-day drying cycles from Chapter Four. For the first five days of corrosion, the figure shows that in beams 13 to 16 that were corroded under a sustained load of 8% (with no flexural cracks), the measured stiffness was about 8% larger than the theoretical gross stiffness. This can be attributed to errors in the theoretical model for gross stiffness but mostly, to errors in measurements of rather-low longitudinal strains from the low applied loads on the beams. As expected from the variation of the depth of the neutral axis, beam 20 also exhibited a large stiffness. Beam 3 and beams 17 to 19 which had flexural cracks showed expected stiffness that were between the gross and the fully-cracked stiffness.
Designation: UC = non-corroded; AC = accelerated corrosion with 2-day drying cycles; and a number after the two letters indicates the level of sustained load as a percentage of the load-bearing capacity of a control beam.

Figure 5.11  Variation of stiffness of corroding RC beams with time

During the entire testing period, the stiffness of beam 3 (non-corroded) was near-constant. As previously mentioned and as expected from the low applied loads, this implies that the effect of creep on beams was minimal. Interestingly, Figure 5.11 shows that there was a significant loss in stiffness of corroded beams during the first 20 days of testing. In fact after 20 days, the stiffness of some beams (beams 15, 16, 19 and 20) had reached their fully-cracked stiffness. Even more intriguing, beams 15 and 16 had no flexural cracks whilst beam 19 had flexural cracks.
Equation 5.3 shows that the only varying factor of stiffness during corrosion was the longitudinal strains on the tensile face of beams. They were however, shown earlier to be independent of the level of the sustained load. Expectedly, the stiffness was also independent of the level of the sustained load. Also as expected from the variation of longitudinal strains, from about 20 days of testing to the end of the accelerated corrosion phase, insignificant changes in stiffness were recorded. This was despite a continued impressed constant current hence an increase in the level of steel corrosion. It will be shown in Chapter Six that after 20 days of testing, the cover concrete was cracked due to stresses applied on the concrete by the expansive corrosion products. Most certainly, corrosion cracks allowed for a free discharge of corrosion products from the corroding area and hence relieved the corrosion pressure on the cover concrete. With a reduced pressure, further steel corrosion was not expected to cause major increments in the longitudinal strains (Figures 5.2, 5.7 and 5.9) and hence the stiffness of beams.

Chapter Four indicated that the maximum mass loss of steel in corroded beams shown in Figures 4.5 and 4.7 was between 1.1 and 1.68 g/day/m of bar length. Therefore, after 20 days of corrosion, mass loss of steel in beams was between 22 and 33.6 g/m of bar length. These mass losses convert to percentage losses of steel of 2.5 and 3.8%, respectively. They are lower than those found by Ballim et al. [7,8] (which were about 5%) and by Yoon et al. [6] (which were between 4 and 7%). This is probably because longitudinal strains here gave localised stiffness whilst deflections from Ballim et al. [7,8] and Yoon et al. [6] gave global stiffness.

A best-fit for the variation of stiffness of corroded beams with time is given by a power-function;

\[ EI = 70.42 * t^{-0.39} \quad (R^2=0.7) \]  \hspace{1cm} (5.4)

Where; \( EI \) = stiffness of corroded beams (kN-m^2); and \( t \) = time of testing (days).

Based on the rate of loss of steel from Chapter Four, equation 5.4 can be written as;
EI = (31.2 to 36.8) * \( Q^{0.39} \) \hspace{1cm} (5.5)

Where; \( Q \) = percentage mass loss of steel.

To verify results for the variation of stiffness of corroded RC beams in Figure 5.11, central deflections on beams during the corrosion process were also measured and are shown in Figure 5.12. As expected, instantaneous deflections on beams 3, 17 and 20 (average of 0.28 mm) which were tested under a load of 12% were larger than deflections on beams 14, 15 and 16 (average of 0.02 mm) which were tested under a load of 8%. After about 20 days of testing, deflections on beams that were corroded under a load of 8% (beams 14 to 16) exceeded the deflection of the control beam (beam 3) despite being tested under a higher load. This suggests that deflections were mainly controlled by the steel corrosion and not by creep effects. Similar results were found in [6-8] even for RC beams that were tested under sustained loads equivalent to 75% of the ultimate capacity of the uncorroded beams as discussed in Chapter Two. It is also interesting to mention that during the corrosion process, deflections on beams that were corroded whilst under a load of 12% (beams 17 and 20) were consistently about 1.5 times larger than deflections on beams that were corroded whilst under a load of 8% (beams 14 to 16). This indicates that the gradient of the deflection curves, and hence the stiffness of the various beams was about the same through the entire accelerated corrosion phase. Similar to the variation of longitudinal strains during the corrosion process, Figure 5.12 shows that deflections increased monotonically with time of testing but at a decreasing rate. Very little changes in deflections were recorded after 40 to 50 days of testing.

Based on the loading arrangements and the level of loading on beams, the gradient of the curves of the measured deflections in Figure 5.12 were used to calculate the global stiffness of the beams and is shown in Figure 5.13 (equation 5.6).

\[
EI = \frac{M}{24\Delta} \left(3l^2 - 4l_i^2\right) \hspace{1cm} (5.6)
\]
Desination: UC = non-corroded; AC = accelerated corrosion with 2-day drying cycles; and a number after the two letters indicates the level of sustained load as a percentage of the load-bearing capacity of a control beam.

Figure 5.12 Variation of deflections of corroding RC beams

Where; $EI =$ stiffness of corroded beams (kN-m$^2$); $M =$external applied moment; $\Delta =$ measured deflection; $l =$ beam span (2600 mm); and $l_s =$ shear span (800 mm).
Designation: UC = non-corroded; AC = accelerated corrosion with 2-day drying cycles; and a number after the two letters indicates the level of sustained load as a percentage of the load-bearing capacity of a control beam.

Interestingly, the stiffness of beams based on deflections had a lower rate of decrease compared to the stiffness of beams which was derived from the variation of longitudinal strains (Figure 5.11). This is understandable as in this research, corrosion was concentrated to the middle part of the beams so that outside the corrosion region, the steel bars remained fully-anchored to the concrete. The global stiffness calculated from deflections was therefore, a mean stiffness between the gross stiffness outside the corrosion region and the reduced stiffness within the corrosion region. In addition, the end-supports from the test rig were not pure-pin supports. They therefore provided some moment resistance which
could have reduced the beam deflections. For example, the expected instantaneous deflection of beams under a load of 12% was 0.38 mm whilst the average measured deflection was 0.28 mm.

After 40 to 50 days of corrosion, the global stiffness from beam deflections ceased to increase despite a continued impressed current. Stiffness from deflections was about 1.4 times larger than the fully-cracked stiffness. Similarly, instantaneous deflections in Figure 5.12 were about 1.4 times the theoretical deflections. This suggests that if the frame did not provide some moment resistance, the stiffness after 40 to 50 days could have been equal to the fully-cracked stiffness. From the variation of mass loss of steel in Chapter Four, after 40 days, mass loss of steel was between 5 and 8%. This critical mass loss of steel is similar to that obtained by Ballim et al. [7,8] and Yoon et al. [6].

5.6 Results on longitudinal strains and stiffness during natural corrosion

Results presented above are when corrosion was accelerated by impressing an anodic current and the sustained load was kept constant. In real structures, steel corrosion is natural and applied loads often change with time. To simulate this, after accelerated corrosion, beams 16 and 20 were firstly allowed to corrode naturally before repairs. As mentioned in Chapter Three, during this natural corrosion phase, the load on beam 16, which was initially at 8%, was increased to 12% whilst the load on beam 20, which was initially at 12%, was reduced to 8%. The changes in the load levels were intended to simulate real situations where in the worst case, no action is taken after steel corrosion and yet there is a corresponding increase in the level of the load that is imposed on the structure (beam 16), or the load on a structure is reduced after noticing corrosion damage (beam 20).

Figure 5.14 a,b shows the variation of average longitudinal strains on beams 16 and 20. Compared to strains observed during the accelerated corrosion phase, little changes in longitudinal strains were observed when the sustained load in beam 16 was increased. Note that when accelerated corrosion was stopped, longitudinal strains in beam 16 had already stabilised (at about 50 days).
suggests that after about 50 days of accelerated corrosion, as well as during natural steel corrosion, stresses from steel corrosion no longer deformed the cover concrete in the longitudinal direction. It will be shown in Chapter Six that after 50 days of testing, corrosion cracks were developed on the side faces of the beam and near the end of the corrosion region. They therefore most probably allowed free discharge of corrosion products and hence reduced pressure applied on the cover concrete. As expected from the variation of longitudinal strains, there was little change in the stiffness of the beam during natural steel corrosion (Figure 5.14c).

Near-similar results were observed in beam 20. Reduction in the applied load had little effect on the stiffness. Contrary to beam 16 and other beams, beam 20 exhibited a reduction in longitudinal strains on the tensile region during the accelerated corrosion phase (after about 50 days of testing) as well as during natural corrosion phase. It exhibited a dominant longitudinal crack on one side face, small cracks on the tensile face and discrete cracks on the other side face. It will be shown in Chapter Six that the time when longitudinal strains decreased (after 50 days) coincided with the appearance of the discrete cracks. This further emphasises the notion that longitudinal strains on corroded RC beams are primarily dependent on corrosion crack patterns. It again shows that it is difficult to use longitudinal strains and its derivatives to indicate the level of steel corrosion.

5.7 Discussion on longitudinal strains and stiffness of corroded beams

As previously mentioned, large longitudinal strains in corroded beams can be attributed to cracking of the cover concrete. This was confirmed by comparing results from a beam that was corroded in the absence of a sustained load (beam 4) with a non-corroded beam tested under a load of 12% (beam 3). Note that when steel corrosion was started, RC beams tested under load already had strains from the external applied loads. As expected from normal behaviour of flexural-cracked RC beams (and shown from beam 3), longitudinal strains on the tensile face were larger than absolute longitudinal strains on the compression face (Figure 5.15b). Stresses and strains from steel corrosion due to deformation of the cover concrete
Designation: NC = natural corrosion; AC = accelerated corrosion with 2-day drying cycles; and a number after the two letters indicates the level of sustained load as a percentage of the load-bearing capacity of a control beam.

Figure 5.14 Longitudinal strains and stiffness during natural corrosion
were internal and directly applied on the concrete. They were expected to be more balanced across the depth of a beam and have a zero value at the gross position of the neutral axis (Figure 5.15c). The net strains, in agreement with Figure 5.9, were therefore as shown in Figure 5.15d. Since strains due to steel corrosion were much larger than strains from external loads, they controlled the net strains as well as the position of the neutral axis. As found in Figure 5.10, there was little change on the depth of the neutral axis of beams with continued steel corrosion. The implication to an engineer is that the depth of the neutral axis cannot be used to indicate the level of steel corrosion. In fact it might lead to some engineers incorrectly concluding that since it doesn’t change then the structure is sound.

Designation: $\varepsilon_{c\text{\_load}} =$ longitudinal strains on the compression face due to load; $\varepsilon_{c\text{\_corrosion}} =$ longitudinal strains on the compression face due to steel corrosion; $\varepsilon_{c\text{\_net}} =$ net longitudinal strains on the compression face; $\varepsilon_{lt\text{\_load}} =$ longitudinal strains on the tensile face due to load; $\varepsilon_{lt\text{\_corrosion}} =$ longitudinal strains on the tensile face due to steel corrosion; $\varepsilon_{lt\text{\_net}} =$ net longitudinal strains on the tensile face; $x_{\text{load}} =$ depth of the neutral axis due to load; $x_{\text{corrosion}} =$ depth of the neutral axis due to steel corrosion; $x_{\text{net}} =$ net depth of the neutral axis.

Figure 5.15  Net strains on a RC beam due to applied load and steel corrosion
The variation of strains shown in Figure 5.15 is for average strains over the corroded region and across the depth of a beam. At location of corrosion cracks on the tensile face of beams that split and diverged to the side faces of beams (Figure 5.3), compatibility of strains along the tensile face was lost. This resulted in a non-uniform variation of strains across the depth of a beam (Figure 5.8). In addition, discrete cracks on the side faces resulted in a non-uniform profile of the tensile face. At the location of these cracks, rises on the tensile faces of beams were observed (Figure 5.6). At the ends of the discrete cracks, depressions on the tensile face were observed. The result was that longitudinal strains varied along the tensile face with larger strains at the rises. Regrettably, the location of large longitudinal strains on the tensile face did not coincide with the location of maximum mass losses of steel. This again points out to the difficulty of trying to use longitudinal strains to indicate the level of steel corrosion.

Contrary to the depth of the neutral axis which remained constant through the entire testing time, there was a noticeable loss in stiffness (from longitudinal strains) at mass losses of steel between 2.5 and 3.8%. It will be shown in Chapter Six that at these mass losses of steel, corrosion crack widths were between 0.2 and 0.4 mm. According to DuraCrete Final Technical Report [13], a corrosion crack width of 0.3 mm (which is within the range of crack widths above) represents the lower limit for end of service life of corroding RC structures. This implies that engineers can use the variation of stiffness shown in Figure 5.11 (represented by equation 5.5) to indicate when repairs on a structure should be carried out. It is also important to note (as was discussed in Chapter Two) that corrosion cracks can only be seen when they are of certain widths (0.05 to 0.1 mm if to be seen with a naked eye). Therefore, as a measurable parameter of a corroding RC structure, stiffness is a better indicator of the start of the corrosion propagation period compared to concrete cover cracking. It is worth noting that it took only 20 days for the stiffness of corroded beams in this research to reduce from their gross stiffness to a fully-cracked stiffness due to accelerated steel corrosion. However, for in-service structures where the rate of corrosion is much lower, this period can last tens of years [5]. This emphasises the earlier notion that it might be of value to use the change in stiffness of beams due to corrosion as an indicator of the level of steel corrosion during the early corrosion stages.
According to Chapter Four, a mass loss of steel of 1% corresponded to a loss of load-bearing capacity of 0.8%. Therefore, at mass losses of steel where stiffness ceased to change (2.5 and 3.8%), the loss in load-bearing capacity of beams was between 2 and 3%. Owing to the large safety factors that are often used in design for structural strength (because of the severe consequences of structural collapse), this loss on the load-bearing capacity is insignificant. Furthermore, Figure 5.11 shows that after 20 days of testing (despite continued impressed current), there was little noticeable change in stiffness. Therefore, measuring stiffness of heavily corroded beams is of little value and should be avoided.

Another needed-discussion is on changes in longitudinal strains and stiffness of beams during natural steel corrosion. At the start of natural steel corrosion phase when a load applied on beam 16 was increased from 8% (no flexural cracks) to 12% (with flexural cracks), the beam already exhibited extensive corrosion cracks (≈ 1 mm). The variation of stiffness (Figure 5.11) indicates that it was behaving like a fully-cracked RC beam. In a non-corroded beam (beam 3), an increase in the sustained load from 8 to 12% induced flexural cracks and hence a corresponding increase in longitudinal strains. In this case, all tensile stresses from the increased load were essentially carried by the steel. From the normal variation of strains in a fully-cracked RC beam in flexure, low compression strains were expected. The result was that little changes in strains measured on the concrete were observed. At much larger loads, strains will still be recorded on the tensile face of beams but little strains will be expected on the compression face. The behaviour found in beam 16 is similar to that observed by Zhang et al. [5] at corrosion levels above 40% (Chapter Two).

Results on stiffness for beams 16 and 20 during natural steel corrosion clearly show that once the flexural stiffness of a corroded RC beam has reached its fully-cracked state, the level of steel corrosion cannot be detected by measuring changes in the bending stiffness by means of exciting the beam using additional external loads. From another viewpoint, stiffness remaining constant despite an increased load might be interpreted as a sign of a relatively sound structure. It was however shown in Chapter Four that the rate of steel corrosion during the natural corrosion process was significantly large (up to 66.5 µA/cm²) and had more
pitting corrosion. This indicates that there was a significant and yet unnoticeable (using stiffness) reduction in the load-bearing capacity of the structure. These results underline the earlier notion that load tests for stiffness on heavily corroded RC structures are of little value.

