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Department of Civil Engineering

Improvement of the approaches to monitoring of reinforced concrete bridges subjected to chloride-induced reinforcement corrosion in bridge management systems

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Declaration

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Dedication

To my family for

their love and support.

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Foremost, I want to thank God for giving me the strength and courage to persist throughout my study. I could not do this without him. He provided a way where there seems to be no way; I will forever be grateful.

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Executive summary

Reinforcement corrosion in reinforced concrete (RC) bridges has undeniably become the main cause of civil infrastructure durability problems in many countries. The steel reinforcing in concrete corrode when certain conditions (e.g., carbonation, chlorides, insufficient cover and moisture) are met. Even though RC bridges are built to provide a service over a specified period, their serviceability changes over time due to gradual deterioration. In the marine environment, this deterioration is mainly caused by the ingress of chlorides in concrete, which results in chloride-induced steel corrosion. Thus, effective maintenance and management are needed to ensure the functionality of bridges throughout their designed service life.

Bridge management systems (BMSs) are effective means for managing bridges throughout their design life. BMSs require the collection of data related to the condition of the bridge for decision making. South Africa uses the Struman BMS to assess and prioritise bridges for maintenance and repair. The system solely relies on visual inspection as a basis for the identification of deterioration processes and bridge condition rating. However, visual inspections do not allow detection of rebar corrosion until damage has occurred; and once the damage has occurred, it becomes very costly to repair. Thus, this research focuses on identifying available reinforcement corrosion monitoring methods that can be used for condition assessment to supplement visual inspections in the Struman BMS.

The research involved reviewing different BMSs with the emphasis on their structure, application, and assessment methodology. The review of various BMSs used in the USA, Canada, UK and Southern Africa shows that while BMSs differ, they all assess the risk of failure and prioritise bridges for repair within limited budgets. Focusing on the Struman BMS, shortcomings with respect to the monitoring of reinforcement corrosion were identified. It was found that the condition ratings of RC bridges are based on visible defects assessed during visual inspections, which is still the dominant method used for bridge inspection. There exists a lack of integration of appropriate monitoring systems in the BMS, especially with regards to the monitoring of reinforcement corrosion.

Available corrosion monitoring methods were critically reviewed based on their principles, applications, and technical aspects. These corrosion monitoring methods were categorised into

visual inspections, Non-Destructive Testing (NDT), and remote monitoring methods. It became evident that monitoring technologies are continuously evolving and that the progress achieved to date is promising. However, further development is needed when using these methods in the BMSs for condition assessment, particularly in the continuous monitoring of bridges. The use of these modern technologies will allow earlier detection of problems and hence support sound maintenance and damage prevention. This is expected to simultaneously reduce the costs needed for maintenance and repair. In addition, modern monitoring methods have the potential to enhance the speed and scope of condition assessments, to provide reliable and wide-ranging data, and to reduce traffic interruptions when taking measurements.

An integrated approach is proposed based on applying corrosion monitoring methods as a supplement to visual inspections in the Struman BMS. Integrating these methods with the existing approach to bridge management is expected to compensate for the limitations of the Struman BMS and enhance its capability. The proposed approach involves identifying bridges that need to be assessed (new or existing bridges). Selecting monitoring methods for new bridges based on the needs and the type of data that need to be collected from bridges, and existing bridges based on the past assessment data and visual inspection. Monitoring is proposed to incorporate periodic and permanent monitoring. The former includes NDT measurements using any selected methods, and the latter includes installing sensor systems in new and existing bridges. Condition ratings are defined for monitored parameters, including corrosion risk (from corrosion potentials and electrical resistivity), corrosion rate, pulse velocity, defective areas (from the presence of cracks and delamination), moisture content and visual inspection. Other corrosion-related parameters considered include chloride content, cover depth measurement and monitoring time to corrosion onset. The obtained data from visual inspection and corrosion monitoring methods are then holistically analysed and interpreted to obtain integrated condition ratings, which are used to rate the condition of the bridge and its various RC elements. Consequently, this approach is expected to assist in prioritising critical bridges for repair and maintenance.

Lastly, recommendations are provided related to a comprehensive study on the implementation of available reinforcement corrosion monitoring methods for the monitoring of in-service bridges. The findings of this research also need to be taken forward by an experienced bridge inspector to help develop practical guidelines for applying corrosion monitoring methods in the Struman BMS.

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List of Acronyms

AASHTO	American Association of State Highway and Transportation Official
ACI	American Concrete Institute
ADC	Analogue-to-Digital Converter
AE	Acoustic Emission
Ag/AgCl	Silver/Silver Chloride Electrode
Ag/AgCl ISE	Silver/Silver Chloride Ion-Selective Electrode (Chloride sensor)
AHP	Analytical Hierarchy Process
ALS	Anode-ladder System
ASTM	American Society for Testing and Materials
BCI	Bridge Condition Index
BEADS	Bridge Expert Analysis and Decision Support
BHI	Bridge Health Index
BMS	Bridge Management Systems
BRIME	Bridge management in Europe
BS	British Standards
CD	Chain Drag
CI	Condition Index
CoMSIRU	Concrete Materials and Structural Integrity Research Unit
CoRe	Commonly Recognized Structural Elements
COTO	Committee of Transport Officials
CR	CorroRisk
CSE	Copper-Sulfate Electrode
CSIR	Council for Scientific and Industrial Research
CW	CorroWatch
DERU	Degree Extent Relevancy and Urgency
DOTs	Departments of Transportation
ER	Electrical Resistivity
ERS	Expansion Ring System
FHWA	Federal Highway Administration
GPR	Ground Penetrating Radar
HCP	Half-cell Potential
ICS	Integrated Condition States
IE	Impact Echo
Ir/IrO_2	Iridium/ Iridium Oxide Electrode (pH sensor)
IRT	Infrared Thermography
ISE	Ion Selective Electrodes
LPR	Linear Polarization Resistance

MMO	Metal-Metal Oxide
MnO_2	Manganese Oxide
MRE	Multi-Ring Electrode
MR&R	Maintenance, Repair, and Rehabilitation
NBI	National Bridge Inventory
NBIS	National Bridge Inspection Standards
NDT/E	Non-Destructive Testing/Evaluation
NICET	National Institute for Certification of Engineering Technologies
NSBMS	Nova Scotia Bridge Management System
OBMS	Ontario Bridge Management System
OCP	Open Circuit Potential
OSIM	Ontario Structure Inspection Manual
PE	Professional Engineer
QA	Quality Assurance
QBMS	Quebec Bridge Management System
QC	Quality Control
QFD	Quality Function Deployment
RABIT	Robotics Assisted Bridge Inspection Tool
RC	Reinforced concrete
RE	Reference Electrode
RILEM	International Union of Laboratories and Experts in Construction Materials, Systems and Structures
SACI	Structural Average Condition Index
SADC	Southern Africa Development Community
SANRAL	South Africa National Road Agency Limited
SANS	South African National Standards
SCE	Saturated-Calomel Electrode
SDPM	Structure Deduct-Points Method
SHM	Structural Health Monitoring
SPCI	Structural Priority Condition Index
SSI	Stewart Scott International
Struman BMS	Struman Bridge Management System
TMH19	Technical Methods for Highways – 19
TMH22	Technical Methods for Highways – 22
UAV	Unmanned Aerial Vehicle
UK	United Kingdom
UPV/W	Ultrasonic Pulse Velocity/ Waves
USA	United States of America
VI	Visual Inspection

Chapter 1

1 Introduction

1.1 Background

Reinforced concrete (RC) has been widely used as a construction material since the second half of the 19th Century for civil infrastructure such as bridges, tunnels, buildings, culverts, marine or water structures. Bridges constructed with this material are believed to have reliable structural performance, they are aesthetic, are adaptable in harsh environments, and are resilient against loads and impact [1]. RC bridges are typically designed to remain serviceable over a specified period (i.e., service life). However, the service life is sometimes reduced due to early deterioration and durability problems [2].

RC bridges are prone to deterioration due to mechanical, physical and chemical actions, and reinforcement corrosion [2]. These mechanisms are influenced by numerous factors, including environmental conditions, material quality, design, and construction [3]. Among these mechanisms, reinforcement corrosion has for many decades been a major problem for the durability of RC bridges in terms of assessment, maintenance, repair and rehabilitation, which has been increasing in ageing structures. This is a notable problem in the marine environment, where chloride-induced reinforcement corrosion can result in significant deterioration of the structure's condition. As a result, maintenance and repair costs constitute a major part of national infrastructure funding, which puts a tremendous economic burden on the country's budget.

Mbanjwa [4] indicated that bridges tend to increasingly deteriorate with time, which results in increasingly more resources required to maintain them at an acceptable level of service. In national budgets for infrastructure expenditure, allocation of these resources is now critical as there are limited and constrained funds for maintenance, repair, and rehabilitation in most countries, limiting activities in this area and leading to significant backlogs. An example of this is how South Africa experiences the challenge of having more government-owned bridges needing maintenance or repair and fewer bridges actually being attended to, due to limited budget [5]. Sometimes funds for bridge projects are reallocated to road projects because road failures are more common and more visible, even though bridge failures may be catastrophic when they do occur. Nordengen [6] indicated that in South Africa, only about 2.5% of the budget allocated to road rehabilitation projects is spent on bridge maintenance. Hence, this has led to more focus on preserving the limited national funds for existing infrastructure in need of remedial work rather than constructing new infrastructure.

In the past decades, transportation agencies have been trying to develop systematic approaches to bridge maintenance to ensure bridges remain safe and serviceable within limited budgets [7]. The initiation of the National Bridge Inspection Standards (NBIS) after the Silver Bridge collapsed in 1967 boosted the attention of different agencies to bridge inspections and maintenance in the United States of America (USA) [8]. Since then, bridge management systems (BMSs) have been developed first in the USA and are now practised globally. BMSs were developed to help manage, plan, and optimise allocated financial resources for bridge maintenance, repair, and rehabilitation.

In South Africa, the Struman BMS was initially developed and implemented in 1995 by the Division of Roads and Transport Technology of the Council for Scientific and Industrial Research (CSIR) in collaboration with Stewart Scott International (SSI) [6]. The Struman BMS was first developed for the Taiwan Area National Freeway Bureau and is currently implemented by various agencies in South Africa and neighbouring countries. The Struman BMS comprises inventory, inspection, condition, maintenance, and cost modules [5]. Among the five modules, the inspection module provides input data to the BMS on the condition of the bridges. This data is analysed and used to prioritise bridges for repair and rehabilitation within the limited budget.

For the Struman BMS, visual inspection is the predominant method used to assess and monitor the condition of bridges [4]. This method is used as a basis for the defect-based rating system, where defects on different structural elements are assessed visually and rated in terms of Degree, Extent, Relevancy and Urgency (DERU) using a scale of 1 to 4 [6]. The rating provides an indication of the condition of each bridge element inspected and prioritises the assessed structures in terms of their maintenance needs.

1.2 Problem statement

Monitoring the condition of bridges within their service life has become vital for their maintenance and management [9]. For the past decades, condition monitoring has been used to predict future performance and optimise maintenance and repair strategies. The main purpose is to ensure the functionality of bridges throughout their service life. The Struman BMS practises bridge assessments for prioritising bridges and their associated elements for maintenance [10]. However, in this system, condition ratings of bridge elements are based on visible defects assessed during visual inspections, which might not be adequate for assessing the actual risk of reinforcement corrosion. In addition, bridge elements in good conditions are not rated, which overlooks the possibility of deterioration in the initiation or beginning of the propagation stages.

The Struman BMS relies solely on visual inspection data for bridge condition assessments and prioritisation for maintenance and repair [4,11]. Bridge condition assessments emanating from visual inspections are critical in the BMS as they result in high costs spent on remedial actions and repairs of corrosion damage. However, visual inspections do not allow detection of rebar corrosion until significant damage such as steel reinforcing section loss and internal cracking has occurred. Further, once the damage has occurred, it becomes very costly to repair successfully. The development of successful remedial strategies and the associated allocation of available funds for maintenance projects depend highly on the completeness of the condition assessment and diagnosis process.

In the Struman BMS, visual inspections are carried out according to the Technical Methods for Highways (TMH19) manual, typically every five years. However, these do not adequately address the monitoring of reinforcement corrosion as assessment is only applied to visible defects such as surface cracks, spalls, and rust stains. In theory, reinforcement corrosion damage is typically visible on the concrete surface only after significant deterioration has occurred. At the time the damage depicts on the surface, it might be too late to design economical and long-lasting interventions. This indicates that a sufficient demand exists for methods to establish the condition of the structure before visible damage has occurred. With a proactive approach of using appropriate corrosion monitoring technologies, reinforcement corrosion can be determined early, so that severe damage can be prevented, and sound maintenance can be implemented.

Moreover, in the Struman BMS, principal inspections (involve comprehensive visual inspections of the whole bridge) are the minority in assessing bridge conditions. Monitoring is only considered on critical defects or when unexpected failures occur. There is a lack of integration of appropriate monitoring systems in the management of bridges, especially monitoring of reinforcement corrosion which is the widely noticed deterioration that affects the durability of RC structures.

1.3 Aim and significance

The research aims to identify monitoring methods for RC bridges affected by chloride-induced reinforcement corrosion, that can be included in the overall assessment, rating, and prioritisation of bridges for repair and maintenance in combination with visual inspections in the Struman BMS. The main purpose of doing this is to provide a proactive approach that will prevent agencies from only starting major repair once the damage has occurred. In this way the risk of damage can be quantified early enough so as to prevent severe damage.

The expected output of this research is a strategy for the inclusion of corrosion monitoring systems in the Struman BMS. Topics to be covered include the identification of suitable methods for monitoring reinforcement corrosion-affected structures, as well as guidelines on installation and data analysis. The outcome of this research is expected to help agencies improve the approaches to monitoring corrosion-affected RC structures. This will help to ensure that bridges remain functional and serviceable for the desired service life duration.

1.4 Research objectives

For the main aim of this research to be achieved, the following objectives are to be addressed:

- To review bridge management systems and identify the shortcomings with respect to monitoring of reinforcement corrosion in the Struman BMS.
- To determine available reinforcement corrosion monitoring methods and their potential and limitations.
- To evaluate how corrosion monitoring methods can be integrated into the Struman BMS.

1.5 Scope and limitations

The scope of this research is limited to RC bridges located in the coastal area (marine environment) of South Africa. It should be noted that the marine environment referred to in this study is the atmospheric exposure, of which bridges near to or on the coast are considered. This research focuses on RC bridges subjected to chloride-induced reinforcement corrosion and excludes any in-depth explanation of other deterioration mechanisms. The emphasis on the Struman BMS is placed on the assessment and prioritisation of RC bridges for maintenance and repair, excluding future predictions and financial aspects.

Furthermore, it was planned to verify some of the corrosion monitoring methods in-situ, but due to the pandemic, this was not possible. The research is limited to a literature review and informal industry review; hence, no experimental work is included. Also, the author of this research is not experienced in practical bridge inspection, therefore the knowledge developed in this research has to be taken forward in practice by a bridge inspector or someone who have necessary experience to develop detail guideline and application procedures.

1.6 Research outline

This dissertation will be presented as follows:

Chapter 1 – This chapter introduces the research by outlining the background and problem statement. The aim and objectives of the research are defined, with the research significance highlighted. This section also provides the limitation and scope of the research. The chapter then ended with the outline of all dissertation chapters.

Chapter 2 – The chapter presents an overview of durability in the marine environment. Since this research focuses on chloride-induced reinforcement corrosion, the discussion on the fundamental mechanisms by which this process occurs is crucial. Hence, the chapter provides comprehensive details on reinforcement corrosion, its mechanism and causes, and its effect on the service life of reinforced concrete structures.

Chapter 3 – This chapter provides a review of the current practices of BMSs in the USA, Canada and South Africa in terms of their structure, bridge condition assessment and bridge inspections. The emphasis is placed on the Struman BMS, which is the BMS structure used in South Africa and considered for this study. The shortcoming identified in the Struman BMS was also outlined in this chapter.

Chapter 4 – The chapter presents a comprehensive review of existing corrosion monitoring methods of reinforcement corrosion-affected RC bridges in the marine environment. Their principles, application, possibilities and limitations in relation to reinforcement corrosion assessment were outlined and discussed.

Chapter 5 – This chapter presents the proposed strategy of integrating corrosion monitoring methods in the Struman BMS. The chapter highlighted previous and related work on the integration of multiple technologies in condition assessment and BMSs. It also provides a guideline on how to select appropriate corrosion monitoring methods for a project or research.

Chapter 6 – The dissertation ends with this chapter, highlighting present research and its contributions to corrosion monitoring in the BMS. It also suggests recommendations for future research.

Chapter 2

2 Reinforcement corrosion in the marine environment

2.1 Overview

This chapter presents a review of reinforcement corrosion in the marine environment. It discusses the durability of reinforced concrete (RC) bridges in marine environments and presents a fundamental understanding of reinforcement corrosion (initiation and propagation), focusing on chloride-induced corrosion. The discussion highlights how reinforcement corrosion causes deterioration and how it impacts the performance of RC bridges.

2.2 Durability of RC bridges in the marine environment

The term durability relates to the capability of a structure or element to endure the design environment over the design life without unnecessary loss of serviceability or repair [2]. Plain concrete has been used in ancient structures as a very durable construction material, with some of the structures still standing. However, with its inability to withstand tensile stress when in service, it has become unsuitable for modern structures. Therefore, concrete is commonly used with steel reinforcement as an overall contributor to the structural capacity, though steel is susceptible to corrosion.

RC has been widely used to build various structures, including bridges, tunnels, dams and others. It is a durable material when properly designed and is adaptable to its environment, as the concrete cover depth acts as a protective barrier to the reinforcement. This is due to the highly alkaline nature of the cement paste that forms a passive film over the steel reinforcing to protect it from corroding [12].

However, structures located in marine environment are vulnerable to corrosion as the exposure to seawater may directly or indirectly cause reinforcement corrosion [13]. This deterioration is the major durability issue for RC bridges in marine environments, affecting both reinforcing steel and concrete. This mechanism is commonly caused by the diffusion of chloride ions and the carbonation mechanism [2]. The ingress of chlorides in concrete may result in chloride-induced corrosion in RC bridges which is of significant concern as it results in the pitting of reinforcement steel and an associated loss in structural capacity, as well as gradual damage in the concrete such as cracking, delamination and spalling.

The durability performance of the structure and materials depends on the environmental conditions to which the structure is exposed and the degree of resistance of the material to the aggressive substances [14]. Ballim et al. [2] refer to concrete durability as the interaction of concrete as a system and its environment. Factors of the concrete system affecting the capability of concrete to withstand deterioration include the intrinsic (e.g., binder type, water to cement ratio) and extrinsic (e.g., temperature, curing, and construction quality) factors. Furthermore, the presence of aggressive agents, e.g. chlorides and carbon dioxide, are regarded as environmental factors.

The degree of aggressiveness of marine environmental exposures influences the rate of deterioration due to corrosion [2]. Following the generally accepted convention, Shekarchi et al. [15] classify these exposures into the submerged, tidal, splash, and atmospheric zones, as shown in *Figure 2-1.* According to Moore [16], the submerged zone has high availability of chlorides and moisture, and low oxygen availability due to continuous immersion in seawater. The possibility of reinforcement corrosion occurring in this zone is thus very low. The tidal zone is constantly saturated with water, with high availability of chlorides due to alternating wetting and drying. This zone is also associated with limited exposure to oxygen; hence, low reinforcement corrosion typically occurs in this exposure zone [16]. The splash/spray zone experience splashes of high tide wave motion, which results in high exposure to chlorides, with enough moisture and oxygen. This zone is regarded as the most aggressive, where severe reinforcement corrosion occurs [16]. Lastly, the atmospheric zone is not in direct contact with sea water and therefore generally experiences lower chloride ion and moisture concentrations; hence moderate corrosion is experienced in this zone.