5.8 Results on longitudinal strains and stiffness after patch repairs

As mentioned in Chapter Three, after the accelerated corrosion process, selected beams were patch repaired. Subsequently, loads on repaired beams were increased to assess the performance of repairs. It is important to mention that since repairs were carried out under load, RC beams had residual stresses from applied loads as well as the corrosion process. Figure 5.16 shows the variation of longitudinal strains as well as the corresponding stiffness of beams 14 and 18 after patch repairs. A similar behaviour was observed in other beams. It is clear from the figure that within the first 10 to 14 days of repairs, a significant loss (up to 40%) in longitudinal strains on the tensile face (within repairs) was recorded. Strains then reduced at a much lower rate. Certainly, the reduction of longitudinal strains on the tensile face was due to compressive longitudinal strains from shrinkage of patch repairs. Since shrinkage reduces as wet concrete hardens, very little changes in strains were observed after 10 to 14 days. Interestingly, little changes in longitudinal strains were observed near the compression face. The result was that up to 50% gain in stiffness was recorded (from about 20 kN-m to 30 kN-m). When the applied load was increased from 8 to 12% in beam 18, a slight increase in longitudinal strains on the tensile face was recorded. The strains then reduced and became constant. Flexural cracks on the patch with a spacing of about 100 mm were clearly visible after the load increment. This implies that the patch was carrying tensile stresses from the applied load. The slight reduction in strains can therefore be attributed to redistribution of stresses. These results agree with the failure mode of patch repaired beams discussed in Chapter Four.

Another notable observation on Figure 5.16 is that corrosion strains on beams during the accelerated corrosion phase were much larger than corresponding strains during the curing period of patch repairs. Therefore, the loss in stiffness due to steel corrosion was much more pronounced than its ‘gain’ due to shrinkage
Designation: AC = accelerated corrosion with 2-day drying cycles; PR = patch repair; and a number after the two letters indicates the level of sustained load as a percentage of the load-bearing capacity of a control beam.

Figure 5.16 Longitudinal strains and stiffness of beams after patch repairs
strains. For example, the stiffness in beam 14 was reduced by 70% during the accelerated corrosion phase and only about 15% of the initial stiffness was recovered during the curing of the patch repairs. This clearly shows that the loss in stiffness due to steel corrosion on RC specimens under service loads is difficult to recover when structures are subsequently repaired still under load. Other techniques of reversing the distribution of corrosion induced stresses during repairs of corroded RC structures such as prestressing are therefore needed if it is desirable.

5.9 Results on longitudinal strains and stiffness after FRP repairs

Figure 5.17 shows results on longitudinal strains and stiffness of beams 16 and 20 that were repaired with FRPs but without patch repairs. As outlined in Chapter Three, FRP repairs were carried out at a load of 8%. After repairs, the load was increased to 12% and then specific to beam 16 and 20, it was increased further to 16%. During curing of FRPs, a slight reduction of longitudinal strains on the tensile face of beams was recorded. As a result, there was a slight increase in stiffness. Similar to behaviour of beams 14 and 18 (Figure 5.16), this can be attributed to shrinkage strains from the epoxy used to bond FRPs to the substrate concrete. Interestingly, when the load was increased from 8 to 12%, longitudinal strains on the tensile faces of beams increased significantly (from 300 to 420 micro strains for beam 16). However, little increase in strains was recorded at other positions (including 30 mm from the tensile face). As a result, there was little change in stiffness. When the load was further increased to 16%, a larger increase in longitudinal strains on the tensile face of beams was recorded. Again, little changes on strains near the compression face were recorded. This can be attributed to cracking of the cover concrete (to be discussed later).

Figure 5.18 shows that a different behaviour was observed when a corroded RC beam was firstly patch repaired prior to repair with a FRP plate. When the applied load was increased from 8 to 12%, similar to beams 16 and 20, longitudinal strains on the tensile face slightly increased. Rather than continuously increasing (as in beams 16 and 20), they became constant. The result was that stiffness of beams which was increased during curing of patch repairs became constant.
Designation: AC = accelerated corrosion with 2-day drying cycles; NC = natural corrosion; FP = FRP repair; and a number after the two letters indicates the level of sustained load as a percentage of the load-bearing capacity of a control beam.

Figure 5.17 Longitudinal strains and stiffness of beams after FRP repairs
Designation: AC = accelerated corrosion with 2-day drying cycles; PR = patch repair; FP = FRP repair; and a number after the two letters indicates the level of sustained load as a percentage of the load-bearing capacity of a control beam.

Figure 5.18 Longitudinal strains and stiffness after patch and FRP repairs
Furthermore, unlike in beam 18 where flexural cracks were observed when the applied load was increased after patch repairs, in beam 14, no flexural cracks were observed.

5.10 Discussion on longitudinal strains and stiffness of repaired beams

After patch repairs, corroded beams did not experience any longitudinal strains from further steel corrosion. This was confirmed with results from mass loss of steel in Chapter Four. Changes in longitudinal strains, especially during curing of patch mortars, were therefore only due to shrinkage of repair mortars. Note that stresses due to steel corrosion are nearly orthogonal to the longitudinal axis of a beam. They therefore apply longitudinal stresses only for particular crack patterns that were discussed earlier. On the contrary, stresses due to restraint of shrinkage are in the longitudinal axis of a beam. They therefore directly result in longitudinal strains on the substrate concrete. Since these stresses are axial they are (together with the strains) expected to be constant across the depth of a beam (Figure 5.19c). Note that patch repairs were carried out away from the position of the neutral axis. Therefore, additional stresses and strains due to eccentricity of shrinkage forces were expected. They linearly varied across the depth of a beam with compressive strains on the tensile face of the beam (near repairs) and tensile strains on the compression face (Figure 5.19d). It is clear from Figure 5.19 that on the tensile face of a repaired beam, axial strains from shrinkage and strains due to eccentricity were in the same direction (both compression). However, on the compression face they opposed each other. The result was that there was significant reduction of strains recorded on the tensile face with little change of strains on the compression face (Figure 5.16a,b).

As expected from the net strains in Figure 5.19, there was an increase in the stiffness of beams. This good performance of patch repairs was thwarted by flexural cracking of the repairs when the load was increased. In real structures, these cracks would allow ingress of corrosion agents which may result in further steel corrosion.
Designation: \( \varepsilon_{c_{\text{residual}}} = \) residual longitudinal strains on the compression face from load and steel corrosion; 
\( \varepsilon_{c_{\text{axial shrinkage}}} = \) longitudinal strains on the compression face due to axial shrinkage stresses; 
\( \varepsilon_{c_{\text{ecc shrinkage}}} = \) longitudinal strains on the compression face due to eccentricity of shrinkage stresses; 
\( \varepsilon_{lt_{\text{residual}}} = \) residual longitudinal strains on the tensile face from load and steel corrosion; 
\( \varepsilon_{lt_{\text{axial shrinkage}}} = \) longitudinal strains on the tensile face due to axial shrinkage stresses; 
\( \varepsilon_{lt_{\text{ecc shrinkage}}} = \) longitudinal strains on the tensile face due to eccentricity of shrinkage stresses; 
\( x_{\text{residual}} = \) depth of the neutral axis due to load and steel corrosion; 
\( x_{\text{net}} = \) net depth of the neutral axis.

Figure 5.19 Net strains on a patch-repaired RC beam

It is important to mention that this behaviour can only be observed when repairs are carried out under load. In its absence, Figure 5.7 (beam 4) indicated that there would be no residual strains on the compression face of a beam. Expectedly, eccentricity will not result in strains on the compression face either. The performance of patch repairs, shown in Figure 5.16, will therefore not be appreciated. However, cracking in patch repairs even at low loads was attributed to this redistribution of stresses and strains. In corroboration, Rio et al [14] reported flexural cracks on corroded then patch-repaired beams to appear at a higher load that on non-corroded beams. As discussed in Chapter Two, they corroded and repaired their beams in the absence of a sustained load. If engineers were to rely on results from patch repairs that were carried out in the absence of a
sustained load, they would certainly overestimate the performance of patch-repaired beams.

When corroded RC beams were repaired with FRP plates, shrinkage of the epoxy used to bond FRPs to the substrate concrete caused a similar response (reduction of strains on the tensile face) as shrinkage of repair mortars. More interesting results were found when the applied load was increased. It will be shown in Chapter Six that by bonding FRPs to the tensile face, the transverse stiffness of the cover concrete, especially the tensile face, was increased by the bond (from the epoxy) between FRPs and the concrete. As a result, continued corrosion reported in Chapter Four (Figure 4.7) resulted in new corrosion cracks developing on the side faces of beams. Clearly for those cracks to widen in the vertical direction (which they did), the cover concrete was pushed outwards from the parent concrete beam. Rather than causing small rises and depressions on the cover concrete as was the case with non-repaired beams (Figures 5.5 and 5.6), stresses from corrosion after repairs were uniformly resisted by the FRP plate and distributed over the entire beam. The result was an increase in longitudinal strains on the FRP plate (to allow vertical cracks to widen), but with little change of strains on the compression face. When FRP repairs were carried out subsequent to patch repairs, longitudinal strains on beams were minimised. The result was that stiffer beams were obtained.

It should be pointed out again that for corroded but non-repaired beams, increase in the applied load did not result in an increase in longitudinal strains on the tensile face of beams. In those beams, no new corrosion cracks were developed and the existing cracks primarily widened in the transverse direction.

5.11 Conclusions

This chapter presented results on longitudinal strains and stiffness on RC beams that were corroded and repaired with patch mortars and FRPs whilst under sustained service loads. A discussion of results and how they can be used to predict the level of steel corrosion as well as performance of repaired structures was presented. In the majority of the discussion points, the chapter strongly
discouraged the use of stiffness and longitudinal strains to measure performance of corroded and repaired RC beams. The following are specific conclusions to this chapter.

1. Stresses due to steel corrosion are well understood to be orthogonal to the longitudinal axis of a beam. It was however, shown that corrosion crack patterns often result in these stresses applying significant longitudinal strains in concrete. It was these stresses that controlled the flexural behaviour of RC beams. Since they depend on crack patterns, the variation of resulting strains at each section was not uniform. Moreover, their maximum did not coincide with the location of maximum steel loss. Average longitudinal strains measured over the entire corrosion region were however, found to uniformly vary across the depth of a beam. It was therefore recommended that monitoring corroding RC structures using the variation of longitudinal strains should avoid measuring strains at a point and rather measure average strains over the entire corroding region.

2. The variations of longitudinal strains during the corrosion process were such that there were unnoticeable changes in the depth of the neutral axis despite significant changes in the level of steel corrosion. Clearly, if in assessing a corroded structure using the variation of the depth of the neutral axis, the ‘practitioner’ is not fully aware of the behaviour of corroded RC structures, he/she is likely to believe from the variation of the depth of the neutral axis that the structure is free from damage or its functionality is not affected by damages that other indicators such as corrosion cracks might be showing. It was therefore recommended in the chapter that the depth of the neutral axis should be avoided as an indicator of the level of steel corrosion or as an indicator of the structural integrity of a corroded structure.

3. Flexural stiffness was generally found to decrease sharply at the early corrosion stages (2.5 to 3.8% mass loss of steel) and then become constant despite a continued increase in the level of steel corrosion. Since the sharp decrease in stiffness was even before cracking of cover concrete, it was concluded that stiffness could be used to indicate the level of steel corrosion
during the corrosion propagation period. However, the level of steel corrosion at which stiffness remained constant despite a continued increase in the level of corrosion was contended to be too low to indicate repairs of structures on the basis of the criterion of load-bearing capacity. It was therefore concluded that there is little value in measuring stiffness of heavily corroded RC structures as a test of its structural integrity. In corroboration, it was shown that changes in the level of the applied load (especially increasing it) on corroded RC structures had little effect on their stiffness. This could deceive practitioners into assuming that a corroding RC structure is structurally sound.

4. At an equivalent level of sustained load, larger longitudinal strains were recorded on corroded RC beams that were repaired with FRPs than on corroded but non-repaired beams. This was attributed to development of new cracks due to further steel corrosion of FRP-repaired beams. There was however, a little change in stiffness to indicate this further steel corrosion.

5. Shrinkage strains from patch repairs carried out under load were found to redistribute residual stresses in a corroded RC beam such that it appeared stiffer. A slight increase on the applied load (after hardening of repairs) however, caused cracking on the repairs. These cracks would allow more corrosion agents to reach the steel and hence further corrode. This implies that patch repairs, especially on structures where the load will subsequently be increased, are inadequate to increase the service life of a corroded structure.

6. In terms of performance of repaired structures, the above shortcomings were avoided when corroded RC beams were firstly repaired with a mortar followed by FRPs. Similar to conclusions made in Chapter Four, this implies that repairs of corroded structures should entail combined patch and FRP repairs.

5.12 References


CHAPTER SIX

COVER CRACKING OF RC BEAMS CORRODED AND REPAIRED UNDER LOAD

6.1 Introduction

Research has identified the primary cause of degradation of RC structures that are exposed to the marine environment as corrosion of reinforcing steel embedded in concrete. For academia and practitioners in the field of concrete structures, service life analysis of these structures has recently emerged as one of the focal points of research. The principal interest of majority of researchers has been to identify measurable parameters that can be accurately related with the level of steel corrosion. As already mentioned, the most common are the time to first cracking, corrosion crack widths and stiffness.

Chapter Five presented results and a discussion on longitudinal strains and stiffness of RC beams in this research that were corroded and repaired under load. It largely indicated them not to sufficiently vary with the level of steel corrosion for them to be used as ‘dependable’ measurable parameters. In few instances where they varied, their variation was either limited to low levels of steel corrosion (stiffness) or it did not relate to maximum steel loss (longitudinal strains). In repaired beams, they either also did not respond to repairs or their response was deceiving. For example, there was little change in stiffness after FRP repairs. On the other hand, patch repairs demonstrated an increase in stiffness which practitioners might see as ‘good performance’. However, they cracked extensively at a slight increase of the applied load. If the concern is on durability of repairs, this will aggravate asset managers as the cracks will allow more ingress of corrosion agents. Longitudinal strains and stiffness were only able to accurately indicate good performance of a corroded RC beam that had a combined patch and FRP repair. It was therefore recommended that they should be avoided as indicators of the level of steel corrosion as well as indicators of the structural integrity of heavily corroded RC structures even after repairs. Cracking of the
cover concrete therefore remains probably the most important measurable parameter of corroding RC structures that can be related to the level of steel corrosion.

As discussed in Chapter Two, extensive work has been done on time to cover cracking as well as corrosion crack widths. However, the chapter also criticised previous work for failing to accurately and consistently measure time to first cover cracking. Their results significantly varied and made it difficult to predict it. Despite that, analytical models were developed and calibrated by these results. Regrettably, they also proposed various criteria for cracking that were different to those used in experimental works. Expectedly, they failed to predict it.

Another setback with previous work on cracking of cover concrete which was discussed in Chapter Two is that majority of researchers often measured corrosion crack widths and mapped crack patterns at the end of corrosion tests [1-6]. Therefore, the sequence and propagation of corrosion cracks during the corrosion process was unknown. This is however, needed to relate the rate of widening of corrosion cracks with the rate of steel corrosion.

It was also pointed out in Chapter Two that most work on cover cracking was carried out in the absence of a sustained load. For the few where the effects of load on these parameters were looked at, they were limited to the corrosion phase. Performance of repairs, also under load, in terms of controlling further steel corrosion and increasing the load-bearing capacity of RC structures were not dealt with. As a result, much-needed parameters to assess performance of repairs carried out under load remain unknown. Research on this was outlined in Chapter Three.