The marine environment exposure zone referred to in this study is the atmospheric exposure, of which bridges near to or on the coast are considered. Often, bridges are several km away from the sea but may still be experiencing chloride ingress; these are classified as XS1 by the European Standards [17], as shown in *Table 2-1*.

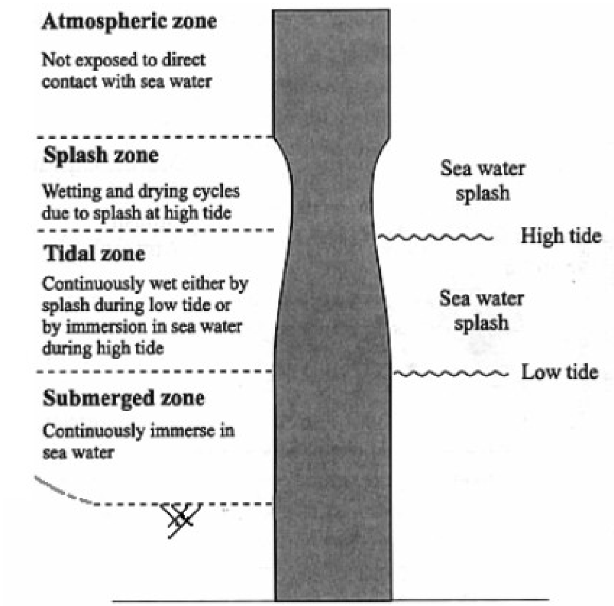


Figure 2-1: Exposure zones on marine structures [15]

Table 2-1 Exposure classes (adapted from Beushausen et al. [17])

Corrosion induced by chlorides from seawater		
Where concrete containing reinforcement or other embedded metal is subject to contact with chlorides from seawater or air carrying salt originating from seawater, the exposure shall be classified as follows:		
XS1	Exposed to air-borne salt but not in direct contact with seawater	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures

2.3 Reinforcement corrosion in the marine environment

Naturally, steel is prone to corrosion, but in RC, the alkaline solution in concrete pores stimulates the formation of a thin protective oxide on the surface of the steel (passivation), which prevents the steel from corroding, as shown in *Figure 2-2*. The passive layer is profound as it forms, maintains and repairs itself as long as the alkaline environment is present to regenerate it. This layer is believed to be ultrathin with <10 nm protective oxide or hydroxide film that decreases the dissolution rate of steel to negligible levels [18]. Though Poursaee [18] argued that within this passive layer, the inner oxide layer adjacent to the steel is protective as its rich in Fe^{2+}, and the outer layer mainly composed of Fe^{3+} is nonprotective. Hence when the chlorides contact the inner layer, they reduce its protective nature by converting the Fe^{2+} to Fe^{3+} oxide/hydroxides.

The steel passivity is preserved by high alkalinity of pH > 12.5 from a large amount of calcium hydroxide ($Ca(OH)_2$) in the cement paste. The concrete cover also provides extra protection by restricting access of aggressive agents to the reinforcement [19]. However, the steel passivity is not always maintained. The passive layer of steel can be broken down by the depassivation process, which is the destruction of the passive film of the steel by the aggressive agents. This destruction may be caused by the carbonation of concrete cover resulting in lower pH at the steel interface, the ingress of chloride ions to the steel level in concrete, or the combination of carbonation and ingress of chloride ions [9,11]. After depassivation, steel corrosion may occur depending on the availability of moisture, oxygen, chlorides, and carbon dioxide.

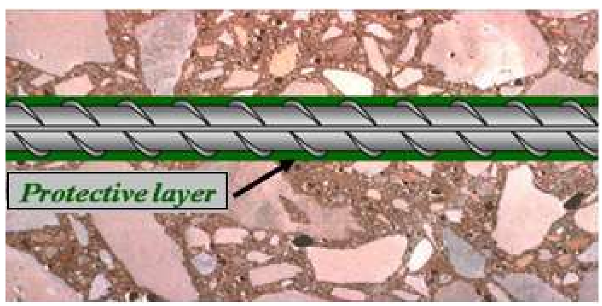


Figure 2-2: The passive film of the steel in concrete [20]

2.3.1 Mechanism of steel corrosion in concrete

Steel corrosion is an electrochemical process where the steel degrades at the anode, and oxygen is reduced at the cathode. This result in the flow of electrons between the anodic and cathodic site on the steel [21]. A corrosion cell consists of an anode where iron ions go into solution, the cathode, which is part of the steel where oxygen is reduced, an electrolyte (concrete pore solution), and a metallic path (steel) for connection between the anode and cathode [19].

The chemical reactions of the process are the same regardless of how corrosion occurred by chloride attack or carbonation. There are two well-known reactions, the oxidation and reduction reaction, that form the basis of the corrosion process. Initially, the steel at the anode is oxidised to form ferrous ions, which enter the surrounding solution on the steel surface, as illustrated in Equation 2.1.

$$Fe \rightarrow Fe^{2+} + 2e^{-} \qquad \text{Equation 2.1}$$

The released electrons then flow from the anodic site through the steel to the cathodic site. This results in oxygen reduction at the cathode, as shown in *Figure 2-3* [22] and illustrated in Equation 2.2.

$$2H_2O + O_2 + 4e^{-} \rightarrow 4OH^{-} \qquad \text{Equation 2.2}$$

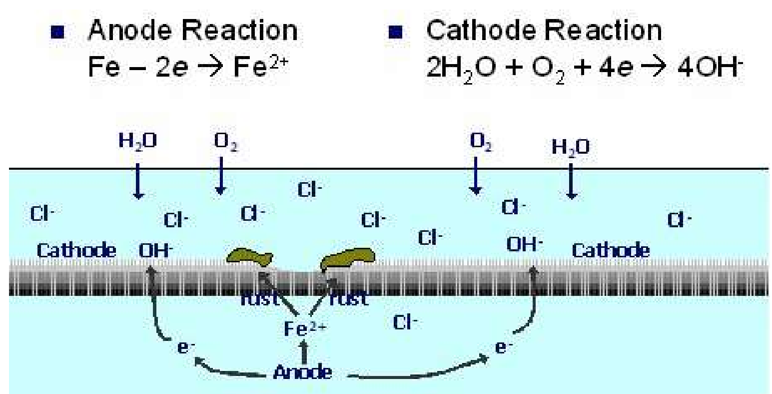


Figure 2-3: Mechanism of steel corrosion in concrete [22]

The cathodic reaction requires the presence of moisture and oxygen to proceed, as depicted in Equation 2.2. For this reason, RC structures in the dry environment usually show lower rates and extent of corrosion than the ones in humid conditions. However, this is not the case in submerged conditions, as corrosion is usually stifled due to a lack of oxygen.

2.3.2 Corrosion products and damage indicators

The anodic product Fe^{2+} from iron oxidation can result in the formation of corrosion products. When the Fe^{2+} from Equation 2.1 reacts with the cathodically formed hydroxyl ions in Equation

2.2, they produce ferrous hydroxide, as shown by Equation 2.3. The ferrous hydroxide further becomes ferric hydroxide and then hydrated ferric oxide (also known as rust), illustrated by Equation 2.4 and 2.5, respectively [21].

$$Fe^{2+} + 2OH^{-} \rightarrow Fe(OH)_2 \quad \text{Equation 2.3}$$

$$4Fe(OH)_2 + O_2 + 2H_2O \rightarrow 4Fe(OH)_3 \quad \text{Equation 2.4}$$

$$2Fe(OH)_3 \rightarrow Fe_2O_3H_2O + 2H_2O \quad \text{Equation 2.5}$$

The resulting oxides, hydroxides, or oxyhydroxides formed at the anode are known as corrosion products. These corrosion products take up a larger volume than the steel, as shown in *Figure 2-4*. Kumar et al. [23] indicated that corrosion products could expand and occupy a volume of about 6-10 greater than that of steel. It should be noted that the concrete will only expand when the porous region around the steel rebar has filled with corrosive products.

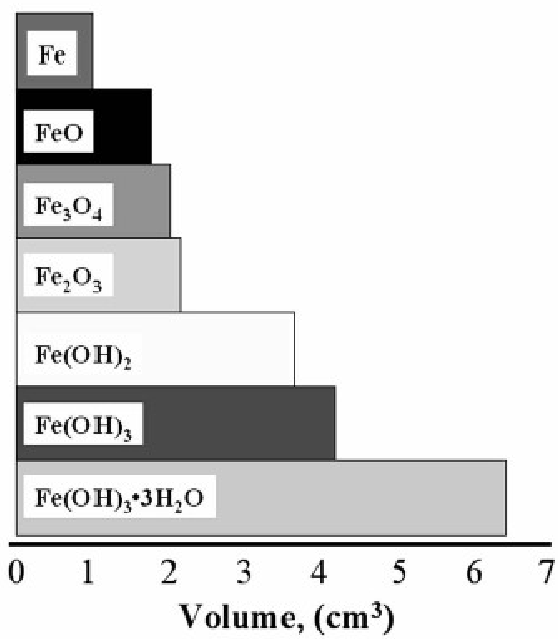


Figure 2-4 Relative volumes of iron and its corrosion products [19]

The corrosion products result in expansive forces that cause cracking and spalling of the concrete cover when the concrete tensile strength is exceeded [6,14]. According to Botes [24], the magnitude and development of stresses produced by the corroding steel are rarely quantified, and quantitative descriptions are still scarce. Therefore, volumetric increases by corrosion products can vary and influence potential cracking [16].