6.2 Objectives of the chapter

This chapter presents results and a discussion on time of first cracking of the cover concrete. It then examines the potential of relating the rate of widening of corrosion cracks with the rate of steel corrosion. It also looks into the ability of repairs (carried out under load) to control cover cracking.
6.3 Results on time of first cracking of the cover concrete

Table 6.1 shows results on time of first appearance of visible corrosion cracks for RC beams in this research. Days reported in the table are only during wetting of beams when artificial current was impressed. The table shows that it ranged from 4 to 10 days with a mean of 7.2 days and a standard deviation of 0.5 days. From these results, it is difficult to associate time of first cracking with the level of sustained load. For example, in beams 18 and 19 (under 12% load) cracks were observed after five days whilst they were observed after four days in beams 4 to 6 (1% load). In addition, similar to beam 7 (4-day drying cycles), beams 9 to 11 (1% load and 2-day drying cycles) as well as beams 15 and 16 (8% load and 2-day drying cycles), cracks in beams 17 and 20 (12% load and 2-day drying cycles) were observed after eight days. It is therefore also difficult to associate time of first cover cracking with the duration of drying cycles. This is despite Chapter Four clearly indicating longer drying cycles to correspond with large corrosion rates. Note that beams 4 to 6 cracked during the first wetting cycle (prior to drying). In beam 7, certainly additional two days of drying before cover cracking compared to beams 8 to 20 cannot have sufficiently dried the corroding area to affect the time to cover cracking.

Another interesting finding in Table 6.1 is that it is also difficult to associate the compressive strength of concrete with the time of first cover cracking. This is confirmed using Figure 6.1 where the recorded times of cover cracking are plotted against the compressive strength of concrete. The low correlation coefficient (R^2 = 0.02) for data with a population of 17 is a sufficient indicator of no relation between compressive strength of concrete and time of first cover cracking. These results are in agreement with discussions in Chapter Two on results from Liu and Weyers [7] and Rasheeduzzafar et al. [8] as well as equations 2.3 to 2.8 from El Maaddawy and Soudki [9].
Table 6.1 Experimental results on time for first cover cracking

<table>
<thead>
<tr>
<th>Beam no.</th>
<th>$f'_{c}$, MPa (s.d.)</th>
<th>Sustained load as % of ultimate capacity during accelerated corrosion phase</th>
<th>Crack pattern</th>
<th>Time to cover cracking (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4*</td>
<td>46.5 (1.1)</td>
<td>0%</td>
<td></td>
<td>4</td>
</tr>
<tr>
<td>5*</td>
<td>35.0 (1.2)</td>
<td></td>
<td>B</td>
<td>4</td>
</tr>
<tr>
<td>6*</td>
<td>46.6 (1.1)</td>
<td></td>
<td></td>
<td>4</td>
</tr>
<tr>
<td>7*</td>
<td>46.6 (1.1)</td>
<td></td>
<td></td>
<td>4</td>
</tr>
<tr>
<td>8</td>
<td>40.2 (1.2)</td>
<td>1%</td>
<td>A</td>
<td>9</td>
</tr>
<tr>
<td>9</td>
<td>38.9 (1.4)</td>
<td></td>
<td>C</td>
<td>8</td>
</tr>
<tr>
<td>10</td>
<td>38.9 (1.4)</td>
<td></td>
<td></td>
<td>8</td>
</tr>
<tr>
<td>11</td>
<td>35.2 (0.8)</td>
<td></td>
<td>A</td>
<td>8</td>
</tr>
<tr>
<td>12</td>
<td>46.6 (1.4)</td>
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<td>10</td>
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<td>35.2 (0.8)</td>
<td></td>
<td>B</td>
<td>9</td>
</tr>
<tr>
<td>14</td>
<td>40.1 (1.3)</td>
<td>8%</td>
<td>B</td>
<td>9</td>
</tr>
<tr>
<td>15</td>
<td>40.1 (1.3)</td>
<td></td>
<td></td>
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<td>16</td>
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<td>17</td>
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<td>B</td>
<td>5</td>
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<tr>
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<td>40.0 (1.1)</td>
<td></td>
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<td>5</td>
</tr>
<tr>
<td>20</td>
<td>40.2 (1.2)</td>
<td></td>
<td></td>
<td>8</td>
</tr>
</tbody>
</table>

Average (s.d.) 7.2 (0.5)

* 4-day drying cycles

$f'_{c}$ = compressive strength of concrete

s.d. = standard deviation
It was pointed out in Chapter Two that the time of first cover cracking (similar to that indicated in Table 6.1) often records cracking only when cracks are wide enough to be seen with a naked eye. It may therefore overestimate the resistance of RC structures to cover cracking. In this research, this was avoided by monitoring transverse strains on tensile faces of beams and vertical strains on their side faces. To help the reader, the location of strain measurements is shown again here (Figure 3.9) but without the position of longitudinal strains which were discussed in Chapter Five. As shown in the figure, seven sections at spacings of 100 mm and limited to the corrosion region were monitored for transverse strains on tensile faces of beams. Vertical strains on side faces were also measured at seven sections that were directly opposite the measuring sections on tensile faces.
Current input side  Corrosion region = 700  Current output side

a) Tensile face

Current input

Tensile face

3 x 12 mm steel bars

30

50 100 100 100 100 100 100 100 50

Current input side  Corrosion region = 700  Current output side

b) Side face

Designation: tt = transverse strains on the tensile face; and ts = vertical strains on the side face

NB! All dimensions are in mm

Figure 3.9  Location of targets for strain measurements
6.4 Results on transverse and vertical strains before cover cracking

Figure 6.2 shows the average variation of transverse strains recorded on tensile faces and vertical strains on side faces of selected beams up to a strain of 1000 micro strains. This strain corresponds to a crack width of 0.1 mm which is small and yet clearly visible with a naked eye. Averaging strains at different positions was specifically done to enable a clear presentation of the results even though (as will be shown later in the chapter), there was a definite variation of strains along the corroded region.

It is clear from Figure 6.2 that prior to appearance of first visible corrosion cracks, compared to vertical strains on side faces, large transverse tensile strains were recorded on tensile faces of beams where corrosion agents were drawn. This is in consent with Figure 4.1 that losses in steel and hence the applied pressure from corrosion products was concentrated on internal surfaces of the cover concrete that were adjacent to the top surface of corroding bars (Figure 4.2). Bear in mind that during testing, beams were inverted with tensile face (where corrosion agents were introduced) at the top.

Figure 6.2 also shows that transverse strains were recorded even twelve hours from the start of the accelerated corrosion test. This implies that corrosion initiated just after impressing anodic current. This is expected as the current density applied (189 µA/cm²) was much larger that the threshold current density of 0.1 µA/cm² that indicates corrosion initiation [10]. Moreover, it indicates that after twelve hours, corrosion products were already applying pressure on the cover concrete. From Chapter Two, this means that the porous zone was fully-filled with corrosion products. Note that Figure 6.2a is for beams that were corroded under a load of 1% (without flexural cracks). Therefore, the level of impressed current density was such that the presence of flexural cracks (in Figure 6.2c) did not influence the corrosion initiation period. This is also confirmed by results on time of first cover cracking in Table 6.1. As mentioned in Chapter Two, for in-service structures where the current density is low, flexural cracks are expected to have a great influence of the corrosion initiation period.
Designation: tt = transverse strains on the tensile face; and ts = vertical strains on the side face

Figure 6.2 Transverse and vertical strains at the early testing stages
Another interesting feature from Figure 6.2 is that there was a bilinear increase of transverse strains on tensile faces of beams. It should be mentioned that the applied current density in this research was constant so that following Faraday’s Law, the rate of steel corrosion was also expected to be constant. It can be shown using equations 2.10 and 2.12 that the rate of increase of pressure applied on the surface of the cover concrete was also constant. If equations 2.10 and 2.11 are substituted into equation 2.12, then the resulting equation simplifies to equation 6.1.

$$\varepsilon_{tt} = P \frac{k}{E_c}$$

(6.1)

Where; $\varepsilon_{tt} =$ transverse strains on the surface of concrete; $P =$ internal pressure applied on the cover concrete by expansive corrosion products; and $k/E =$ stiffness of the cover concrete which is a function of the bar diameter ($d$), thickness of the porous zone ($t_p$), and cover depth ($c$); and $E_c =$ modulus of elasticity of concrete.

For a given RC member that is uncracked, $k$ (or thickness of the porous zone, bar diameter and cover depth) does not vary with age. On the other hand, for in-service structures which can take tens of years before cracking due to steel corrosion, the modulus of elasticity is expected to decrease. Therefore, an effective modulus of elasticity of concrete (such as given by equation 2.7) is often used rather than the 28-day modulus of elasticity. However, the testing periods for time of cover cracking in this research shown in Figure 6.2, were too small for the modulus of elasticity to substantially change. The initial increase in transverse strains shown in Figure 6.2 can therefore be attributed only to the pressure applied by corrosion products. Expectedly, since it increased at a constant rate, transverse strains also increased at a constant rate. Most importantly, for transverse strains shown in Figure 6.2 to suddenly increase, then from equation 6.1, either the internal pressure increased or the stiffness reduced. Certainly, the applied pressure from a constant rate of steel corrosion was not expected to suddenly increase. Cracking of the cover concrete is therefore, the only phenomenon that could have substantially reduced the stiffness. In this research, the point where the rate of
increase of transverse strains suddenly increased was taken as the point of cracking of the cover concrete.

From Figure 6.2 the strain that indicated cracking of the cover concrete ranged from 200 to about 400 micro strains. At these tensile strains, normal strength concrete is expected to be cracked. For the set-up in this research however, such a strain is equivalent to maximum crack widths between 0.02 and 0.04 mm which are too small to see with a naked eye. At the time of the first appearance of visible corrosion cracks (shown in Table 6.1), maximum crack widths were found to be about 0.08 mm (800 micro strains), which is about the smallest crack width that can be seen with a naked eye. This emphasises the earlier notion that recorded times in Table 6.1 as well as those discussed in Chapter Two were after corrosion cracks had occurred and then widened enough to be seen.

Chapter Three pointed out that in this research, strains were measured before and after each wetting cycle. Therefore, there is likelihood that strains that indicated first cracking shown in Figure 6.2 overlapped the two definite stages of cracked and uncracked cover concrete. In corroboration, Table 6.1 indicates that after the first wetting cycle, visible cracks on some beams had already appeared (those with time to cracking of less than four days). To overcome this hurdle, during the first two wetting cycles, transverse strains on the tensile face of beams 8 and 12 were monitored every 12 hours (Figure 6.2a). It is clear from Figure 6.2a that a change in the variation of transverse strains or cracking of the cover concrete occurred at a strain of about 200 micro strains.

As already stated, these results indicate that in calibrating models for the time of first cover cracking of concrete, the variation of strains applied on the concrete by the corrosion products (Figure 6.2) should be used instead of the time for the first appearance of visible corrosion cracks. In addition, they emphasise that losses of sections of bars due to the corrosion process when chlorides reach the steel from one face of the structure should not be taken as uniformly distributed around the bar, but rather as localised at the section of the bar that faces the direction of ingress of corrosion agents. The following is an analytical model for time of cover cracking that uses these concepts.
6.5 Description of model for cover cracking

6.5.1 The porous zone and diffusion of corrosion products

Concrete is a porous material and contains voids which corrosion products must first diffuse into before applying stresses on the cover concrete [7,9,11,12]. Regrettably, the non-homogeneity of concrete offers a difficult challenge to quantify these voids accurately. As a simplification of the problem (especially when modelling the time of cracking of the cover concrete), the voids in concrete are often represented by a porous zone around steel bars that has a uniform thickness ranging from 10 to 20 micrometers [7,9,11,12].

The assumption of the existence of the porous zone necessitates the relation between the expansion of the cover concrete and the loss in the area of steel during the period from the activation of the corrosion process to the first cracking of the cover concrete to be modelled in two definite stages. The first stage corresponds to the time required for corrosion products to completely fill the porous zone around the corroding steel bars. During this stage, corrosion products are assumed to diffuse into the porous zone without applying stresses on the cover concrete. It is logical that corrosion products, despite being produced only around the corroding section, will freely flow around the steel so that they completely fill the porous zone.

The second stage is when the porous zone has been fully-filled with corrosion products so that continued steel corrosion necessitates the surrounding concrete to expand so as to allow for deposit of new corrosion products. Unlike in stage 1, it is logical that with the porous zone fully-filled, corrosion products in stage 2 will be accumulated around the corroding area. It is important to note that contrary to partial surface steel corrosion described here, under uniform steel corrosion (described in Chapter Two), corrosion products are assumed to remain uniformly distributed around the surface of bars even after filling the porous zone.
6.5.2 Amount of corrosion to fill the porous zone

From Chapter Four (Figure 4.1), it is reasonable to assume that the remaining section of the corroded steel that faces the direction of ingress of corrosion agents will be elliptical shaped as shown in Figure 6.3. If the maximum radius of steel that must be lost to fill the porous zone is, \( \Delta r_p \), then the area of steel lost, \( \Delta A_{st,p} \), is given by equation 6.2.

\[
\Delta A_{st,p} = \frac{1}{2} \pi r \Delta r_p
\]

(6.2)

Where \( r \) is the radius of uncorroded steel bars.

Since corrosion products occupy a larger volume that the volume of steel lost, the corresponding volume of corrosion products to fill the porous zone, \( A_{cor,p} \), is given by equation 6.3.

\[
A_{cor,p} = \frac{n}{2} \pi r \Delta r_p
\]

(6.3)

Where \( n \) is the ratio of the volume of corrosion products deposited to the volume of steel lost.

Researchers have detected various corrosion products in corroding RC structures, all with different densities and volume expansion as shown in Figure 2.2 [13]. It is well documented that the type of each corrosion product is primarily dependent on the pH and the availability of oxygen [7,9,10,13]. These factors (pH and quantity of oxygen) are extremely variable and difficult to quantify in a corroding RC structure. As mentioned in Chapter Two, various researchers assume that for corrosion of steel that is embedded in concrete, ferrous hydroxide is the fundamental corrosion product [7,9,11,12]. However, with an increase in the supply of oxygen (especially after cracking of the cover concrete), more stable corrosion products such as haematite and magnetite are formed. Interestingly, the volume expansion of ferrous hydroxide is about 1.7 times larger than the volume expansion of the more stable compounds [7,10,13]. When modelling the time to
cover cracking of concrete due to steel corrosion, researchers have therefore found it convenient and conservative to use ferrous hydroxide as the primary corrosion product [7,9,14].

Designation: $P =$ pressure applied by corrosion products; $\Delta r_p =$ the maximum radius of steel that must be lost to fill the porous zone; $r =$ the radius of uncorroded steel bars; $t_p =$ thickness of the porous zone; $\mu_{cr} =$ the maximum radial expansion of the concrete that surrounds the corroding steel bars at the time of cracking of the cover concrete; $\Delta r_{cr} =$ the maximum radius of steel that must be lost for cover concrete to crack; and $c =$ cover diameter.

Figure 6.3 Partial surface corrosion of steel bars
Moreover, accelerated corrosion tests in laboratory RC specimens often involve full immersion of specimens [1,2,5,8] or cyclic wetting of the specimens [15,16] with salt water (as used in this research). It is logical therefore that prior to cracking of the cover concrete, the supply of oxygen to the corrosion region will be compromised so that the dominant corrosion product at that stage will be ferrous hydroxide. In corroboration, corrosion products observed around the steel when beam 11 was opened for repair just after cover cracking were greenish-black in colour indicating a large presence of ferrous hydroxide. In the contrary, reddish-brown products were found after testing other beams to failure indicating a large presence of the more stable corrosion products such as haematite.

As previously mentioned, during the process of filling the porous zone, corrosion products are expected to freely flow around the steel bars till the porous zone is fully-filled with the products. Assuming that the porous zone has a uniform thickness, \( t_p \), around the corroding steel as in [7,9,11,12], it can easily be shown from equations 6.2 and 6.3 and the resultant volume of the porous zone that to fill the zone with corrosion products, the maximum radius of steel lost, \( \Delta r_p \), is given by equation 6.4.

\[
\Delta r_p = \frac{2r_p(2r + t_p)}{r(n-1)} \tag{6.4}
\]

The corresponding level of steel corrosion (as a percentage mass loss of steel) necessary for corrosion products to apply stresses on the internal surfaces of the concrete can be calculated from equations 6.2 to 6.4. In addition, knowing the rate of steel corrosion, the time required to fill the porous zone (which some researchers such as [7,9,11,12] refer to as the free expansion period) can be determined.