The resultant cracks and spalls are considered the most common indication of severe corrosion though they usually develop or are seen at the concrete's surface after the reinforcement has suffered significant loss [25]. In addition, the formation of cracks increases the vulnerability of concrete to the penetration of corrosion inducing agents, which further increases the corrosion [19]. Apart from cracking and spalling taken as corrosion indicators, rust staining is also considered a corrosion damage indicator. This has thus been considered as a basis for structural condition assessments.

2.4 Service life of corrosion-affected RC structures

The service life of a structure is a period during which the structure is used for its intended purpose with anticipated maintenance but without major repair [26]. This depends on the structure being able to withstand the deterioration mechanisms over the predictable period. Deterioration due to chloride-induced reinforcement corrosion in RC structures can be divided into three stages; initiation, propagation and acceleration, as shown in *Figure 2-5* [2,17].

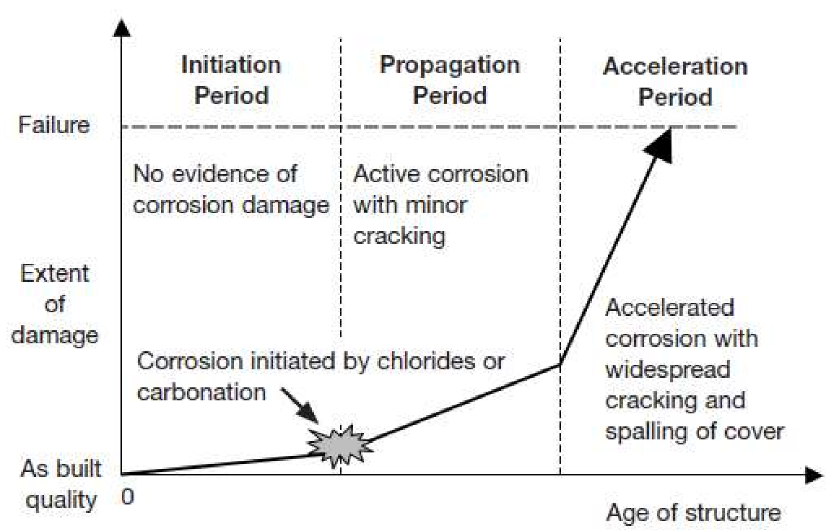


Figure 2-5: Three-phase corrosion damage model [2]

The initiation phase is the period during which the aggressive agents (chlorides or carbonation) penetrate the concrete cover and lead to the de-passivation of the steel embedded in concrete. Little or no damage occurs during this stage because the reinforcement is still in the passive state. The propagation phase is the period after de-passivation of the steel has occurred, and the steel experience active corrosion until the corrosion damage indicators such as cracks and spalls become visible and undesirable. The corrosion rate increases rapidly in the acceleration period due to the increased availability of corrosion sustaining agents (chlorides, moisture, and oxygen) through cracks and spalls [2]. The corrosion-induced damages are visible at this stage, and the concrete cover can no longer protect the steel. This may result in the structure being rehabilitated or replaced as it has grasped the end of its service life. Furthermore, deterioration of both the steel reinforcing and concrete compromises the integrity of RC bridges and typically reduces its load-bearing capacity. The initiation and propagation phase of deterioration are described in detail in the following section.

2.4.1 Corrosion initiation

Active steel corrosion is commonly initiated by carbonation or chloride ingress in RC. In the marine environment, the ingress of chloride ions is the foremost concern affecting the durability of RC bridges. According to James [27], chloride-induced corrosion may occur at a rate of 3 to 5 mm per year, while carbonation-induced corrosion typically occurs at about 0.05 mm per year. In addition, the combined action of both ingress of chloride ions and carbonation is known to increase the deterioration rate of the steel in concrete. This happens as the pH lowering by carbonation causes the release of chlorides chemically bonded to the cement, which increases the availability of free chlorides that cause de-passivation of the steel [2].

2.4.1.1 Carbonation-induced corrosion

Carbonation-induced corrosion occurs when carbon dioxide (CO_2) from the atmosphere diffuses into concrete and reacts with the cement hydration products (e.g., calcium hydroxide) in the presence of moisture to form calcium carbonate [10]. The CO_2 neutralise the alkalinity of concrete. Even though other gases (e.g., SO_2) can also neutralise alkalinity, their effects are minimal [27]. The chemical reaction of carbonation involves two processes [19]:

1. Carbon dioxide reacts with water in concrete pores to form carbonic acid, as illustrated in Equation 2.6.

$$CO_2 + H_2O \rightarrow H_2CO_3 \qquad \text{Equation 2.6}$$

2. Carbonic acid H_2CO_3 reacts with calcium hydroxide $Ca(OH)_2$ in concrete to form calcium carbonate $CaCO_3$, as shown in Equation 2.7.

$$Ca(OH)_2 + H_2CO_3 \rightarrow CaCO_3 + 2H_2O \qquad \text{Equation 2.7}$$

The reaction results in calcium carbonate that reduces the alkalinity of the concrete pore solution. This reaction progresses as carbonation front to the surface of the steel. For carbonated concrete, the pH value decreases from a value greater than 12 to between 8.5 and 9 [19]. Lower pH reduces the effectiveness of the passive firm of the steel, which makes it susceptible to corrosion. Thus, corrosion may initiate with the availability of oxygen and moisture.

Carbonation rate is affected by the concrete quality (cement type, water to cement ratio, cement content). The carbonation rate is increased by low cement content, high water to cement ratio (at or above 0.6), and high porosity of the concrete [10]. The relative humidity typically in the range of 40 - 70% also influences the carbonation rate [12]. In addition, carbonation does not occur in saturated concrete or dry conditions because CO_2 is not soluble in water and moisture is required for the reaction to occur [12].

Note: This research focuses on chloride-induced corrosion (explained in Section 2.4.1.2); hence more detail on carbonation-induced corrosion is not be provided.

2.4.1.2 Chloride-induced corrosion

Chloride-induced corrosion is initiated by the ingress of chloride ions in concrete, with the presence of moisture and oxygen. Chloride ions become available in concrete during construction from admixtures or accelerating agents containing calcium chloride and from the use of chloride contaminated aggregates in the concrete mix. The other common source of chlorides can be diffusion of chlorides due to exposure to seawater in marine areas, deicing salts and the use of chemicals [21]. The discussion in this dissertation will be more focused on the diffusion of chlorides into the concrete from seawater exposures.

The chloride ions are typically found in two forms: free or bound chlorides [28]. Free chlorides are present in concrete pore water, while bound chlorides are chemically bound to the hydrated cement paste. According to Otieno [19], it is believed that only free chlorides dissolved in the pore water are responsible for initiating corrosion. However, the concentration of free chlorides reduces with age due to chloride binding, which is the removal of chloride ions from pore water through interaction with the cement matrix. The chloride binding also presents a corrosion risk as it allows building up of higher chloride content, which increases the probability of corrosion. In addition, a fall in pH by bound chlorides may induce dissolution of the complexing phases, releasing free chloride ions; hence bound chlorides can also participate in the corrosion initiation [29].

2.4.1.2.1 Chloride attack mechanism

As described in Section 2.3.1, the presence of chlorides leads to the destruction of the passive film of the steel and initiate the process of localised corrosion. The chloride ions penetrate the concrete cover to the steel and act as a catalyst to corrosion when the chloride threshold level (see Section 2.4.1.2.2) is reached. The chloride ions are not consumed in the process, but help breaks down the passive film on the steel and allow the corrosion process to proceed rapidly. Once the chloride ions reach the steel level, they activate the surface of the steel to form an anode and the passivated surface being the cathode, which results in localised pitting. The breakdown of steel passive film by chloride ions illustrated in Equation 2.8 results in an unstable complex ($FeCl_3^-$), which can be drawn back into solution and reacts with hydroxyl ions to form $Fe(OH)_2$ and chloride ions Cl^-as shown in Equation 2.9 [19].

$$2Fe + 6Cl^- \leftrightarrow 2FeCl_3^- + 4e^- \qquad \text{Equation 2.8}$$

$$2FeCl_3^- + 4OH^- \leftrightarrow 2Fe(OH)_2 + 6Cl^- \qquad \text{Equation 2.9}$$

Chloride-induced corrosion is, therefore, a self-sustaining process whereby the chloride ions released lead to further depassivation of the steel. With sufficient moisture and oxygen, the deterioration process may propagate.

The process of how chlorides initiate the depassivation and its involvement in the passive film breakdown is still not fully understood. Montemor [29] explained the three models that are used to describe the passive film destruction by chloride ions, which are summarised as follows:

1. Adsorption-displacement – involves preferential adsorption of Cl^- with simultaneous dislocation of O^{2-} from the passive layer, leading to initiation of film destruction.

2. Chemico-mechanical – involves lowering interfacial steel/film surface tension by the chloride ions, which results in the formation of cracks and flaws when the repulsive forces between adsorbed ions are sufficiently large.
3. Migration-penetration – involves ion migration through the exchange process; when Cl^- reaches the steel, it occupies the O^{2-} vacancy leading to the formation of complexes with Fe^{2+}. The decrease of oxygen vacancies at the film/solution interface caused by Cl^- leads to the formation of voids due to faster iron dissolution, leading to pit growth.

Though debate of the above-mentioned models is still going on, it has been shown through electrochemical studies that sufficient concentrations of chloride ions are needed at the film/solution interface to initiate the depassivation process [11,22,23]. This threshold concentration increases with the pH of the pore solution, and it is still unknown why the threshold concentration is pH-dependent.

2.4.1.2.2 Chloride threshold level

Otieno [19] defines the chloride threshold level as the concentration of chlorides at the steel depth necessary to sustain the steel depassivation and initiate the corrosion process. The chloride threshold level is an essential influence on the service life of concrete structures exposed to marine environments. Generally, this threshold level can be presented by free chloride content, the ratio of chloride to hydroxyl ions (Cl^- / OH^-), or the percentage of total chloride content relative to the weight of cement.

Free chloride content has been regarded as the best representation of the chloride threshold level under the assumption that bound chlorides are relatively immobile and may not be transported to the steel surface [31]. This has been challenged by current thinking, considering that when the pH-value drops due to depassivation, the bound chlorides at the steel depth are released to form free chlorides, and cement hydration products (e.g., calcium hydroxide) resist the fall in pH value at a particular level. The chloride threshold level may not be well represented by the ratio of (Cl^- / OH) as it ignores the inhibitive effect of the cement matrix, which may include a relative denser hydration product layer on the steel surface and it does not consider the dependence of chloride binding capacity on the hydroxyl concentration [24,25]. Representing chloride threshold with total chloride is the most widely used approach, and standards adopt it. This is because it is easy to determine and involves the corrosion risk of bound chloride and the inhibitive effect of cement hydration products.

The chloride threshold level is unique for each structure and depends on the pH of the pore solution, the penetrability of the concrete, and the electrochemical potential of the steel [12]. According to Alexander et al. [12], the most widely used chloride threshold value varies from 0.02 to 3.08 % total chloride by mass of binder. Montermor [29] indicated that total chloride content ranges between 0.17% and 2.5% by weight of cement. Song and Ann [31] mentioned that the conservative value of 0.2% or 0.4% by weight of cement had been used to predict corrosion-free life. *Table 2-2* summarises the maximum limit of total chloride content adopted by the American Concrete Institute (ACI) and British Standards (BS)/European Standards (EN).

Table 2-2: Chloride content values set by BS/EN and ACI standards [19]

Type of element	**Maximum chloride content (%, cem.)**			
	BS EN 206-1	**ACI 201**	**ACI 357**	**ACI 222**
Prestressed concrete	0.10	-	0.06	0.08
Reinforced concrete exposed to chloride in service	0.20	0.10	0.10	0.20
Reinforced concrete that will be dry or protected from moisture in service	0.40	-	-	-
Other reinforced concrete	-	0.15	-	-

A wide range of chloride threshold values has been noticed in a considerable amount of research, which shows much variation. The chloride concentration vary significantly even at a constant depth, making it possible for the chloride threshold level to be over or underestimated [33]. This may be due to different measurement methods, methods of presentation, the condition of the steel/concrete interface and the influence of environmental factors. Gartner et al. [33] indicated that the most significant influencing parameter is the steel/concrete interface, which influences the local susceptibility of the reinforcement to corrosion initiation. Stainless steel is considered the best for RC structures exposed to chloride environments as it can have up to 10 times higher critical threshold level than carbon steel; hence it can be preferably used.

2.4.2 Corrosion propagation

Corrosion propagation of steel is a stage at which steel corrosion occurs at a rate after the steel depassivates until complete disintegration of the steel. The phase starts as chloride-induced depassivation of the steel surface has occurred and ends with corrosion-induced damages if the corrosion agents such as oxygen, moisture and chloride ions are continually available at the corrosion site. The corrosion-induced damages result from the expansive stresses caused by a substantial volume of corrosion products on the steel. This results in cracking of concrete cover, spalling, delamination, and loss of bond strength between the concrete and steel.

Corrosion propagation is usually characterised by corrosion rate, which is defined as the loss of steel per unit surface area per unit time. There are few predictions of corrosion rate as shown in *Table 2-3* [24]. However, in most studies, the corrosion rate is taken as a constant value of 1 $\mu A/cm^2$, with the transition from passive to active corrosion taken as 0.1 $\mu A/cm^2$ [17,32]. A corrosion current density of 1 $\mu A/cm^2$ is considered to be equivalent to a steel loss of 12 µm/year, which is deemed too high and likely to result in cracking and spalling within one year [34].

Table 2-3 Corrosion states based on the corrosion rate measurements [24]

Estimated corrosion rate ($\mu A/cm^2$)	Corrosion state
0 – 0.1	Passive
0.1 – 0.5	Low
0.5 – 1.0	Moderate
>1.0	High

According to Alexander et al. [12], the propagation phase is considered by active corrosion with rates above 0.1 $\mu A/cm^2$; this goes in hand with what is depicted in *Table 2-3*. High corrosion rates are often related to high temperatures between 25 and 40 °C, moisture availability, and optimal relative humidity around 90 to 95% [35]. Other factors that might affect corrosion propagation include; w/c ratio, binder type, concrete cover, availability of oxygen, the concentration of the chemicals, presence of cracks, and the electrochemical resistance of the concrete [10,17].

The length for the propagation phase depends on the pre-defined limit state and the related damage indicators [19]. The limit state indicator can be selected from the range of corrosion damage indicators, including loss of steel cross-sectional area, the appearance of cracks, loss of steel-concrete interface bond, loss of a member's ultimate load capacity, and loss of flexural stiffness. The corrosion-induced damages are not usually visible in the initiation stage, although in the propagation stage, there might be clear indicators of damage in the structure in the form of cracks, spalls, or rust staining. These are typically used as a basis for corrosion assessments [25]. Because some RC bridges are already in the propagation phase of corrosion, with or without visible damage indicators, RC bridges are assessed and monitored for proper maintenance and repair.

2.4.3 Factors affecting the deterioration rate

Structures deteriorate over time, and the rate of deterioration is a function of various parameters. As mentioned previously, binder type, concrete resistivity, w/c ratio, concrete cover depth and quality, and concentration of aggressive agents influence the corrosion initiation and propagation. Otieno [36] reviewed the effect of these factors on the rate of chloride-induced corrosion. Factors such as binder type, cover concrete quality and depth, and w/c ratio are typically considered at the design and construction stage. However, they influence the deterioration of a structure in different ways during its service life. Different binder types, e.g. supplementary cementitious materials and blended cement, limit the corrosion rate by densifying the concrete microstructure and promoting chloride binding [19]. Concrete resistance and quality (depend on the w/c ratio and binder type), when increased it suppresses the corrosion rate by reducing the penetrability of corrosion agents (chlorides, moisture, oxygen) to the steel. On the other hand, the cover depth governs the travel path of these corrosion agents [12]. These factors thus need to be carefully chosen to avoid the aftermath effects during the service life.

Other essential factors considered during the service life of a structure include the environmental condition, presence of cracks, age of the structure, and the quality of inspection and monitoring [37]. Constant exposure to aggressive agents (e.g., Chlorides) and the presence of cracks speed up the corrosion rate. The cracks allow the ingress of chlorides freely to the steel reinforcement, depending on its crack widths, frequency and orientation to the reinforcement [22]. Mbanjwa [4] indicated that RC structures worsen in condition with age increase and tend to show predominant cracking, spalling and reinforcement corrosion defects. Verma [38] added that the deterioration rate is attributed mainly to the environmental exposures and quality of designs, construction, and maintenance. Furthermore, if proper inspection and monitoring of reinforcement corrosion are not carried out, then no interventions could be done to suppress or stop the corrosion process. Hence, resulting in detrimental effects on the durability and serviceability of such structures.

2.5 Summary

This chapter presented a review of the aspects of the durability of RC structures affected by reinforcement corrosion. This type of deterioration is the most critical durability issue for RC structures, affecting both the reinforcing steel and the concrete. The review progresses with the fundamentals of reinforcement and mechanisms of reinforcement corrosion, which is commonly caused by the diffusion of chloride ions and carbonation of concrete. The emphasis was placed on chloride-induced reinforcement corrosion, which is of significant concern in the marine environment as it results in the pitting of reinforcement steel, rust staining on the concrete surface, and gradual corrosion damage such as cracking, delamination, and spalling. These visually evident signs of corrosion damage are considered the most common indicators for reinforcement corrosion. However, they usually develop and become visible at the concrete's surface only after the reinforcement has experienced significant corrosion, which often is associated with a loss in steel cross-section and damage to the concrete surface. The reinforcement corrosion progresses in two stages after the initiation - the propagation and acceleration stages. These stages are influenced by different factors, including binder type, concrete resistivity, w/b ratio, concrete cover depth and quality, availability of corrosion sustaining agents (chlorides, moisture, and oxygen), the presence of cracks, and the age of the structure.

Chapter 3

3 Bridge Management Systems

3.1 Introduction

Bridge management is a means of controlling bridges from conception to the end of service life and ensures bridge safety and functionality. Dinh [39] defines a bridge management system (BMS) as a reasonable and systematic way to organise and carry out all tasks linked to bridge maintenance. The system employs a wide range of activities that are commonly encountered in the day-to-day management of bridges. Such activities begin once the bridge is commissioned and entail inventory data collection, regular inspections, assessment of bridge condition, repair and maintenance, prioritisation of funds, and ensuring the safety of users. Bridge management practices thus aim for all bridges to achieve their design life, such that they remain open to traffic and their risk of failure is reduced.

Prior to the 1980s, engineers kept records of bridges in handwritten registers as part of bridge management [40]. This system used to work for some developing countries where there were relatively few bridges to manage. However, since the 1980s, there has been an increase in the number of bridges to manage, an increase in deteriorating bridges and occasional bridge-related fatal events (e.g. bridges collapsing due to delayed or neglected maintenance). Hence, this has resulted in the development of BMSs, which effectively manage a network of bridges. These systems are advantageous as they process abundant data and save time compared to the paper system [40].

The main goal of BMSs is to assist bridge engineers and managers to make decisions on the selection and planning of bridge maintenance, repair, and rehabilitation (MR&R). This is done in such a way that the allocation of financial resources is optimised, and the safety of bridge users is increased [6]. The system applies to both project and network-level bridge management, where the former focuses on managing individual bridges and the latter concentrates on managing a network of bridges. However, different agencies focus on obtaining benefits from the whole network rather than individual bridges [39].