6.5.3 Expansion of the cover concrete at the time of cracking

As discussed above, when the porous zone is fully-filled, additional corrosion products apply tensile stresses on the surrounding concrete and eventually cause cracking of the cover concrete. During this stage, the corrosion products are
expected to accumulate only around the corroding section of the steel and most importantly, to mirror the elliptical shape of loss in the section of steel as shown in Figure 6.3. Conversely, the expansion of concrete is also expected to copy the shape of the accumulation of the corrosion products. As previously discussed, this assumption is validated by the principal transverse strains recorded on the surface of the concrete that faces the direction of ingress of corrosion agents (Figure 6.2).

Let the maximum radial expansion of the concrete that surrounds the corroding steel bars at the time of cracking of the cover concrete be \( \mu_{cr} \) and the corresponding maximum radius of steel that is lost be \( \Delta r_{cr} \). Similar to equation 6.2, the area of steel lost at the time of cracking of the cover concrete \( (A_{st,cr}) \) is given by equation 6.5 whilst the corresponding volume of corrosion products deposited \( (A_{cor,cr}) \) is given by equation 6.6.

\[
\Delta A_{st,cr} = \frac{1}{2} \pi r \Delta r_{cr} \quad (6.5)
\]

\[
A_{cor,cr} = \frac{n}{2} \pi r \Delta r_{cr} \quad (6.6)
\]

From equations 6.5 and 6.6 and from Figure 6.3, it can be shown that the maximum expansion of concrete near the corroding area, \( \mu_{cr} \), necessary to accommodate the corrosion products, is given by equation 6.7.

\[
\mu_{cr} = \frac{(n-1)r\Delta r_{cr} - 2t_p(2r+t_p)}{r+t_p} \quad (6.7)
\]

It should be noted that equation 6.7 is valid only when the porous zone is fully-filled or if presented mathematically, when \((n-1)r\Delta r \geq 2t_p(2r+t_p)\).

From Figure 6.2, it is reasonable to assume that at the early corrosion stages, the rate of expansion of concrete was linearly related to the rate of steel corrosion. It is also reasonable to assume that the cover concrete behaves like a thick-walled cylinder with varying internal pressure around the surface of the corroding bars.
that is dependent on the level of steel corrosion. Under these assumptions, internal pressure is expected to be maximal on the surface of the corroding steel bars that faces the direction of ingress of corrosion agents. Conversely, little pressure is expected on the surfaces of the steel bars that are opposite the direction of ingress of corrosion agents (Figures 6.2 and 6.3). From basic mechanics and Figure 6.3, the theoretical maximum radial expansion of concrete near the corrosion products is therefore given by equation 6.8.

\[
\mu_{cr} = \frac{P(r + t_p)}{E_c} \left[ \left( r + t_p \right)^2 + \left( r + c + t_p \right)^2 \right] + \nu \left( r + c + t_p \right)^2 - \left( r + t_p \right)^2 + \nu \right] 
\]

Where \( P \) is the internal pressure applied on the inner surfaces of the cylinder by the expansive corrosion products; \( E_c \) is the modulus of elasticity of concrete; and \( \nu \) is the Poisson’s ratio of concrete.

The corresponding theoretical maximum transverse strain on the surface of the concrete, \( \varepsilon_{tt} \), is given by equation 6.9.

\[
\varepsilon_{tt} = \frac{2P \left( r + t_p \right)^2}{E_c \left( r + c + t_p \right)^2 - \left( r + t_p \right)^2} \]

Having developed the model, the following is a procedure for calculating the loss in the area of steel when knowing the external applied strains (which can easily be measured on the surface of the concrete).

1. Use equation 6.9 to calculate the ratio between the maximum internal pressure, \( P \), applied by the corrosion products and the modulus of elasticity of concrete, \( E_c \).
2. Use equation 6.8 to calculate the maximum radial expansion of the concrete near the corroding area, \( \mu_{cr} \).
3. The maximum loss in the radius of steel, \( \Delta r_{cr} \), can then be calculated from equation 6.7.
4. Finally, the loss in the area of steel can be calculated from equation 6.5.
6.5.4 Accuracy of the model

If equations 6.5 to 6.9 are used with the assumptions that; the average thickness of the porous zone is 15 micrometers as used in [9]; the ratio of volume of corrosion products to the volume of steel lost is 3.21 as used in [7,9,11,12,14]; the Poisson’s ratio of concrete is 0.18 as used in [7,9,11,12,14]; and cracking occurs when the strain on the exterior surface of concrete is 200 micro strains as shown in Figure 6.2, then the theoretical mass loss of steel at cracking was 0.39%. It was pointed out in Chapter Four that the rate of steel corrosion in beams was dependent on the dryness of the corroding area. Therefore, during the first wetting cycle, beams were expected to exhibit the lowest rate of steel corrosion which was found to be 1.1 g/day/m of bar length. At this rate of steel corrosion, it takes 3.1 days to reach a mass loss of steel of 0.39% which equations 6.5 to 6.9 found to cause cracking of the cover concrete. From Figure 6.2, 3.1 days is within the two to four days when measured transverse strains indicated cover cracking.

If however, uniform steel corrosion was assumed instead of partial surface steel corrosion, it can be shown that to obtain an external strain of 200 micro strains for beams that were used in this research, the level of steel corrosion must be 1.1%. Since to obtain a given strain on the external surface of a corroding RC structure, uniform steel corrosion requires a larger mass loss (about three times more) compared to partial surface steel corrosion, assuming uniform steel corrosion underestimates the maximum internal pressure applied by corrosion products under partial surface steel corrosion.

6.6 Discussion on time of first cracking of the cover concrete

As discussed in Chapter Three, RC beams in this research did not have shear stirrups within the corrosion region. The transverse stiffness of the cover concrete to resist cracking was therefore, only provided by the concrete. In the presence of shear stirrups, a larger stiffness is expected. In such instances, equations 6.8 and 6.9 in the presented model will need to be modified to include stiffness provided by shear stirrups. This model however, presents the worst scenario where steel corrosion occurs in regions of RC members that do not have shear stirrups.
When developing the model, it was assumed that steel bars corrode at the same rate. However, in in-service structures, natural steel corrosion often sacrifices some bars whilst others act as cathodes and do not extensively corrode. Certainly, the pressure applied by corrosion products will differ between bars. In such cases, it is important to measure strains across each bar and use the largest rate of increase of strains.

As already mentioned, this model, similar to other models discussed in Chapter Two \[7,9,11,12\], assumes ferrous hydroxide to be the dominant corrosion product. In in-service structures, concrete is expected to be drier (even prior to cover cracking). Therefore, more stable products such as haematite should be used instead. Since they occupy a smaller volume than ferrous hydroxide, they are expected to take longer to cause cover cracking.

### 6.7 Results on corrosion crack patterns

Table 6.1 gives corrosion crack patterns that were exhibited by beams in this research namely; crack pattern A, crack pattern B and crack pattern C. Similar to results from Ballim et al. \[1,2\], El Maaddawy et al. \[3,4\] and Zhang et al. \[17-19\], discussed in Chapter Two, these results indicate that it is difficult to associate corrosion crack patterns with a level of sustained load. For example, Table 6.1 shows that crack pattern B was found for all sustained load levels. Interestingly, crack pattern B was also exhibited by beams 4 to 7 which Chapter Four indicated to have larger mass losses of steel due to long drying periods. The implications of this on the rate of widening of corrosion cracks will be discussed later.

In crack pattern A, a single crack that propagated parallel to corroded steel bars was observed on the tensile face of a beam where corrosion agents were drawn into the concrete. Within the corrosion region, this crack was mostly at the centre of the beam. However, at a distance of about 50 mm before or beyond each end of the corrosion region, the crack split into two and the split cracks diverged to opposite side edges of the beam. The split cracks then crossed to the side faces and propagated to a distance of about 150 mm beyond the ends of the corrosion region and also parallel to the corroded bars as shown in Figure 6.4a. The ends of
the crack (now on the side face) coincided with the edges of the constant moment region. As shown in Table 6.1, only one heavily corroded beam (beam 16) and all lightly corroded beams (beams 8, 11 and 12) from beams tested in this research exhibited this crack pattern.

In crack pattern B, a beam initially cracked on its tensile face as in crack pattern A. However, as corrosion continued, another crack was observed on one side face of the beam whilst the other side face remained uncracked. The crack on the side face of the beam was at the level of the corroding tensile reinforcement and propagated (parallel to the bars) to a distance of about 150 mm beyond the ends of the corrosion region as shown in Figure 6.4b. At a distance of about 50 mm beyond each end of the corrosion region, the crack on the tensile face diverged to the side edge of the beam that corresponded to the uncracked side face. In contrast to a crack in pattern A, the crack on the tensile face for cracks in pattern B did not split into two and ended on the side edge of the beam. It did not propagate on the side face of the beam. The majority of beams in this research exhibited this crack pattern as indicated by Table 6.1. The times at which the crack patterns in various beams changed from crack pattern A to crack pattern B were however, different and ranged from about 20 days to about 50 days.

In crack pattern C, the beam initially cracked on its tensile face (crack pattern A) followed by a crack on one side face (to give crack pattern B) and finally cracked on the other side face as corrosion continued. In this pattern, the crack on the tensile face ended in the middle of the face (at a distance of about 150 mm beyond the end of the corrosion region) and did not diverge to the side edges of the beam as in the other crack patterns. Cracks on the side faces were similar to a single crack on one side face in crack pattern B. Three beams in this research exhibited this crack pattern. It is important to mention that cracks on side faces in crack pattern C were observed at different times. For beam 17, a crack on one side face was observed after 25 days and on the other, after 45 days of testing.
a) Crack on the tensile face of a beam (crack pattern A)

b) Crack on the side face of a beam (crack pattern B or C)

Figure 6.4 Corrosion crack patterns
The above discussed crack patterns were the main patterns that were observed. In a few beams, side faces that were considered uncracked actually had small isolated corrosion cracks which probably indicated a change between crack patterns. They were also shown in Chapter Five to cause rises and depressions on tensile faces of beams (Figure 5.3). As clearly pointed out in Chapter Two, similar crack patterns have been reported elsewhere [1-6]. Change in crack patterns as corrosion progressed was also reported by El Maaddawy and Soudki [20]. The primary difference between crack patterns obtained here to those found by other researchers is that they at times found multiple cracks on tensile faces of beams that propagated along positions of each reinforcing bar [3-5,20]. This is probably because they used larger spacing between bars. For example, El Maaddawy and Soudki [20] used bar spacing of about 80 mm whilst in this research, a spacing of 18 mm between bars was used. Both bar spacings are practical and accepted in various design codes such as the South African National Standard [21].

It was found here and also by other researchers [1-4,17-20] that even for nominally identical specimens, it is difficult to predict the type of crack pattern that each specimen will exhibit. As already mentioned, the majority of specimens here and elsewhere [20], firstly exhibited crack pattern A at the early corrosion stages (such as beams 8, 11 and 12) followed by crack pattern B as corrosion was continued. Relations between the various crack patterns and the rate of widening of each crack, which is the more important concern, is however, uncertain. This is most likely because previous researchers either measured crack widths at one location along specimens during the corrosion process or they measured crack widths at the end of corrosion as pointed out in Chapter Two.

6.8 Results on the rate of widening of corrosion cracks

It should be mentioned again that rather than directly measuring crack widths with various devises such as magnification lenses and crack compactors, in this research, transverse and vertical strains on beams were monitored and then converted to crack widths. Advantages of this set-up were outlined in Chapter Three. Transverse strains on tensile faces of beams as well as vertical strains on their side faces due to steel corrosion were found to be closely related to corrosion
crack patterns. Therefore, the following sections separate their discussion according to each crack pattern.

6.8.1 Widening of corrosion cracks in crack pattern A

Transverse and vertical strains in beam 16 (with crack pattern A) are shown in Figure 6.5. It is clear from the figure that large transverse tensile strains were recorded on the tensile face of the beam. However, vertical strains on its side face were largely contractional. The figure also shows that each measuring point along the tensile face of the beam demonstrated a near-constant rate of increase of transverse tensile strains during testing. This implies that the width of corrosion cracks on the tensile face of the beam widened at a near-constant rate as steel corrosion continued. A maximum rate of widening of corrosion cracks of 15.6 µm/day was observed near the centre of the corrosion region (tt4 and tt5). The lowest rate of 6 µm/day was observed at the ends of the corrosion region. In Chapter Five, larger longitudinal tensile strains on tensile faces of beams were recorded at the ends of corrosion regions. They were attributed to corrosion crack patterns. This suggests that if deformation of the cover concrete is largely along the longitudinal direction to give large longitudinal strains, its deformation in the transverse direction is smaller. To predict accurately the rate of widening of corrosion cracks, it is therefore important to understand the behaviour of the cover concrete due to steel corrosion.

From mass losses of steel found in Chapter Four for beams with two-day drying cycles (1.1 to 1.68 g/day/m of bar length), the rate of widening of maximum corrosion cracks in Figure 6.5a indicates that 1% mass loss of steel corresponded to a maximum crack width between 0.07 and 0.11 mm. These relations are similar to those found by other researchers that were discussed in Chapter Two [1-6,22-24]. They will however, be discussed again later with relations found in other crack patterns. The following explains the behaviour of the cover concrete in crack pattern A to give contractional vertical strains on side faces of beams and tensile transverse strains on their tensile faces.
Figure 6.5 Transverse and vertical strains for crack pattern A

NB! Crack width (mm) = tensile strains (micro strains) * 0.0001
It was shown in Chapter Four using failure modes of corroded beams (Figures 4.11 and 4.12) that where a beam had a corrosion crack on the tensile face and another on the side face, the cover concrete between the cracks was detached from the parent concrete. However, where corrosion cracks were limited to the tensile face, such as in crack pattern A here, ultimate failure of beams did not include delamination of the cover concrete (sections of Figure 4.11b). It is therefore reasonable to assume that for crack pattern A, the cover concrete was attached to the parent concrete beam. However, surely for a crack that is limited to the cover concrete to widen as indicated by Figure 6.5a, some sections of the cover concrete must be detached from the parent concrete beam to permit slippage between them. Note that the process of widening of corrosion cracks is fundamentally different to that of flexural cracks. Instead, flexural cracks run through the parent concrete beam and widen by increasing the curvature of the RC member. Expectedly, flexural cracks alone rarely cause spalling of the cover concrete (because they don’t detach it from the parent concrete) whilst spalling of the cover concrete is common in corrosion cracks.

It is logical that during corrosion, first sections of the cover concrete to be detached from the parent concrete are within the corrosion region. This is because in those regions, the cover concrete will be pushed outwards (as indicated by $\mu_{cr}$ in Figure 6.3) to allow for deposit of voluminous corrosion products. Figure 6.6 shows a picture of the exposed corroded area when beam 11 (crack pattern A) was opened for repair. From the figure, corrosion products were largely observed within the corroded region with little products on the side cover concrete. It is therefore reasonable to assume that for crack pattern A, the cover concrete is only detached from the parent concrete within the corrosion region. The side cover concrete (shown in Figure 6.6) remains fully-attached to the parent concrete beam.

From the above-discussion, a cross-section through E-E of Figure 6.4a can be schematically represented by Figure 6.7. For clarity, only one corroding bar, instead of three used in this research, is shown in the figure. As the figure shows, to allow for deposit of corrosion products around the bar, the cover concrete was pushed outwards. Since it was fixed on the sides, pressure from steel corrosion induced internal moments where it was fixed. Unlike deformation of the cover
concrete discussed in Chapter Five (Figure 5.4) where its fixity induced longitudinal moments, here the fixity induced transverse moments.