3.2 Bridge management systems components

Every BMS contains several essential components that enable it to function as a fully integrated system capable of storing and analysing data, promoting the interaction of components, and integration of new data. Though individual BMSs have different components, Ryall [40] suggested that BMSs include five main components or modules: Inventory, Inspection, Bridge Condition, Maintenance, and Cost, as illustrated in *Figure 3-1*, which perform unique functions presented in *Table 3-1*. The five components make up the database for the entire system, where information is processed and analysed by the management control (also known as an administrative module, that deals with changes in other modules) to select alternative maintenance actions. The maintenance actions are then implemented and bring new data (e.g., the present condition of bridges and type of remedial work carried out) to the database for updates.

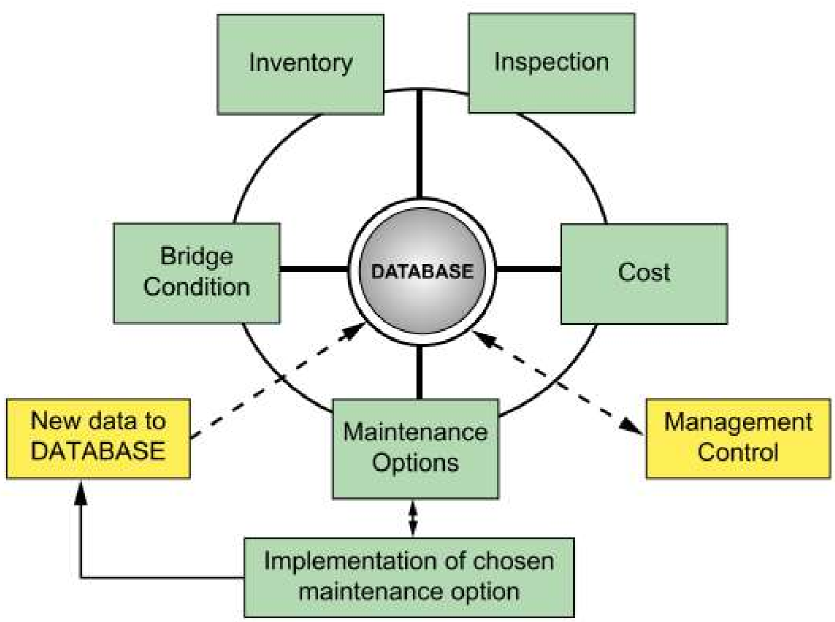


Figure 3-1: Typical components of a Bridge Management System [40]

Table 3-1: BMS modules description and requirements[adapted from Ryall [40]]

BMS Modules	Function	Requirements
Inventory	Stores all information about the bridge: name, location, construction, dates, structural details, design characters, and maintenance history.	Drawings, maintenance records, site visit, design data, traffic volume, record cards, and individual reports (of all activity updates)
Inspection	Stores information from the inspection reports including, the condition of the bridge, specified treatment, priorities given to past remedial work and the associated cost.	Detailed inspection drawings and inspection reports
Cost	Processes all the cost information from past and present projects.	All the bridge expenditures
Maintenance	Avails information on the nature of maintenance carried out on the bridges in the form of sketches, photographs and final costs	Maintenance records of bridges
Condition	Allocates priority to bridges based on the nature of their defects or condition	Historical and present condition data

According to the BRIME (Bridge management in Europe) report [41], BMSs are considered to have six modules, as shown in *Figure 3-2*. These modules basically represent the steps followed in the implementation of a BMS in Europe. The modules are based on the input parameters of the condition and load-carrying capacity, which are required for maintenance optimisation. The output parameters of deterioration rates and cost of maintenance options provide a basis for optimum maintenance programs within limited budgets. These basic characteristics are in relation to the five essential BMS components that Ryall suggested, which were discussed previously.

Apart from the BMSs with components or modules outlined above (in *Figure 3-1* and *Figure 3-2*) there have been many BMSs developed worldwide. These BMSs varies in structure, principles, decision-making and assessment methodology; however, their applications are similar. They all have principal objectives of guaranteeing safety to the bridge users, ensuring that bridges reach their targeted level of service and remain in service for a long time without or with minimal maintenance.

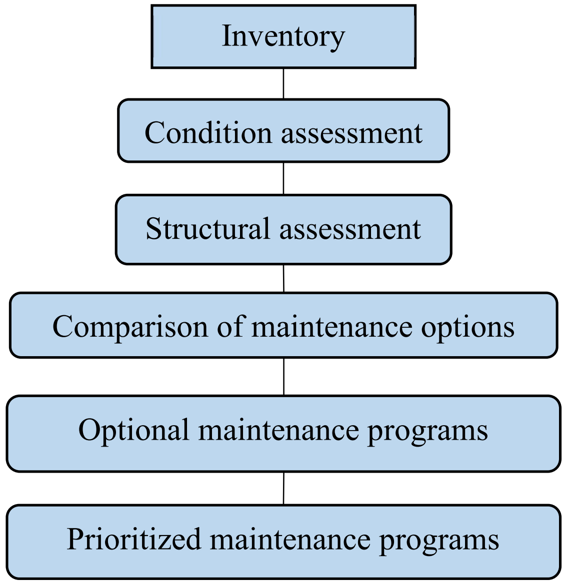


Figure 3-2: Schematic diagram for a BMS in Europe [41]

3.3 Current practice in Bridge Management Systems

As previously mentioned, BMSs differ in structure and principles. Various BMSs exist worldwide, and a comprehensive review of all of them is not considered necessary for this discussion. Consequently, only practices in existing BMSs of the United States of America (USA), Canada and South Africa are described and discussed. The purpose is to identify features existing in various BMSs and to provide an overview of what constitutes these BMSs in different countries. Emphasis is placed on the BMS structures and their approaches to dealing with the inspection data. In addition, the approach to bridge inspections is discussed in Section 3.4.

3.3.1 BMSs in the USA

In the USA, the Federal Highway Administration (FHWA) and the American Association of State Highway and Transportation Official (AASHTO) play an essential role in bridge management; for example, they initiated the development of BMSs in 1987 [40]. Although the development of BMSs was optional, many States opted to implement BMSs since the 1990s, with the Pontis BMS (now known as AASHTOWare Bridge Management) gaining popularity. This BMS is currently used by most State Departments of Transportation (DOTs) and transportation organisations across the USA [34,35]. According to Transportation Research Board [8], forty-two States have implemented the Pontis BMS, some use the Bridgit BMS, and only a few preferred developing their own BMSs (e.g. Pennsylvania, Alabama, New York, and North Carolina, to name a few).

3.3.1.1 The Pontis BMS

As mentioned above, the Pontis BMS is one of the most commonly used BMS in the USA. The Pontis BMS was developed after the FHWA identified the widening difference between the funds needed for necessary treatments of bridges and the available allocated budgets [43]. This BMS is set as a solution to assist engineers, bridge managers, and decision-makers in selecting and planning maintenance, repair, and rehabilitation (MR&R) of bridge structures within limited funds.

Figure 3-3 shows the structure of the Pontis BMS. Its database uses information mainly from inventory collection and condition surveys of the bridge network. Inventory data includes unchangeable details about the bridge, and the condition surveys are done periodically as bridge inspections. The bridge inspections generally involve visual inspections or the use of NDT to determine the condition of the bridges. However, agencies typically focus on visual inspections for their assessment [39].

During inspections, the inspectors assess the condition of each bridge element based on the defects detected and indicate the feasible actions or treatment to be undertaken. The condition states are considered as quantitative measures for deterioration. The inspection data are then used to predict the future condition of each element using the deterioration models. The Pontis BMS models the deterioration of bridge elements as a Markov process, a probability-based model for estimating bridge condition changes over time [43]. The deterioration models and the cost of each feasible treatment are then used as input for the MR&R optimisation model to obtain the long-lasting measures.

In addition, the Pontis BMS has an improvement action, used when the bridge is functionally obsolete [39]. A bridge is considered functionally obsolete when one of its elements is rated with a poor condition. This improvement action enhances the level-of-service of the bridge by reducing the cost to the end-user, making the bridge safe for traffic and reducing travelling time. The user cost model in the Pontis BMS can thus provide an estimated cost for accidents and user costs from travelling time and detours [35,37]. This information enables trade-offs between options in the improvement optimisation model. The optimised MR&R and improvement actions are integrated as project programs and prioritised depending on the bridges that need urgent actions. The system thus keeps track of current conditions, predicts future conditions, identifies project and network-level needs, and makes project recommendations to gain maximum benefits from limited funds [8].

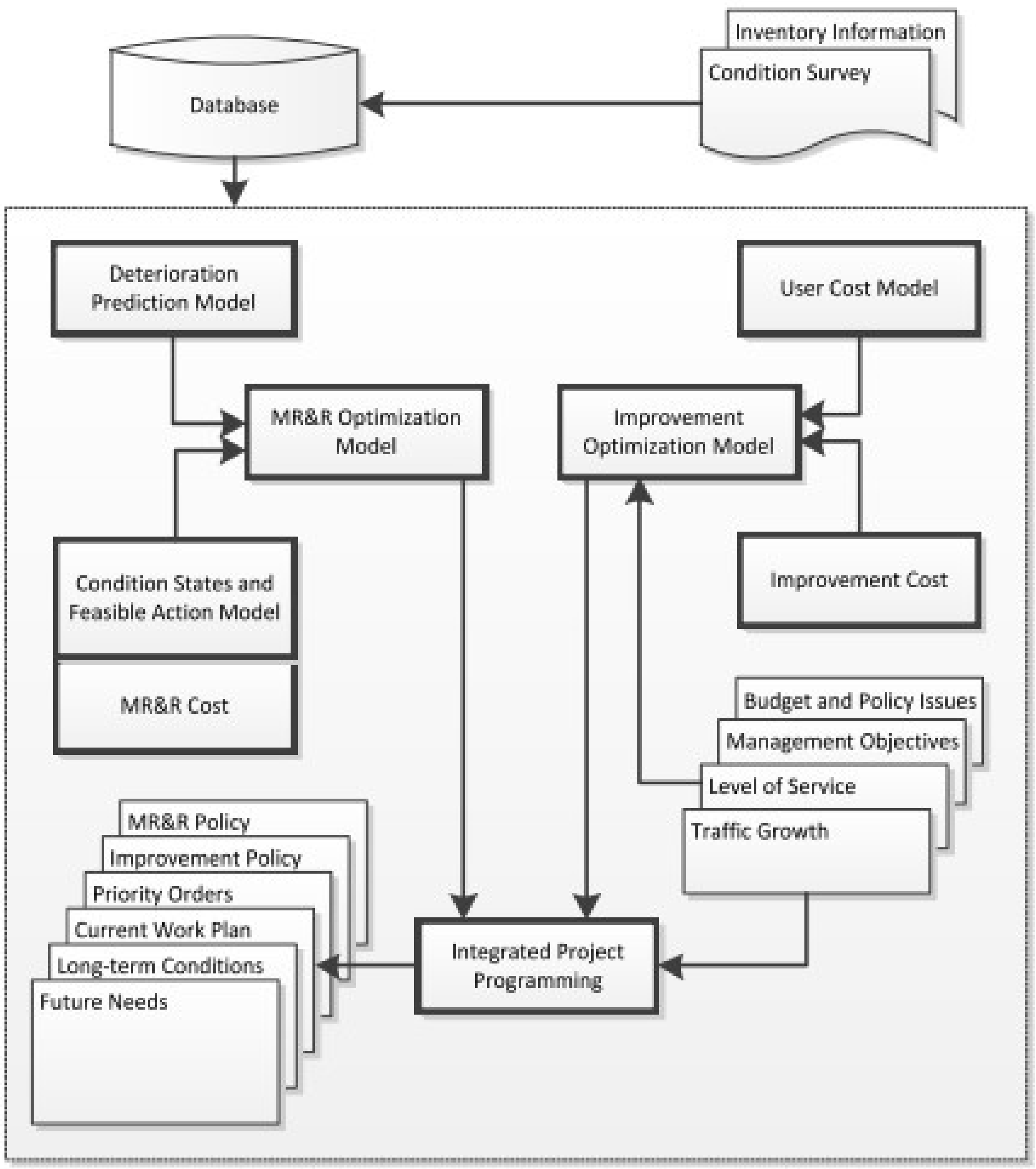


Figure 3-3: The Pontis BMS structure [39]

3.3.1.2 Bridge condition in Pontis BMS

Bridge condition in the Pontis BMS is determined in terms of condition ratings. Condition ratings are values allocated to defects based on inspectors' decisions on the severity of the defect, and they determine the physical condition of the bridge elements [24]. The ratings are to provide simplicity in the direct use of inspection data by transportation agencies for decision making. There are two commonly used bridge condition rating systems in the USA, the National Bridge Inventory (NBI) and Pontis. The third one used is the Bridge Health Index (BHI), based on Pontis BMS [44].

All national data on bridge conditions are held by the NBI, a record of data reported to FHWA from the States, Federal agencies, and Tribal governments on the condition of the Nation's bridges [42]. Prior to implementing the Pontis BMS, the USA followed the *National Bridge Inspection Standards (NBIS)* for bridge inspections. These standards were established to create a uniform program that governs the minimum requirements for bridge inspection and inventory reporting for all State DOTs [45]. The Pontis BMS was designed with a new standard named "*Commonly Recognised Structural Elements (CoRe)"* and is now revised to *Bridge Element Inspection* [46]. Both standards are used in the USA, each State with its preferences.

As per NBIS, three main categories of items are considered, inventory, condition rating and appraisal rating items [8]; this data are collected from every State annually and aggregated in the NBI system. Inventory outlines the information about the bridge being inspected, condition rating evaluates the bridge's current condition compared to its new condition, and appraisal rating indicates the effect of the rated element on the bridge as a unit. The bridge condition is based on four defined elements the deck, superstructure, substructure and culverts. The NBIS condition and appraisal rating range from 0 to 9, the bridge being excellent with 9-point, poor when below 4-point, and absolute failure with 0-point [36,41]. *Table 3-2* shows the descriptions of both condition and appraisal rating.

The Pontis standard employs the element-level condition inspections on the element set consisting of National Bridge Elements and Bridge Management Elements depicted in *Table 3-3* [46]. The standard also allows agencies to define their own element without ties to the elements defined in the standards. Pontis inspections thus provide the agencies with more detailed condition data than the earlier minimum requirement of NBIS. For example, instead of rating the whole superstructure condition (NBIS case), an element-level inspection considers the condition of individual components of the superstructure, such as girders, floor beams, pins, hangers, bearings, etc. The standard employs different Condition States/ratings of 1 to 4, with general descriptions shown in *Table 3-4* [46]. The Condition States are allocated to the bridge elements based on the visibility of pre-defined defects. Both severity and extent (in terms of percentage) of deterioration of an element are indicated. If multiple defects appear in the same vicinity, the inspector reports the most severe Condition State. If two Condition States operate at the same defined space, the inspector determines the predominant defect for reporting [46].

Table 3-2: NBI Condition and Appraisal rating guidelines [36,41]

Rating	Bridge condition	Appraisal
N	Not applicable	Not applicable
9	Excellent condition	Superior to present desirable criteria
8	Very good condition (no problems noted)	Equal to present desirable criteria
7	Good condition (some minor problems)	Better than present minimum criteria
6	Satisfactory condition (minor deterioration in structural elements)	Equal to present minimum criteria
5	Fair condition (sound structural elements with minor section loss, deterioration and spalling)	Somewhat better than minimum adequacy to tolerate being left in place as is
4	Poor condition (advanced section loss, deterioration and spalling)	Meets minimum tolerable limits, to be left in place as is
3	Serious condition (affected structural elements from section loss, deterioration and spalling)	Basically intolerable, requiring high priority of repair
2	Critical condition (advanced deterioration of structural elements – bridge need to be closed)	Basically intolerable, requiring high priority of replacement
1	“Imminent” failure condition (major deterioration affecting structural stability – bridge is closed)	Rating code not used
0	Failed condition (out of service)	Bridge closed

Table 3-3: The bridge element levels for inspection [adapted from AASHTO (35)]

National Bridge Elements		Bridge Management Elements	
Major element grouping	**Sub-elements number**	**Major element grouping**	**Sub-elements number**
Decks and Slabs	9	Joints	7
Superstructure	9	Approach slabs	2
Substructure	7	Wearing surfaces, protective coatings and reinforcing steel protective systems	4
Railings	5		
Bearings	7		
Culverts	1		

Table 3-4: Pontis bridge condition rating [adapted from AASHTO (35)]

Condition State/ rating	Description	Weight factors
1	Good	1
2	Fair	0.67
3	Poor	0.33
4	Severe	0

3.3.1.3 Bridge health index

The Bridge Health or Condition Index (BHI or BCI) is determined from the Condition States allocated to each bridge element through the condition rating system. This is one of the performance measures used to assess the structural or functional health of a bridge or network of bridges [35,37]. The index identifies the most deteriorated bridges within the inventory that need urgent repairs, hence prioritising bridges for maintenance. The BHI provide ratings of structural health between 0 - 100, with 100% indicating the best state and 0% indicating the worst [45]. It also helps keep track of the condition of the bridges over time, which act as a basis for resources allocation to bridges within the network and help agencies plan for MR&R programs [44]. The BHI uses the element-level inspection data from the Pontis BMS and is calculated as the ratio of the combined current condition value of the bridge elements to the total initial condition value of the bridge elements as illustrated in Equation 3.1 [31,38].

$$BHI = \frac{\sum CEV}{\sum TEV} * 100 \qquad \text{Equation 3.1}$$

Where: Current Element Value (CEV) = $\sum (Q_i * WF_i) * FC$

Total Element Value (TEV) = Total Element Quantity *Failure Cost (FC)

Weight Factor of condition i (WF_i) = $1 - \frac{Condition\ State\ Number - 1}{Number\ of\ Condition\ States - 1}$

Q_i is the quantity in Condition State i, where i can be any of the Condition State indicated in *Table 3-4*. The quantities are typically noted as the extent of deterioration, which is the percentages of condition in each Condition States during inspections. The aggregation of the index at a bridge or network level is thus dependent on the weighting factor and failure cost of each element. The weight factors for each Condition State in a 4-point scale used in Pontis BMS are indicated in *Table 3-4*. This is relative to the failure costs of the elements, which is the economic consequence of the element's failure with respect to the overall bridge health [31,38]. Thus, elements with failure of little economic effect will receive less weighting than safety threatening elements. The failure cost of each element is estimated from the agency and users cost (e.g. operation costs, inspection costs, traffic flow delays), or it can be estimated by expert Bridge Engineers [44].

This calculation is based on the idea that the initial value of the bridges when commissioned depreciates due to deterioration, which results from different factors such as environmental conditions and traffic loading. However, with the interventions of feasible MR&R, the BHI can be improved and hence the condition of the bridges as well.

3.3.2 BMSs in Canada

Canada is facing critical problems dealing with the complexity and fragmentation of the current infrastructure management due to the ageing of infrastructures [43]. More than 40% of the bridges in the country were built over 50 years ago and are in the need stage for repair, rehabilitation and replacement. However, maintenance budgets are very limited [43]. As in other countries, different agencies in Canada utilise the BMS to manage their bridges effectively and ensure bridge safety within the limited maintenance funds. Some of the BMSs used in Canada include the BMS for Ontario (OBMS), Quebec (QBMS), Nova Scotia (NSBMS), and Alberta (BEADS); some provinces use the USA Pontis BMS. All these BMSs differ in terms of architecture, functionalities and interfaces [31,35,37,39]. However, the OBMS is widely used in Canada, which is discussed in this section.