Figure 6.6 Distribution of corrosion products in a beam with crack pattern A

It should be mentioned that prior to cover cracking, targets on tensile faces of beams recorded actual transverse tensile strains on fibres of the concrete. Vertical strains on side faces of the beam were therefore due to Poisson’s effects. This can be confirmed by ratios between strains indicated in Figure 6.2. However, after cracking, rather than recording strains on fibres of the concrete, targets on tensile faces recorded widening of corrosion cracks. This is because they were purposely placed across the cracks. Vertical strains on side faces were therefore, not a Poisson’s effect of recorded transverse strains on tensile faces. Moreover, values of vertical strains recorded on side faces of beams were too large to have been due to Poisson’s effects of actual strains on other faces. For example, for a compressive strain of 600 micro strains on concrete to be due to Poisson’s effects, the corresponding tensile strain should be about 3000 micro strains which is not possible for uncracked concrete. Therefore, similar to targets on tensile faces, targets on vertical faces of beams recorded translation of the cover concrete rather than actual strains on fibres of the concrete. The word ‘translation’ is used here to avoid mistaking it for deformation which is a direct result of stresses on the concrete.
From Figure 6.7, widths of corrosion cracks on tensile faces of beams recorded as tensile transverse strains (Figure 6.5a), is because point A shifted to points A'. If points B_1 and B_2 were to be stationary or fixed, then the cover concrete was supposed to significantly compress to allow for the A-A' movement. Considering the width of corrosion cracks that were recorded here (up to 1.6 mm) and by other researchers (up to 3 mm [6, 22]), this is not possible. Therefore point B had to move to points B'. The movement B-B' was possible because fixity of the cover concrete discussed earlier was along C-C whilst point B was free. It is this...
movement that is attributed to the large contractional vertical strains recorded on the side faces of beams.

Certainly vertical stresses applied on the cover concrete by corrosion products shown in Figure 6.7 were resisted by the tensile strength of concrete along C-C. As corrosion continued, the applied stresses increased so that section C-C finally cracked to give crack pattern B.

6.8.2 Widening of corrosion cracks in crack pattern B

Figure 6.8 shows transverse strains on tensile faces and vertical strains on side faces of beams that exhibited crack pattern B. As mentioned in Chapter Three, it was not possible to measure vertical strains on all side faces of a beam. A number of beams under nominally similar conditions such as beams 14 and 15 were therefore tested. The figure shows that under this crack pattern, tensile strains (transverse and vertical) were recorded on faces of beams that had corrosion cracks (Figures 6.8a,c,d) whilst contractional strains were recorded on uncracked side faces (Figure 6.8b). Figures 6.8c,d show that before recording vertical tensile strains on side faces of beams, rates of increase of transverse tensile strains on tensile faces of beams were nearly constant and about the same as in beams with crack pattern A. This is expected since at that stage, they were under the category of crack pattern A. Interestingly, when vertical strains were recorded on side faces of beams such as after 20 days for beam 15, rates of increase of corresponding transverse tensile strains on tensile faces decreased. However, rates of increase of vertical tensile strains on the (now) cracked side faces of beams were nearly constant through the entire testing period as shown in Figure 6.8d. In beam 14, cracks on the side face (which was not monitored) appeared after 20 days. Similar to beam 15, rates of increase of transverse strains on the tensile face of the beam decreased. Interestingly, contractional vertical strains on the uncracked side face of the beam were not affected by cracking of the other side face of the beam. This implies that a longitudinal crack on the tensile face of the beam resulted in the divided parts of the cover concrete behaving independent of each other.
Figure 6.8 Transverse and vertical strains for crack pattern B

(a) Transverse strains on the tensile face of beam 14

(b) Vertical strains on the side face of beam 14

(c) Transverse strains on the tensile face of beam 15

(d) Vertical strains on the side face of beam 15

NB! Crack width (mm) = tensile strains (micro strains) * 0.0001
Despite beams 14 and 15 having similar crack patterns, beam 15 had overall maximum rate of widening of corrosion cracks of 13.5 µm/day compared to 8.9 µm/day in beam 14. This shows the complexity of the rate of widening of corrosion cracks. It will be discussed again later. The following section looks into the behaviour of the cover concrete for this crack pattern.

From the behaviour of the cover concrete in Figure 6.7, vertical tensile strains on side faces of beams can only be recorded when section C-C is cracked. This would imply that within the corrosion region, the part of the cover concrete with a corresponding cracked side face was completely detached from the parent concrete beam. In corroboration, Figure 4.11b showed delamination of part of the cover concrete that exhibited corrosion cracks on the tensile face as well as on the side face. Similar to crack pattern A, the distribution of corrosion products (Figure 6.9) can also be used to assess sections of the cover concrete that were attached or detached from the parent concrete beam. As previously mentioned, where they are observed, then the cover concrete was pushed outwards (detached from the parent concrete) to allow for them to be deposited. Figure 6.9 clearly shows detached sections in crack pattern B to have been the corrosion region (as in crack pattern A) as well as the side cover concrete which corresponded to a cracked side face.

![Image](image_url)
A cross-section of a corroding beam with crack pattern B can therefore be schematically presented by Figure 6.10. As shown in the figure, corrosion products pushed part of the cover concrete with a cracked side face (designated by a subscript 1) directly upwards. The other part behaved like those in crack pattern A. Since point $A'_1$ no longer moved in the transverse direction but rather in the vertical direction, transverse strains recorded on the tensile face of the beam were only due to the movement of $A$ to $A'_2$. Expectedly, Figure 6.8 shows that rates of increase of transverse strains on the tensile face of beam 15 decreased when vertical tensile strains were recorded on the side face. Moreover, part of the cover concrete with a corresponding uncracked side face (designated by subscript 2) continued to exhibit large vertical contractional strains to suggest the $A$-$A'_2$ movement (Figures 6.8b and 6.10).

Figure 6.10 Cross-section of a corroding beam with crack pattern B
6.8.3 Widening of corrosion cracks in crack pattern C

As shown in Figure 6.11, the variation of transverse strains on the tensile face and vertical strains on the side face found in crack pattern C was initially similar to that recorded in crack pattern A followed by that in crack pattern B. Contrary to crack patterns A and B, when both side faces of the beam cracked in crack pattern C (33 days for beam 17), transverse tensile strains on the tensile face of the beam stabilised and ceased to increase despite continued corrosion. After cracking, the rate of increase of vertical tensile strains on the side face of the beam however, remained near constant as shown in Figure 6.11b.

Unlike in crack patterns A and B where it was possible to relate the rate of widening of maximum corrosion cracks with the rate of steel corrosion, in crack pattern C, maximum corrosion cracks (tt4) did not widen for about 50% of the testing time. In this case, it is logical to use the rate of widening of corrosion cracks on the side face. From 28 days (when tensile strains on one side face were recorded) to 64 days, their maximum rate of widening was 8.1 µm/day. In relation to mass losses of steel from Chapter Four for beams with two-day drying cycles, this suggests that 1% mass loss related with a crack width from 0.04 to 0.06 mm. This relation is similar to those found by other researchers (discussed in Chapter Two). It will be discussed again later. The following section explains behaviour of the cracked cover concrete.

It was pointed out under crack pattern B that part of the cover concrete which exhibited cracks on the tensile face as well as on the side face was detached from the parent concrete beam. In crack pattern C here, all side faces were cracked. This implies that over the corroded region, the entire cover concrete was detached from the parent concrete beam. This was corroborated by delamination of the entire cover concrete when beams 9 and 17 (with crack pattern C) were tested to failure (Figure 4.12). Corrosion products in Figure 4.12b were uniformly distributed around the corroded area as well as on the side cover concrete. This again suggests that within the corrosion region, the cover concrete was completely detached from the cover concrete. A cross-section of the beam on the corrosion
a) Transverse strains on the tensile face of beam 17

b) Vertical strains on the side face of beam 17

NB! Crack width (mm) = tensile strains (micro strains) * 0.0001

Figure 6.11 Transverse and vertical strains for crack pattern C
region is shown in Figure 6.12. The figure shows that corrosion products pushed \( A'_{1} \) and \( A'_{2} \) upwards. As indicated by transverse strains on the tensile face, there was little transverse movement of \( A \) to \( A' \). However, vertical strains on the two side faces of the beam increased. Note that here, the cover concrete was attached to the parent concrete only at the ends of the corrosion region.

![Figure 6.12 Cross-section of a corroding beam with crack pattern C](image)

6.9 Discussion on the rate of widening of corrosion cracks

As previously mentioned, prior to a change of crack patterns, the majority of beams exhibited a similar rate of widening of corrosion cracks. This is confirmed using Figure 6.13 which shows the variation of transverse tensile strains on tensile faces of beams after three wetting cycles or 12 days of impressed current density. This period corresponded to a total of 24 days for beams with four-day drying cycles and 18 days for those with two-day drying cycles. Similar to Figures 4.3 to
4.5, the position along the beam shown in the figure was measured from the current input side of the beam to the current output side. ‘Zero’ in the figure therefore indicates the edge of the corrosion region on the current input side whilst 700 mm indicates the edge of the region on the current output side.

Designation: * = corroded using four-day drying cycles; letter in brackets indicates the crack pattern at the end of accelerated corrosion; and a number in brackets indicates the level of steel corrosion during accelerated corrosion.

Figure 6.13 Transverse strains after 12 days of accelerated corrosion

NB! Crack width (mm) = tensile strains (micro strains) * 0.0001
Beams 9, 10 and 20 aside, after three wetting cycles, maximum crack widths from various beams ranged from 0.35 to 0.55 mm. Beams 9 and 10 had maximum crack widths on the side face which (as pointed out in Chapter Three), the initial experimental set-up could not monitor. Beam 20 had maximum crack widths on the side face which also could not be monitored. It however, showed interesting behaviour after repair to be discussed later.

From the variation of mass loss of steel under two-day drying cycles discussed in Chapter Four (1.1 to 1.68 g/day/m of bar length), it can be shown that for maximum crack widths in Figure 6.13 (below 0.6 mm), 1% mass loss of steel corresponded to maximum crack widths between 0.14 and 0.22 mm. These relations give larger rates of widening of corrosion cracks than those presented in Chapter Two from previous researchers. They contended that 1% mass loss of steel corresponds to crack widths between 0.03 and 0.14 mm. As mentioned in Chapter Two, previous researchers monitored crack widths at the end of corrosion. Therefore, there is likelihood that the rates of widening of corrosion cracks that they provided included a change of crack patterns. However, Figure 6.13 is only when beams exhibited crack pattern A. Note that crack widths in Figure 6.13 exceed the lower limit of corrosion crack width of 0.3 mm that DuraCrete Final Technical Report [25] specifies as a criterion for end-of-service life of corroding RC structures. Therefore, on the basis of this crack width, previous relations may overestimate the service life of corroding RC structures.

As already mentioned, above relations were based on mass losses of steel with two-day drying cycles. This is because Chapter Four pointed out that long drying cycles caused larger losses of steel because they allowed more drying of the corrosion region. At the early corrosion stages such as here as well as prior to cover cracking (discussed earlier), the influence of long drying cycles on the rate of steel corrosion was not yet expected. However, for in-service structures where it would take tens of years to reach a crack width of 0.6 mm, different relations are expected. They are discussed using crack widths at the end of steel corrosion as well as during natural steel corrosion.
At the end of accelerated corrosion (64 days for beams with two-day drying cycles and 80 days for those with four-day drying cycles), beam 16 (still with crack pattern A) had a maximum crack width of 1 mm compared to 0.54 mm in beam 17 (now with crack pattern C). Moreover, the widest crack in beam 17 was dormant whilst cracks on the side face of the beam, which were actively-widening, had a maximum width of 0.25 mm (Figure 6.11). In contrast, the crack width of 1 mm on the tensile face of beam 16 was still actively widening. The obvious implication is that end-of-service life that is based on the criterion of maximum corrosion crack width of 1 mm is likely to suggest repair of RC structures with crack pattern A prior to those with crack pattern C. Regrettably, Figures 4.11 and 4.12 indicated crack pattern C to represent a greater degree of total damage. It is therefore important to understand how these crack patterns relate to the level of steel loss and subsequently, to the residual load-bearing capacity of corroded RC structures.

It should be pointed out that Figures 6.5, 6.8 and 6.11 are not always possible to draw for in-service structures because practitioners often measure corrosion crack widths on corroded RC structures at discrete times. They then use maximum corrosion crack widths to predict the level of steel corrosion as well as to assess if there is need for repair [25-27]. To understand the implication of this, Figure 6.14 shows maximum tensile strains (or crack widths) at the end of corrosion for various beams in this research.

Only transverse tensile strains on tensile faces of beams are shown in Figure 6.14 because except for beams 9, 10 and 20 (already discussed), they still exhibited maximum crack widths. Beam 17, despite having a dormant crack on the tensile face for nearly half the total time of accelerated corrosion testing, also exhibited maximum cracks on the tensile face. If steel corrosion was to be continued, it is expected however, that the maximum crack widths in beam 17 would belong with the side face as opposed to the tensile face. Interestingly, other beams that exhibited crack pattern B and had maximum cracks on the tensile face such as beams 6, 7 and 14 had comparable crack widths as beam 17. This points out to the complexity of the rate of widening of corrosion cracks, especially at larger crack widths.
Designation: * = corroded using four-day drying cycles; letter in brackets indicates the crack pattern; and a number in brackets indicates the level of steel corrosion during accelerated corrosion.

Figure 6.14 Transverse strains at the end of accelerated corrosion

Contrary to maximum crack widths after three wetting cycles discussed earlier, maximum crack widths at the end of corrosion were very different. Owing to varying times at which different corrosion crack patterns were observed on beams and since rates of widening of corrosion cracks were closely related to crack patterns, beams were expected to show these varying corrosion crack widths. As
discussed earlier, the figure shows that a beam with crack pattern A (beam 16) had widest corrosion cracks. Unfortunately, narrowest cracks belonged with a beam with crack pattern B (beam 7). Probably the most important information from Figure 6.14 is that despite having larger rates of steel corrosion, beams with four-day drying cycles did not necessarily have the widest corrosion cracks. For example, beam 6 had the largest maximum mass loss of steel (Figure 4.5) and it was within a group of beams (beam 6, 7, 14 and 17) that exhibited the narrowest corrosion cracks. As discussed in Chapter Four, large rates of corrosion on beams with four-day drying cycles was attributed to longer drying periods allowing for more drying of the corrosion region and hence resulting in formation of less voluminous products such as haematite. This surely allowed for more corrosion agents to reach the steel. It also left more space which permitted deposit of additional corrosion products without applying excessive stresses on the cover concrete. However, other beams with four-day drying cycles, especially beam 4, exhibited maximum corrosion crack widths that were comparable with those in beam 16. This was despite beam 4 having crack pattern B. Contrary to other beams where crack pattern B was observed after about 20 days of corrosion, in beam 4 it was observed after 50 days. Therefore, when rates of widening of corrosion cracks on its tensile face were reduced by a change of crack patterns, it had already extensively widened. To confirm this notion, after three wetting cycles, beams 5 and, interestingly, 17 had wider cracks than beam 4.

The overall implication from Figure 6.14 is that for wider cracks, it is difficult to provide a deterministic value for maximum crack widths of corroding beams. Results on crack pattern A are really not conclusive since it was only found in one heavily corroded beam. Previous discussion however, indicated that as corrosion progresses, it changes to crack pattern B. Since this research and others did not find a definite level of steel corrosion at which crack pattern A changed to crack pattern B, there is large variation of crack widths in crack pattern B. It is therefore difficult to predict accurately crack widths in this crack pattern. However, crack pattern C offers the most challenge because unless corrosion cracks are continuously monitored as in Figure 6.11, it is difficult to predict accurately when it occurred.
Discussions of results from Alonso [23] and from Cabrera [5], pointed out that crack pattern C (as in this research) only occurs where the cover depth of exterior bars to the tensile face is equal to their cover depth to the side face. Where they are different, RC specimens exhibit crack pattern A. From Figure 6.7, it is reasonable to assume little deformations of the cover concrete under crack pattern A so that crack widths on the tensile face were mainly due to its translation (movement without bending). Angles $\alpha$ and $\beta$ (shown in Figure 6.7) were therefore nearly equal. This implies that vertical movement of the cover concrete (which gave vertical tensile strains on the side face if C-C was free as in Figure 6.12) was equal to A-A′ movement. It follows then that the rate of widening of corrosion cracks on the side face of a RC beam with crack pattern C should be half its rate of widening on the tensile face if it exhibits crack pattern A. To confirm this notion, for the first 20 days, the rate of widening of corrosion cracks in beam 17 (then with crack pattern A) was 19 µm/day. When it exhibited crack pattern C, the rate of widening of cracks on the side face was 8.1 µm/day. In consent with the above discussion, Zhang et al. [19] recommends that where a corroding bar causes two cracks on adjacent faces of a RC structure, an equivalent crack width should be taken as the sum of the two cracks. If this is implemented on crack widths in beam 17, it gives crack widths that are similar to those found under crack pattern B.

Clearly the variation of strains in Figure 6.14 is best presented as an envelope rather than an average. The lower and upper bounds of the envelope can be represented by equations 6.10 and 6.11. It should be noted that if crack pattern C is treated as discussed earlier, then these envelopes encompass all crack patterns.

Lower envelope (micro strains) = $-0.03x^2 + 24.6x + 34.2$ \hspace{1cm} $R^2 = 0.94$ \hspace{1cm} (6.10)

Upper envelope (micro strains) = $-0.04x^2 + 28.4x + 4815.3$ \hspace{1cm} $R^2 = 0.98$ \hspace{1cm} (6.11)

Where $x$ is the position along the beam (mm), measured from the current input side.
Another interesting feature about Figures 6.13 and 6.14 is that corrosion crack widths on beams varied longitudinally along the corroded region with maximum cracks experienced at the centre of the corrosion region. Smaller cracks were found at the ends of the region. This can be attributed to an increase in transverse stiffness of beams by shear stirrups that were only placed within the shear span. If they were uniformly placed along the beam, maximum corrosion crack widths would probably be at random as found by Zhang et al. [17-19]. Results from El Maaddawy et al. [3,4] where fewer stirrups were placed within the corroded region also showed maximum crack widths at the centre of the corrosion region. On the other hand, Ballim et al. [1,2] found maximum crack widths at beam ends most probably because they had fewer stirrups there. Note that it is a common practice to reduce shear stirrups around constant moment regions. As previously discussed, this research presents the worst scenario where the corrosion region does not have shear stirrups.

The increase in transverse stiffness of the cover concrete by shear stirrups can also be attributed to change of crack patterns at the ends of the corrosion region. At the centre of the region, small transverse stiffness of the cover concrete allowed transverse widening of corrosion cracks which was not permitted at its ends. Bear in mind that Chapter Four revealed steel corrosion beyond the ends of the corrosion region. To allow for deposit of corrosion products there, cracks on the tensile face diverged to side faces as shown in Figure 5.3 and 6.4.

Note that the variation of corrosion crack widths here is similar to the variation of mass loss of steel along the beam that was presented in Chapter Four using Figure 4.5. Therefore, there is merit in relating the envelopes of mass losses of steel (Figure 4.7) with those of corrosion crack widths (Figure 6.14). This is done using Figure 6.15.

It is important to recall that large mass losses of steel belonged with beams with four-day drying cycles. However, those beams did not necessarily exhibit largest corrosion cracks. To be specific, beam 6 had the largest level of steel corrosion and yet was amongst beams with smallest cracks. Furthermore, it exhibited crack pattern B which means it had a continuous widening of corrosion cracks.
Figure 6.15 implies that a corroding RC beam with a maximum crack width of 0.54 mm (crack pattern B) can have mass losses of steel ranging from 7.9% (two-day drying cycles) to 23.4% (four-day drying cycles). A similar range of mass loss of steel can be obtained by a beam with a crack width of 1 mm (crack pattern A). Therefore, according to crack widths limits in [25], one beam may be recommended for repair when it has a mass loss of steel of 7.9% and another considered safe when its mass loss of steel is 23.4%. A conservative approach is to assume that a beam with the smallest crack width has the largest mass loss of steel. In fact, this was the case for beam 6. Under this approach, a crack width of...
0.54 mm corresponded to a mass loss of steel of 23.4%. Assuming a constant rate of widening of corrosion cracks as well as a constant rate of loss of steel, then 1% maximum mass loss of steel corresponded to a maximum crack width of 0.02 mm. 

Note that the above relation follows the recommendation that maximum crack widths in crack pattern C are the sum of two adjacent cracks. If it is not followed, then practitioners are likely to underestimate the level of steel corrosion in RC structures that have this crack pattern. For example, if repair was to be carried out at a crack width of 1 mm, then beam 17 was only going to exhibit it on the side face after about 150 days. If the beam had the maximum rate of steel corrosion found in this research, then at repair, it was expected to have a mass loss of steel of about 53%.

6.10 Results on corrosion crack widths during natural steel corrosion

Results presented above are when corrosion was accelerated by impressing an anodic current and the sustained load was kept constant. In real structures, steel corrosion is natural and applied loads often change with time. To simulate this, after accelerated corrosion, beams 16 and 20 were firstly allowed to corrode naturally before repairs. As mentioned in Chapter Three, during this natural corrosion phase, the load on beam 16, which was initially at 8%, was increased to 12% whilst the load on beam 20, which was initially at 12%, was reduced to 8%. The changes in load levels were intended to simulate real situations where in the worst case, no action is taken after steel corrosion and yet there is a corresponding increase in the level of the load that is imposed on the structure (beam 16), or the load on a structure is reduced after noticing corrosion damage (beam 20).

From Figure 6.16 it is clear that by increasing the level of the sustained load on a corroded beam that exhibited crack pattern A, the rate of corrosion crack widening on its tensile face that was recorded during the accelerated corrosion phase was maintained during the natural corrosion phase. This was despite no anodic current being applied during the natural corrosion phase in addition to no cyclic wetting of the corrosion region with a salt solution. On the contrary, Figure 6.17 shows that natural corrosion on a beam with crack pattern B that was accompanied by a
reduction of the level of the sustained load did not induce further widening of corrosion cracks. Similar to the variation of strains on cracked faces of beams, compressive strains measured on the uncracked side face of beam 16 increased during the first natural corrosion phase whilst the corresponding strains in beam 20 remained constant. Contrary to results during the accelerated corrosion process where it was difficult to associate corrosion crack widths with the level of the sustained load (Figures 6.13 and 6.14), it is evident here that when steel corrosion is natural, larger sustained loads cause larger corrosion crack widths. This emphasises the importance of assessing behaviour of corroding RC structures when under a sustained load that was discussed in Chapters Two and Three.

In addition to the above discussion, it can be observed from Figures 6.16 and 6.17 that the increase in the level of the sustained load had little influence on existing pattern of corrosion cracks. This finding is of great importance to understanding the serviceability of corroding RC structures. It shows that even though an increase in the level of the sustained load may indirectly result in a corresponding increase in the rate of widening of corrosion cracks, it does not necessarily initiate spalling of the cover concrete. This is because earlier discussions in this chapter as well as in Chapter Four indicated that spalling of the cover concrete only occurs when beams exhibit crack pattern C and have large flexural cracks. Another important observation during the two corrosion phases was that whilst substantial corrosion products were exuded to exterior surfaces of beams during accelerated corrosion phase, little new corrosion products were observed on faces of beams during the natural corrosion phase. This can be attributed to easier discharge of more-soluble products, ‘green-rust’, to the exterior faces of beams during accelerated corrosion compared to less soluble products, ‘red-rust’ during natural corrosion.

6.11 Discussion on corrosion crack widths during natural steel corrosion

As previously mentioned, the rate of corrosion crack widening during the natural corrosion phase in beams when subjected to a load of 12% was essentially the same as the corresponding rate of crack widening during the accelerated corrosion phase. This was despite the rate of loss of steel being significantly larger (about
Designation: NC = natural corrosion; AC = accelerated corrosion; and a number after the two letters indicates the level of sustained load during corrosion.

Figure 6.16 Transverse and vertical strains in beam 16 during natural corrosion
Designation: NC = natural corrosion; AC = accelerated corrosion; and a number after the two letters indicates the level of sustained load during corrosion.

Figure 6.17  Transverse and vertical strains in beam 20 during natural corrosion
three times) during the accelerated corrosion phase than in the natural corrosion phase. Furthermore, more stable corrosion products such as haematite and magnetite were expected during the natural corrosion phase when the concrete was drier and with sufficient oxygen. On the other hand, unstable products such as ferrous hydroxide were expected during the accelerated corrosion phase when the corrosion region was wet and probably with less oxygen. Corrosion products observed around the steel when beams were opened for repair were greenish-black in colour indicating a large presence of ferrous hydroxide whilst reddish-brown products were found after testing beams to failure indicating a large presence of more stable corrosion products such as haematite. Interesting though, as mentioned in Chapters Two and Four, less-stable corrosion products occupy a larger volume than more-stable products. Similar to the rate of corrosion, this supports the rate of crack widening to have been larger during the accelerated corrosion phase.

The possible cause for the high rate of widening of corrosion cracks during the natural corrosion phase was the level of the sustained load. Evidently, with the concrete fully-cracked, additional tensile strains within the corrosion region due to an increase in the load were balanced only by the corroded steel. Large deformations of the steel were as such expected (especially since the area of steel was reduced) which could have resulted in the steel being pressed against the cover concrete. It was shown in Chapter Four that corrosion of steel bars is often concentrated on surfaces of the steel that face the direction of ingress of corrosion agents. For the set-up in this work, it is this corroded surface of the bar that would be pressed against the cover concrete due to the increased load. An increase in the level of the sustained load on beams therefore resulted in a reduced space for deposit of new corrosion products during the natural corrosion phase. It is as such expected that continued steel corrosion applied stresses on the cover concrete which consequently caused further widening of the corrosion cracks. On the other hand, a reduction in the level of the sustained load caused a relaxation of the steel and thus created more space for deposit of new corrosion products. This explains no additional widening of the cracks with the reduced load (Figure 6.17).
Another parameter that might be responsible for the rates of corrosion crack widening during the corrosion phases is the rate of egress of corrosion products to the exterior surfaces of the concrete. As previously mentioned, substantial corrosion products were observed on the exterior surfaces of the concrete during the accelerated corrosion phase whilst little new corrosion products were observed during the natural corrosion phase. Obviously discharge of corrosion products from the corroding area relieves the corrosion pressure on the cover concrete. On the other hand, accumulation of corrosion products on the corroding area during the natural corrosion phase probably increased the corrosion pressure on the cover concrete which then caused further widening of corrosion cracks.

On the overall, these results imply that for a selected level of steel corrosion, natural steel corrosion caused wider corrosion cracks than accelerated steel corrosion. As already mentioned, the rate of loss of steel during accelerated corrosion was about three times larger than the corresponding rate during natural steel corrosion. However, the rate of widening of corrosion cracks during the two corrosion phases was similar. Therefore, if practitioners are to use crack widths on in-service structures to predict the level of steel corrosion using relations from accelerated tests, they are likely to underestimate it.

Note that accelerated corrosion used in this research was below the maximum current density that according to El Maaddawy and Soudki [20] does not alter the structural response that would be obtained under natural steel corrosion. The recommended current density by the researchers is therefore most probably limited to specimens that are corroded in the absence of a sustained load. This clearly shows that results from various laboratory tests that continue to be conducted worldwide on accelerated steel corrosion are not fully representative of natural corrosion of in-service structures. Owing to the slow rate of natural steel corrosion, there is merit in conducting accelerated corrosion tests in laboratories. However, they must be standardised and calibrated by results from natural steel corrosion under a sustained load. The results from this chapter (which are a step forward towards this calibration process) indicated that under the accelerated corrosion test, a corrosion crack width of 1 mm corresponded to a mass loss of
steel of 50%. However, when corrosion was natural and under a sustained load, a corrosion crack width of 1 mm corresponded to a mass loss of steel of 8%.

### 6.12 Results on corrosion crack widths after repairs

As mentioned in Chapter Three, after the accelerated corrosion process, selected beams were patch repaired. Subsequently, loads on repaired beams were increased to assess the performance of repairs. It was shown in Chapter Four that there was no further steel corrosion after patch repairs. As a result, no corrosion crack widening was observed on patch repaired beams. Discussions on this section will focus on RC beams that were repaired with FRPs without patch repairs.

Figure 6.18 shows that the rate of corrosion crack widening on the side face of beam 15 that was recorded during the accelerated corrosion phase was unaffected by bonding a FRP plate on the tensile face of the beam. Interestingly, this rate of crack widening was recorded during the curing period of the epoxy (when the load was at 8%) as well as when the epoxy was fully cured and the load was increased to 12% of the load-bearing capacity of an uncorroded beam. This shows that if the side face of a beam is already-cracked, the rate of further corrosion crack widening during natural corrosion phase is principally controlled by the repair scheme used as opposed to the level of the sustained load. To confirm this finding, a slight reduction in the rate of corrosion crack widening was recorded when the sustained load was reduced from 12% to a load equivalent to 1% of the load-bearing capacity of an uncorroded beam. The load of 1% was achieved by removing the beam on the test frame and placing it (whilst inverted) on concrete blocks of 100 x 100 x 200 mm placed at the middle third points of the beam span as previously discussed in Chapter Three. The rate of corrosion crack widening was however, significantly reduced by wrapping the beam with FRP sheets. Compared to results from beams 16 and 20 before repairs, these results clearly show that (the level of sustained load aside) the type of FRP repair scheme used has a significant effect on the rate of widening of corrosion cracks. It is therefore
Designation: AC = accelerated corrosion; FP = FRP repair; FW = repair with FRP wraps; and a number after the two letters indicates the level of sustained load.

Figure 6.18 Transverse and vertical strains in beam 15 after FRP repair
evident that complete wrapping of corroding RC structures with FRP sheets as done by other researchers [24,28], is not representative of the true response of real structures to steel corrosion where FRPs are only bonded to tensile faces of the structures.

Figures 6.19 and 6.20 show that after bonding FRP plates to cracked tensile faces of beams 16 and 20, corrosion cracks on tensile faces of beams ceased to widen. Interestingly, this was despite an increase in the level of the sustained load. Compared to beam 15, these results suggest that repairs of corroding RC structures with FRP plates may prevent further corrosion crack widening if plates are bonded directly over cracks and along the same line of propagation of cracks. However, immediately after repairs, the side face of beam 16 which initially exhibited constant vertical strains that were largely contractional, started to exhibit an increase in the vertical tensile strains. Similar to results from beam 15, the first rate of increase of the vertical strains in beam 16 was recorded at a sustained load of 8%. The rate of increase of the strains increased substantially and became constant following an increase in the level of the sustained load.

A further increase of the load from 12% to 16% had a little effect on the rate of widening of corrosion cracks in beam 16 however, the side face of the beam that was uncracked developed new corrosion cracks. Similar results were found on beam 20 when the load was increased from 8% to 12%. It can actually be clearly observed in Figure 6.20 that increasing the load from 12% to 16% resulted in a noticeable increase in the rate of widening of corrosion cracks on the side face of beam 20 which was partially-cracked. This indicates that when a fully-cracked corroding RC structure is repaired with FRPs using the scheme in this research, an increase in the level of the sustained load has a little effect on the rate of widening of corrosion cracks. For uncracked or partially-cracked RC structures however, an increase in the level of the sustained load causes new corrosion cracks to develop or an increase in the rate of widening of existing cracks.
Designation: AC = accelerated corrosion; NC = natural corrosion FP = FRP repair; and a number after the two letters indicates the level of sustained load.

Figure 6.19 Transverse and vertical strains in beam 16 after FRP repair
Designation: AC = accelerated corrosion; NC = natural corrosion FP = FRP repair; and a number after the two letters indicates the level of sustained load.

Figure 6.20 Transverse and vertical strains in beam 20 after FRP repair
Another unexpected behaviour from beams 16 and 20 after repairs is that cracks on side faces of beams 16 and 20 were observed when demec gauges recorded compressive vertical strains. This is in contrast to the accelerated corrosion phase where corrosion cracks were observed on faces of beams when tensile strains were about 800 micro strains. For cracks to have been visible (in this case to a naked eye), they ought to have had widths between 0.05 and 0.1 mm which, the demec gauge would have read as tensile strains of 500-1000 micro strains. This indicates that compressive strains that were initially recorded on beams during the accelerated corrosion phase were not recovered before cracking of the side face. The line of deformation of the cover concrete was as such altered by the repair scheme. It can therefore be concluded that rather than controlling widening of corrosion cracks (as was observed with cracks on the tensile face), the repair scheme merely changed crack patterns. For those responsible for monitoring service life of corroding RC structures such as structural engineers and asset managers, the change in crack patterns due to the strengthening scheme might even be worse than widening of already-existing cracks. This is because; new cracks imply further loss in bond between steel and concrete as well as the bond between the cover concrete and the parent concrete; if new cracks join already-existing cracks, they may result in spalling of the concrete; and cracks may cause debonding of FRPs. From these results, the worst case of serviceability failure based on the criterion of corrosion crack widths was therefore recorded when beams were strengthened for flexure using FRPs followed by an increase in the level of the sustained load.

6.13 Discussion on corrosion crack widths after repairs

Bonding FRP plates to tensile faces of beams prevented further widening of corrosion cracks on tensile faces due to the bond between the epoxy adhesive and the concrete. Similar to the behaviour of the cover concrete before cracking, the repair system in beams that only exhibited cracks on the tensile face (such as beam 16) therefore possibly behaved like a thick-walled cylinder under internal pressure. From Chapter Four, the pressure was applied on surfaces of bars that faced the direction of ingress of corrosion agents as shown in Figure 6.21. It is likely that the bond between the epoxy adhesive and the concrete increased the
transverse stiffness of the cover concrete. Further corrosion crack widening due to continued steel corrosion was therefore limited to side faces of beams which was controlled by the now-comparatively-smaller vertical stiffness of the cover concrete. After strengthening with FRPs, the flexural response of the composite beams was controlled by FRPs and corroded steel bars. Changes in the level of the sustained load therefore did not affect the position of corroded steel bars in relation to the cover concrete. Unlike in non-repaired beams, the rate of corrosion crack widening was not overly dependent on the level of the sustained load but on the FRP repair scheme used (Figure 6.18).

Figure 6.21 Cross-section of a corroding beam after FRP repair
6.14 Conclusions

This chapter presented results and discussions on the time of first cracking of the cover concrete. It also discussed the rate of widening of corrosion cracks during accelerated corrosion, natural corrosion of non-repaired beams and natural corrosion of repaired beams. The following are conclusions that are specific to this chapter.

1. Prior to cracking of the cover concrete, transverse tensile strains that linearly increased with the time of testing were recorded on tensile faces of beams which faced the direction of ingress of corrosion agents. Side faces of beams however, largely exhibited contractional strains. At cracking of the cover concrete, a much larger rate of increase of transverse tensile strains was recorded. This change made it possible to easily identify the level of transverse strains on tensile faces of corroding beams that indicated cover cracking. An analytical model, calibrated by these results was developed.

2. After cracking of the cover concrete and with corrosion cracks below 0.6 mm, it was found that various beams exhibited a near-similar rate of widening of corrosion cracks. It was therefore concluded that service life of corroding RC structures that is based on corrosion crack widths below 0.6 mm such as a limit of 0.3 mm by DuraCrete Final Technical Report [25] is unlikely to be significantly variable between identical structures. For corrosion crack widths greater than 0.6 mm, it was found that beams exhibited three different types of crack patterns; crack pattern A, B and C, with crack pattern A having the largest rate of widening of corrosion cracks followed by crack pattern B. Based on the rate of widening of corrosion cracks from the various crack patterns, it was concluded that service life of corroding RC structures that is based on the criterion of corrosion crack widths above 0.6 mm such as a limit of 1 mm [25], is likely to prescribe repairs of structures that exhibit crack pattern A before those that exhibit crack patterns B or C.

3. Chapter Four clearly showed that if steel corrosion entails wetting and drying of specimens with salt solution, longer drying cycles yield larger rates of steel
corrosion. On the contrary, this chapter showed that the patterns of corrosion cracks as well as the rate of widening of the cracks in each pattern were independent of the duration of the drying cycles. It was therefore concluded that the worst scenario to structural engineers and asset managers was when a structure had longer drying cycles (larger corrosion rates) and exhibited crack pattern C (low rate of widening of corrosion cracks). Without a clear understanding of the relations between; the pattern of corrosion cracks; the rate of widening of the cracks; and the rate of steel corrosion, previous relations between maximum corrosion crack widths and maximum loss of steel area due to corrosion are likely to prescribe repairs of structures that exhibit crack pattern C (worse if they have longer drying cycles) at dangerous levels of steel corrosion [3,6,22,23]. Furthermore, current models on the rate of widening of corrosion cracks are of little value to structural engineers principally because they assume that structures only exhibit crack pattern A [29,30].

4. Based on the variation of corrosion crack widths, it was recommended that for beams that exhibit cracks on the tensile face as well as on the near side face (crack pattern C), the maximum crack width should be taken as the sum of the maximum cracks from each face of the beam. If the recommendations in this research are followed then a maximum mass loss of steel of 1% corresponded to a maximum corrosion crack width of 0.02 mm.

5. Another interesting finding from this chapter was that increasing the level of the sustained load did not alter the pattern of corrosion cracks. Bonding FRP plates to tensile faces of corroding RC beams was however, found to stop the rate of widening of immediate corrosion cracks but cause new corrosion cracks to develop on other faces of beams. It was therefore concluded that FRP repairs rather than controlling corrosion cracking, merely altered the pattern of corrosion cracks. Since patch repairs were found to control further steel corrosion but not to increase the load-bearing capacity of the structures, it was recommended that repairs of corroding RC structures should involve patch repairs that are accompanied with FRP repairs.
6.15 References


CHAPTER SEVEN

CONCLUSIONS AND FURTHER RESEARCH

7.1 Summary of research background

As discussed in the thesis, structural engineers and asset managers rely on measurable parameters of corroding RC structures to predict their residual load-bearing capacity as well as to assess effectiveness of repairs. Expectedly, extensive research has been carried out to study these parameters. Unfortunately, previous researchers used test conditions that were far from in-service conditions. The primary matter that was discussed was the paucity of research on the influence of load on performance of corroding RC structures. Results that they obtained from specimens tested in the absence of a sustained load could have therefore been an underestimate of the synergistic effects of load and steel corrosion on RC structures. In addition, most of accelerated corrosion tests used were such that results obtained were difficult to relate to those from in-service structures. For example, current densities used were up to 10400 µA/cm² despite El Maaddawy and Soudki [1] recommending densities below 200 µA/cm². Furthermore, some researchers physically added chlorides to their concrete mixes [1,2] whilst others immersed theirs in salt solution [3,4]. Surely these resulted in uniform distribution of corrosion agents around the steel. However, chlorides are often excluded from corrosion mixes in practice and in in-service structures, limited faces of structures are exposed to corrosion agents. This implies that in-service structures have partial distribution of corrosion agents around corroding steel.

Another setback with previous research was substandard procedures for monitoring experimental results. For example, most researchers measured corrosion crack widths at the end of steel corrosion [5,6]. Information needed by structural engineers on the rate of widening of corrosion cracks was therefore not given. The level of steel corrosion was measured as average mass loss of steel [1,5-7] despite load-bearing capacity being mostly dependent on maximum mass
loss of steel [8]. As a result, researchers were forced to develop unnecessary correction factors to calibrate standard theoretical models [7]. Stiffness was deduced from slopes of load-deflection curves found from subjecting already-corroded specimens to monotonically increasing static loads [2,3,9]. Instead of measuring changes in stiffness due to continued steel corrosion at a given load, which is more relevant to corroding in-service structures, they provided stiffness at a given level of steel corrosion due to increased load. Little research where specimens were corroded under load showed a much larger rate of loss of stiffness due to steel corrosion [5,10,11].

Probably the worst monitored parameter was the time to cover cracking of concrete due to steel corrosion. Most researchers had to wait for cracks to occur and then widen sufficiently to be seen with a naked eye [6,12,13]. Some of their ‘rather complex’ analytical models (which were calibrated by these results) however, postulated that cracking commences at the interface of corroding steel and the concrete and then propagates outwards [14,15]. Unsurprisingly, they yielded poor results.

Repairs on corroded RC structures are often carried out to increase their service life. Similar to laboratory corrosion tests, the majority of laboratory tests on performance of repairs were substandard. For example, no literature was found on patch and FRP repairs that were carried out on corroded RC structures under load. Results available in the literature where specimens were repaired in the absence of sustained loads [16-19] are therefore not very useful to structural engineers and asset managers. In fact, they can be deceiving.

A holistic research where these shortcomings were avoided was presented. As already pointed out, its primary intention was to provide structural engineers and asset managers with measurable parameters that they can use to predict load-bearing capacity of corroding RC structures as well as performance of repairs. However, from the above discussion, it is important to also provide research findings that would improve future research on corrosion process, corrosion damage, performance of repairs and how best to simulate it using laboratory tests.
7.2 Rate of accelerated and natural steel corrosion

This research was carried out because of increased degradation of RC structures within the marine environment due to corrosion of embedded steel. It is logical that prior to providing parameters that can be used to predict the effects of steel corrosion on load-bearing capacities of RC structures, its rate must be well known. Corrosion rate is normally provided in terms of current density which can then be converted to mass loss of steel using Faraday’s Law. For laboratory research such as here, a constant current density is often impressed to shorten the needed testing time. To provide more understanding on how to best represent natural steel corrosion, different corrosion processes were used in this research; accelerated corrosion using two-day drying cycles and four-day wetting cycles and a similar process where four-day drying cycles were used instead; and accelerated corrosion followed by natural steel corrosion. It is worth pointing out that the bulk of natural steel corrosion was after repairs. During accelerated corrosion tests, chlorides were introduced on one surface of concrete. This was meant to best simulate in-service conditions where one face of the structure is exposed to chloride attack. Rates of steel corrosion at the end of tests were assessed by measuring mass loss of steel. Rather than only measuring average mass loss of steel as done by the majority of previous researchers, maximum mass loss of steel was also measured.

It was found that when corrosion involved four-day wetting cycles followed by two-day drying cycles, the maximum rate of steel corrosion ranged from 1.10 to 1.68 g/day/m of bar length. Interestingly, when four-day drying cycles were used, corresponding rates of loss of steel ranged from 1.68 to 2.60 g/day/m of bar length. Moreover, mass loss of steel from long-drying cycles was more localised. Theoretical loss from Faraday’s Law was 1.39 g/day/m of bar length. Large mass losses of steel when long drying cycles were used were attributed to more drying of the corrosion region. It allowed for the formation of stable corrosion products which occupy less volume than unstable products. They therefore left more space for ingress of more corrosion agents.
From these results it is clear that when long drying cycles were used, actual mass loss of steel was much larger than mass loss predicted from Faraday’s Law. However, a closer relation between predicted and measured loss of steel was found when shorter drying cycles were used. As found by previous researchers, if continuous wetting of samples was used, Faraday’s Law often over-predicted mass loss of steel. Note that corrosion in in-service structures involves long drying cycles which can last months. Certainly, they also exhibit rates of steel corrosion that exceed rates from Faraday’s Law. Therefore, if structural engineers were to apply to corroding in-service structures, rates of actual loss of steel obtained from laboratory tests where short or no drying cycles were used, they will likely underestimate their rate of loss of steel. It was recommended that laboratory corrosion tests should employ long drying cycles.

Probably specific to this research, maximum mass loss of steel was found at the centre of the corrosion region. This was attributed to shear stirrups near the ends of the corrosion region preventing widening of corrosion cracks and hence limiting ingress of corrosion agents. The set-up was purposely selected to assess the worst scenario in in-service structures where steel corrosion occurs in constant moment regions with limited shear stirrups.

Another important finding on mass loss of steel was that it was localised at the surface of steel bars that faced the direction of ingress of corrosion agents. This is also expected in in-service structures. The results of this will be addressed again under sections on measurable parameters.

It should be mentioned that no further steel corrosion was measured after patch repairs. For FRP-repaired beams, the maximum rate of loss of steel due to natural steel corrosion ranged from 0.403 to 0.603 g/day/m of bar length. This was equivalent to current densities of 45.5 to 66.5 µA/cm². Similar to accelerated corrosion with long drying cycles, more localised corrosion was found. This emphasises the notion that natural steel corrosion is best represented by accelerated corrosion that involves long drying cycles. Most importantly, it shows that FRP repairs do not solve the problem of steel corrosion. The following
sections give measurable parameters that structural engineers can use to assess performance of corroding and repaired RC beams.

7.3 Longitudinal strains and stiffness of corroded and repaired RC beams

During testing, longitudinal strains were measured at different positions along the corrosion region as well as across the depth of a beam. They were firstly looked at discretely and then converted to the depth of the neutral axis and to stiffness.

Longitudinal strains at some sections of beams were found to significantly increase with continued steel corrosion. However, location of maximum strains did not coincide with the location of maximum loss of steel. To be specific, maximum mass loss of steel occurred at the centre of the corrosion region whilst maximum longitudinal strains were mainly recorded at the ends of the corrosion region. The variation of longitudinal strains was attributed to crack patterns. It is therefore difficult to associate the magnitude of longitudinal strains with the level of steel corrosion.

Another important finding about longitudinal strains was that at a section, strains across the depth of a beam were non-uniform. Average longitudinal strains measured over the entire corrosion region were however, found to uniformly vary across the depth of a beam. It was therefore recommended that monitoring corroding RC structures using the variation of longitudinal strains should avoid measuring strains at a selected point and rather measure average strains over the entire corrosion region.

Variations of longitudinal strains during the corrosion process were such that there were unnoticeable changes in the depth of the neutral axis despite significant changes in the level of steel corrosion. Clearly, if in assessing a corroding RC structure using the variation of the depth of the neutral axis, structural engineers are not fully aware of the behaviour of corroded RC structures, they are likely to believe from the variation of the depth of the neutral axis that the structure is free from damage or its functionality is not affected by damages that other indicators such as corrosion cracks might be showing. It was therefore recommended that
they should avoid using the depth of the neutral axis as an indicator of the level of steel corrosion or as an indicator of the structural integrity of a corroded structure.

Flexural stiffness was generally found to decrease sharply at the early corrosion stages (2.5 to 3.8% mass loss of steel). However, after reaching the fully-cracked stiffness, it became constant despite continued increase in the level of steel corrosion. It was concluded that stiffness could be used to indicate the level of steel corrosion at the early corrosion stages. However, the level of steel corrosion at which it remained constant despite a continued increase in the level of corrosion was contended to be too low to indicate the need for repairs of structures on the basis of the criterion of load-bearing capacity. In corroboration, it was also found that changes in the level of the applied load (especially increasing it) on corroded RC structures had little effect on their stiffness. This could deceive structural engineers into specifying a corroding RC structure as structurally sound. They were therefore cautioned against using stiffness of heavily corroded RC structures as a test of their structural integrity.

At an equivalent level of sustained load, larger longitudinal strains were recorded on corroded RC beams that were repaired with FRPs than on corroded but non-repaired beams. This was attributed to development of new corrosion cracks due to further steel corrosion in FRP-repaired beams. There was however, little change in stiffness to indicate this further steel corrosion.

Shrinkage strains from patch repairs carried out under load were found to redistribute residual stresses in a corroded RC beam such that it appeared stiffer. A slight increase of the applied load (after hardening of repairs) however, caused extensive cracking on the repairs. These cracks would certainly allow more corrosion agents to reach the steel and hence further corrode which would aggravate asset managers. This implies that patch repairs, especially on structures where applied loads will subsequently be increased, are inadequate to increase the service life of a corroded structure. Engineers in the field were also cautioned against using stiffness to measure performance of repairs.
Interestingly, the above shortcomings were avoided when corroded RC beams were firstly repaired with a mortar followed by FRPs. It was therefore recommended that repairs of corroded structures should entail combined patch and FRP repairs.

### 7.4 Cover cracking of corroded and repaired RC beams

Another measurable parameter of corroding RC structures that was assessed in the research was cracking of the cover concrete. Rather than waiting for cracks to appear and then measure their widths using various devices, here transverse strains on tensile faces of beams and vertical strains on their side faces were measured. This set-up made it possible to measure accurately the time to cover cracking as well as the rate of widening of corrosion cracks.

Prior to cracking of the cover concrete, transverse tensile strains that linearly increased with time of testing were recorded on tensile faces of beams that faced the direction of ingress of corrosion agents. Side faces of beams however, largely exhibited contractional strains. This was attributed to loss in steel being localised on surfaces of steel bars that faced the direction of ingress of corrosion agents. At cracking of the cover concrete, a much larger rate of increase of transverse tensile strains was recorded. This change in the rate of increase of strains made it possible to easily identify the level of transverse strains on tensile faces of corroding beams that indicated cover cracking. A simple analytical model, calibrated by these results was developed. It showed first cracking of cover concrete due to steel corrosion to occur at a mass loss of steel of 0.39%. If loss in steel was uniform, it was contended that cover cracking would occur at a mass loss of steel of 1.1%. Therefore, partial surface steel corrosion (which is likely to also occur in in-service structures) applies larger internal pressure than uniform steel corrosion. Researchers were therefore cautioned against adding chlorides to concrete mixes as well as immersing their samples in tanks filled with salt solution.

After cracking of the cover concrete and with corrosion cracks below 0.6 mm, it was found that various beams exhibited a near-similar rate of widening of
corrosion cracks. It was therefore concluded that service life of corroding RC structures that is based on corrosion crack widths below 0.6 mm such as a limit of 0.3 mm by DuraCrete Final Technical Report [20] is unlikely to be significantly variable between identical structures. For corrosion crack widths greater than 0.6 mm, it was found that beams exhibited three different types of crack patterns; crack pattern A, B and C, with crack pattern A having the largest rate of widening of corrosion cracks followed by crack pattern B. Based on the rate of widening of corrosion cracks from the various crack patterns, it was concluded that service life of corroding RC structures that is based on the criterion of corrosion crack widths above 0.6 mm such as a limit of 1 mm [20], is likely to prescribe repairs of structures that exhibit crack pattern A before those that exhibit crack patterns B or C.

Contrary to the rate of steel corrosion, patterns of corrosion cracks as well as the rate of widening of the cracks in each pattern were independent of the duration of drying cycles. It was therefore concluded that the worst scenario to structural engineers and asset managers was when a structure had long drying cycles (larger corrosion rates) and exhibited crack pattern C (low rate of widening of corrosion cracks). Without a clear understanding of the relations between; the pattern of corrosion cracks; the rate of widening of cracks; and the rate of steel corrosion, previous relations between maximum corrosion crack widths and maximum loss of steel due to corrosion are likely to prescribe repairs of structures that exhibit crack pattern C (worse if they have longer drying cycles) at dangerous levels of steel corrosion [1,6,13,21]. Furthermore, current models on the rate of widening of corrosion cracks are of little value to structural engineers principally because they assume corroding RC structures to only exhibit crack pattern A [22,23].

Based on the variation of corrosion crack widths, it was recommended that for beams that exhibit cracks on the tensile face as well as on the near side face (crack pattern C), the maximum crack width should be taken as the sum of the maximum cracks from each face of the beam. A similar recommendation was given by Zhang et al. [24]. If this recommendation is followed then to be conservative, a maximum mass loss of steel of 1% corresponded to a maximum corrosion crack width of 0.02 mm.
Under natural steel corrosion, increasing the level of the sustained load maintained the rate of widening of corrosion cracks and did not alter the pattern of corrosion cracks. Bear in mind that the rate of accelerated steel corrosion was about three times larger than the rate of natural steel corrosion. The implication of this is that at a selected level of steel corrosion, natural steel corrosion caused wider corrosion cracks than accelerated steel corrosion. This was despite using the level of current density that was below the maximum recommended density by El Maaddawy and Soudki [1]. This clearly shows that results from various laboratory tests that continue to be conducted worldwide on accelerated steel corrosion are not fully representative of natural corrosion of in-service structures. Owing to the slow rate of natural steel corrosion, there is merit in conducting accelerated corrosion tests in laboratories. However, they must be standardised and calibrated by results from natural steel corrosion under a sustained load. Results from this thesis (which are a step forward towards this calibration process) indicated that under the accelerated corrosion test, a corrosion crack width of 1 mm corresponded to a mass loss of steel of 50%. However, when corrosion was natural and under a sustained load, a corrosion crack width of 1 mm corresponded to a mass loss of steel of 8%.

Bonding FRP plates to tensile faces of corroding RC beams was however, found to stop the rate of widening of immediate corrosion cracks but to cause new corrosion cracks to develop on other faces of beams. Their rate of widening was similar to the rate of widening of corrosion cracks during natural steel corrosion of non-repaired beams with a high load. It was therefore concluded that FRP repairs rather than controlling corrosion cracking, merely altered the pattern of corrosion cracks.

7.5 **Load-bearing capacity of corroded and repaired RC beams**

The load-bearing capacity of corroded and patch-repaired RC beams was found to decrease with an increase in the level of steel corrosion. Expectedly, it was mostly related to maximum mass loss of steel rather than average mass loss. In fact no correction factors were required in theoretical models when maximum mass loss of steel was used.
Patch-repaired beams exhibited a slightly larger (2.5%) load-bearing capacity than non-repaired beams. Considering large safety factors that are often used in design for ultimate capacity of RC structures, this increase was contended to be insignificant. In summary, for every 1% maximum mass loss of steel, there was a corresponding 0.8% reduction on load-bearing capacity.

As expected, significant increase (about 50%) in load-bearing capacities of RC beams were found when they were repaired with FRPs. To emphasise this increase, it was shown that theoretically, a mass loss of steel of 78% was required for a FRP-repaired beam to have a load-bearing capacity that was equal to the capacity of a non-corroded RC beam. As already discussed, this ‘good performance’ of FRPs was thwarted by excessive corrosion cracking that they indirectly caused on beams. Note that the failure mode of FRP-repaired beams was debonding of FRPs. Therefore, further corrosion cracking might debond FRP-repaired beams at a lesser load than what was found here. This notion requires to be validated. Some FRP-repaired beams (beams 16 and 20) are as such under continued steel corrosion. It is logical that despite the capacity of FRP-repaired beams being much larger than the capacity of non-FRP-repaired beams as well as not being very sensitive to changes in the level of steel corrosion, FRP-repairs of corroded RC structures should be carried out subsequent to patch repairs.

7.6 Summary of variation of measurable parameters

Figure 7.1 shows a summary of measurable parameters discussed above, before and after repairs. At the early corrosion stages (below 3.8%), flexural stiffness varied sufficiently to be used to predict loss in load-bearing capacity. While the loss in stiffness was found to be best presented by a power function, for simplicity, it is shown here (Figure 7.1b) as linear. At a mass loss of steel of 3.8%, when stiffness could no longer predict load-bearing capacity, Figure 7.1a shows that the loss in load-bearing capacity was about 3%. This loss is certainly too small to prescribe repairs. However, Figure 7.1c shows that corrosion crack widths then were about 0.5 mm, which exceeded the limit of 0.3 mm from DuraCrete Final Technical Report [20]. Therefore, at this stage, the two measurable
Figure 7.1 Measurable parameters of corroding and repaired RC beams
parameters can be used to indicate the residual load-bearing capacity of corroding RC beams. Structural engineers would surely opt for corrosion crack widths because they are easier to measure.

Where flexural stiffness is more important is prior to cover cracking. Bear in mind that the model for time to cover cracking provided uses transverse strains. In this research it was easy to identify transverse strains of 200 micro strains and corresponding mass loss of steel of 0.39% when cracking occurred because transverse strains were measured prior to corrosion initiation. For in-service structures, structural engineers may start measuring strains years after corrosion initiation. It is difficult then to know how much expansion of the cover concrete has already occurred. They may therefore over-predict the time to cover cracking. Changes in stiffness will however, more accurately provide how much corrosion has already occurred.

After a mass loss of steel of 3.8%, corrosion crack widths become the only reliable measurable parameter. Relation between corrosion crack widths and mass loss of steel in Figure 7.1c is for crack pattern B and under natural steel corrosion. Therefore, if structures exhibit crack pattern C, it is recommended that they be treated as previously discussed. Also note that it is more conservative to use crack pattern B than crack pattern A.

While patch repairs were found to prevent further steel corrosion and eliminate corrosion cracks (Figure 7.1c), they were not recommended because they cracked extensively at a slight increase in the applied load. They may later allow further steel corrosion to occur. In addition, they did not enhance the load-bearing capacity of corroded RC beams (Figure 7.1a). If only FRPs were used, further corrosion cracking with a similar rate as prior to repair was found. This cracking may cause early debonding of FRPs and extensively reduce the load-bearing capacity of structures. Since this notion was not proven, it is shown in Figure 7.1a as a dotted line. Similar to patch repairs, FRP repairs alone were not recommended. FRP with patch repairs increased the capacity of corroded structures and effectively prevented further steel corrosion.
7.7 Further research

This research provided important information on performance of RC beams that are corroded and repaired with patch and FRPs under various levels of sustained loads. From the scope as well as research findings, the following are principal further research works that need to be carried out on this broad research area;

1. Relations between corrosion crack widths and mass loss of steel during accelerated steel corrosion (despite using recommended current densities) were found to significantly differ with corresponding relations during natural steel corrosion, especially under high sustained loads. Unfortunately, natural steel corrosion used was limited, so that this relation needs to be proven on in-service structures. For future laboratory tests, the rate of natural steel corrosion (even under sustained loads as was shown in this research) is too low to yield appreciable structural response within a reasonable time frame. Therefore, a standardised test for accelerated corrosion in research laboratories, that is calibrated by results from natural steel tests under load needs to be established.

2. Results on cracking of the cover concrete were for RC beams with small bar spacings and where corrosion was limited to regions without stirrups. While this provided the worst scenario, research on cover cracking where stirrups are provided and large bar spacings are used is needed.

3. This research showed that under natural steel corrosion, corrosion crack widths are sensitive to changes in levels of sustained loads. This was however, established from few instances where applied loads were increased or reduced. The majority of results were from constant sustained loads. Therefore, there is need for research on performance of corroding and repaired RC structures under excessively varying loads.

4. Repair of RC beams was carried out at low corrosion levels which gave failure as yielding of steel in tension prior to cracking of concrete in compression. At high levels of steel corrosion, corroded steel bars may fracture instead of yielding. Further research on relation between load-bearing capacity and mass loss of steel at those corrosion levels is needed.
7.8 References


APPENDIX A

MEASURED AND PREDICTED MASS LOSS OF STEEL FROM FARADAY’S LAW

<table>
<thead>
<tr>
<th>Author(s)</th>
<th>Procedure for accelerated steel corrosion</th>
<th>Measured mass loss of steel (%)</th>
<th>Mass loss of steel from Faraday’s Law (%)</th>
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<td>Badawi and Soudki [34]</td>
<td>Concrete mixed with 2.25% chlorides by weight of cement. Specimens placed in 100% humidity chamber during corrosion. $i = 150 \mu A/cm^2$</td>
<td>5.9 5</td>
<td>8.7 10</td>
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<td>11.3 15</td>
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<td></td>
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<td>9.32 10</td>
<td>13.2 15</td>
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<td>3.87 4</td>
<td>4.31 4</td>
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<td>Mass loss of steel from Faraday’s Law (%)</td>
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<td>Cabrera [36]</td>
<td>Concrete mixed with 2% chlorides by weight of cement. Specimens immersed in 5% NaCl solution during corrosion.</td>
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<td>15 V: 9.8 6.2 3.8</td>
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<td></td>
<td>60 V: 15.4 9.8 8</td>
<td>60 V: 39.3 19.7 14.7</td>
</tr>
<tr>
<td>Azad et al. [42]</td>
<td>Concrete mixed with 2% NaCl by weight of cement. Specimens immersed in 5% NaCl solution during corrosion.</td>
<td>$i = 2000 \mu A/cm^2$: 5.4 15.2 21.5 5.5</td>
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<td></td>
<td>$i = 3000 \mu A/cm^2$: 20.1 22.9 8 8.9 7 9.1</td>
<td>$i = 3000 \mu A/cm^2$: 13.44 17.75 10.53 15.78</td>
</tr>
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<td></td>
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<tr>
<td>Ballim et al. [6,7]</td>
<td>Specimens were carbonated at a pressure of 80 kPa for 6 days. They were then immersed in 3% NaCl solution during corrosion. $i = 400 \mu A/cm^2$.</td>
<td>23% sustained load: 6.38 5.88 6.56</td>
<td>23% sustained load: 9.18 9.24 9.23</td>
</tr>
<tr>
<td></td>
<td></td>
<td>34% sustained load: 8.47 5.61 7.98</td>
<td>34% sustained load: 11.3 8.06 11.08</td>
</tr>
<tr>
<td>Author(s)</td>
<td>Procedure for accelerated steel corrosion</td>
<td>Measured mass loss of steel (%)</td>
<td>Mass loss of steel from Faraday’s Law (%)</td>
</tr>
<tr>
<td>-------------------</td>
<td>------------------------------------------------------------------------------------------------------------</td>
<td>--------------------------------</td>
<td>------------------------------------------</td>
</tr>
<tr>
<td>Yoon et al. [5]</td>
<td>Tensile face of specimens constantly wetted with 3% NaCl solution. $i = 370 \mu A/cm^2$.</td>
<td>75% pre-load: 3.2</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>45% pre-load: 3.8</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>45% sustained load: 4</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>75% sustained load: 6</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No load: 1.3</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>20% sustained load: 3.5</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>60% sustained load: 5</td>
<td>4.2</td>
</tr>
<tr>
<td>El Maaddawy et al. [8,9]</td>
<td>Concrete mixed with 2.25% chlorides by weight of cement. Concrete sprayed with mist during the corrosion process. $i = 150 \mu A/cm^2$. 60% sustained load.</td>
<td>9.7</td>
<td>6.06</td>
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<tr>
<td></td>
<td></td>
<td>15.4</td>
<td>13.32</td>
</tr>
<tr>
<td></td>
<td></td>
<td>22.8</td>
<td>25.43</td>
</tr>
<tr>
<td></td>
<td></td>
<td>30</td>
<td>37.54</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9.5</td>
<td>6.06</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.7</td>
<td>13.32</td>
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<td></td>
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<td>23.7</td>
<td>25.43</td>
</tr>
<tr>
<td></td>
<td></td>
<td>31</td>
<td>37.54</td>
</tr>
<tr>
<td>El Maaddawy et al. [60]</td>
<td>Concrete mixed with 3% NaCl by weight of cement. Specimens placed in a humidity chamber and constantly sprayed with fresh water mist during the corrosion process. Wrapped with FRPs.</td>
<td>15 V 6.7</td>
<td>8.6</td>
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<tr>
<td></td>
<td></td>
<td>5.2</td>
<td>3.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.8</td>
<td>2.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10.2</td>
<td>24.7</td>
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<tr>
<td></td>
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<td>8.6</td>
<td>15.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.8</td>
<td>10.3</td>
</tr>
<tr>
<td>Badawi and Soudki [34]</td>
<td>Concrete mixed with 2.25% Chlorides by weight of cement. Specimens placed in 100% humidity chamber during corrosion. $i = 150 \mu A/cm^2$. Wrapped with FRPs after corrosion of 5.9%.</td>
<td>7.4</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8.8</td>
<td>15</td>
</tr>
</tbody>
</table>
APPENDIX B

JOURNAL PUBLICATIONS


APPENDIX C

REFERENCES