3.3.2.1 Ontario BMS

The Ontario Ministry of Transportation developed the OBMS in 1998 and deployed it in 2000 [48]. The OBMS uses both network-level and project-level approaches to decision-making, favouring the project-level functionality. The decision-making process served by the system includes components shown in *Figure 3-4*, which are dependent on each other. The project-level starts with identifying individual bridge element needs based on recent inspections. These inspection data are considered the primary source for the decision-making, stored in the OBMS database, and used to generate condition ratings for bridge elements.

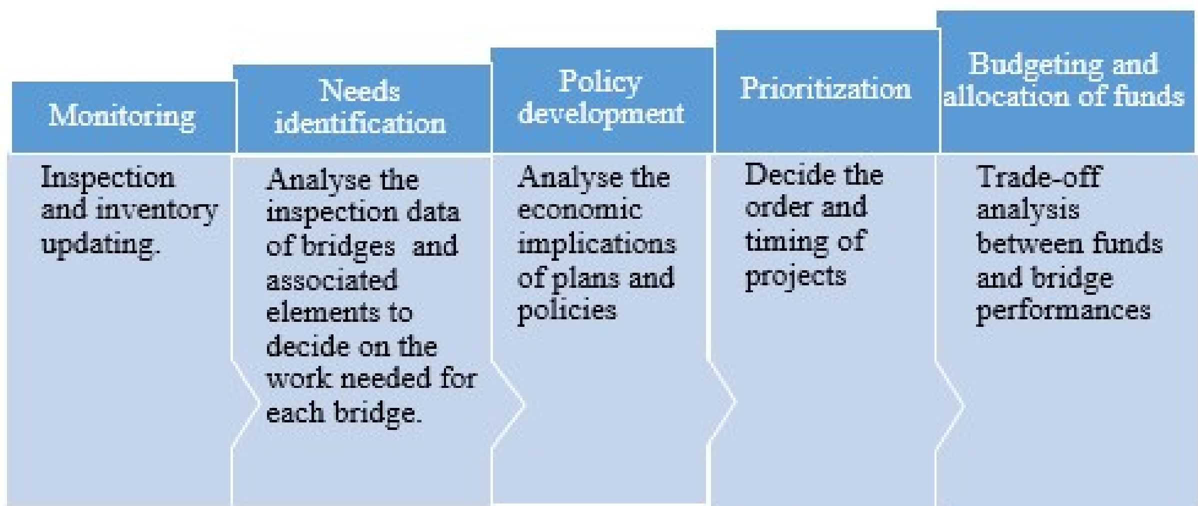


Figure 3-4: Decision process in the OBMS [adapted from Thompson [48]]

Basically, the OBMS comprises three main models: Knowledge Model, Deterioration Model, and Cost Model [35,37]. Based on the condition of the bridge elements obtained during inspections, the Knowledge Model identifies possible alternatives for feasible treatment methods, which are about 50 types in the OBMS. For each possible treatment method, the Deterioration Model (Markovian) is used to predict the condition of the elements, based on the assumption that future deterioration is dependent only on their current condition. These alternatives are considered potential alternative projects, which are then narrowed down using the benefit/cost analysis. The

Cost Model estimates the cost of alternative projects. The alternative projects are then prioritised considering predicted performance and funds.

3.3.2.2 Bridge condition rating in Ontario

The Ontario Structure Inspection Manual (OSIM) has been used for bridge inspections in Ontario since 1985, prior to the implementation of the OBMS [49]. The manual underwent significant alterations in the year 2000, 2003 and 2008. This manual set out the approach to inspection and condition rating of bridge structures to determine their condition. An element-level inspection is utilised as in the Pontis BMS, though the elements considered are not the same. The primary bridge elements considered include abutments, accessories, approaches, barriers, beams, bracing, coatings, culverts, decks, embankments, foundations, joints, piers, retaining walls, sidewalks, and trusses [42,43]. All these primary elements have standardised secondary and auxiliary elements; however, no distinction is made between these types of elements during inspections.

As per the OSIM, the "severity and extent" approach has been used to simplify the use of inspection data to estimate bridge treatment needs and costs. Severity is classified per material type (i.e., timber, concrete or steel) into four levels, as shown by an example in *Table 3-5* for reinforcement corrosion as a concrete defect. On the other hand, the extent is given in terms of quantity [45]. The quantities help estimate the types of repair with associated costs for affected elements.

Table 3-5:Severity levels for reinforcement corrosion as a concrete defect [49]

Severity	Description
Light	Light rust stain on the concrete surface
Medium	Exposed reinforcement with uniform light rust. Loss of reinforcing steel section less than 10%.
Severe	Exposed reinforcement with heavy rusting and localized pitting. Loss of reinforcing steel section between 10% and 20%.
Very severe	Exposed reinforcement with very heavy rusting and pitting. Loss of reinforcing steel section over 20%.

Four Condition States are used to categorise the condition of bridge elements based on the severity given. These Condition States have been defined as Excellent, Good, Fair and Poor [39,42]. Each

Condition State consists of defects of the same severity. *Table 3-6* provides insights on how to grade defects' severity on concrete elements using the four Condition States. The condition of bridge elements is defined as one or more of these Condition States, as an element may be subjected to different deterioration types. Hence, a bridge element can be in different Condition States, or the whole element can be in the same Condition State. However, the emphasis is placed on the importance of recording quantities (in terms of area, length, or unit as appropriate) of defects in the four Condition States of the bridge elements.

Table 3-6: Condition grading for concrete elements – Substructures and Superstructures [49]

Excellent Condition	Good Condition	Fair Condition	Poor Condition
No observed material defects	Light scaling	Medium scaling	Severe to very severe scaling, erosion and disintegration
	Rust stains on concrete due to corroding rebar	Rust stains on concrete due to corroding reinforcing steel	Medium to very severe corrosion of reinforcing steel
	Surface carbonation (reaction with CO_2, associated discolouration, shrinkage and cracks)	Surface defects such as stratification, segregation, cold joints, abrasion, wear, slippery surfaces, wet areas and surface deposits (except on soffits).	
	Light honeycombing and pop-outs	Medium honeycombing and pop-outs	Severe to very severe honeycombing and pop-outs
	Hairline and narrow	Medium cracks	All wide cracks
		Stable relative displacement between precast units. Leaking between precast units	Active relative displacement between precast units
			All delaminated and spalled areas
		Active wet areas on soffit without cracks	Active wet areas or leachate deposits on soffit with associated cracks
			Condition Survey if areas of deterioration in this state >10% for substructures
			Deck Condition Survey if areas of deterioration in this state >10% for superstructures

The OBMS convert the obtained bridge element condition to calculate the Bridge Condition Index (BCI), which is used as a performance measure to prioritise and rank bridges for maintenance projects [31,38]. This index is similar to the Bridge Health Index used in the Pontis BMS discussed in Section 3.3.1.3. It is calculated as a weighted average of the Condition State distribution of various elements, where elements of higher replacement values receive higher weighting.

The OSIM inspection also requires the identification of suspected Performance Deficiencies for each bridge element [49]. These are chosen from the standardised list, including load carrying capacity, excessive deformations, continuing settlements and movements, seized bearing, to name a few. A Performance Deficiency needs to be recorded if the ability of an element to perform its intended function is in question or if performance defects exist. The performance defect in an element can be attributed to design, construction or material defects [49]. Taking a record of this helps hint for possible follow up actions such as additional investigation or maintenance activity. It also allows future repair estimation to include these Performance Deficiencies.

3.3.3 BMS in South Africa

South Africa uses the Struman BMS, which is the commonly used BMS by the Southern Africa Development Community (SADC) countries [5]. The Struman BMS was developed in 1995 by the Council for Scientific and Industrial Research (CSIR) in collaboration with Stewart Scott International (SSI) in South Africa. It was firstly developed for the Taiwan Area National Freeway Bureau and is currently implemented by various road and rail agencies in South Africa and neighbouring countries shown in *Table 3-7*. This was initiated due to limited maintenance budget allocation, which was not adequate to cover increasing maintenance needs for deteriorating bridges. According to Nordengen [6], only about 2.5% of the funds allocated to road rehabilitation projects is spent on bridge maintenance. Hence, a systematic way of prioritising and optimising projects for the benefit of the agencies was needed.

3.3.3.1 Struman BMS

The Struman BMS helps different agencies to allocate scarce funds in a logical and systematic way. For example, in South Africa, because all roads and bridges fall under road infrastructure, they compete for the same national budget [5]. Sometimes funds for bridge projects are being reallocated to road projects because road failures are more common and more visible, though bridge failures may be catastrophic when they do occur. Effective bridge inspections and monitoring are thus needed to be reliable to avoid delaying repairs and preventing catastrophic failures.

Table 3-7: Implementation of the Struman BMS by different agencies [4,5]

Roads and Rail Agencies	Country
The National Roads Agency of South Africa (SANRAL)	South Africa
Spoornet – The South African railway authority	South Africa
The Western Cape Provincial Administration	South Africa
The Eastern Cape Department of Transport	South Africa
Mpumalanga Provincial Government	South Africa
Gauteng Provincial Government	South Africa
Limpopo Provincial Government	South Africa
The KwaZulu -Natal Department of Transport	South Africa
The cities of Cape Town, Port Elizabeth, Johannesburg, and Pietermaritzburg	South Africa
Mangaung Metro	South Africa
Nelson Mandela Metro	South Africa
Sasol (Secunda)	South Africa
N3 Toll Concession Ltd, TRAC and Bakwena	South Africa
Taiwan Area National Freeway Bureau	Taiwan
Dubai Road Transport Authority	United Arab Emirates
Botswana Roads Department	Botswana
Swaziland Ministry of Public Works and Transport	Swaziland
Namibia Roads Authority	Namibia
Namibia Ports Authority (NamPort)	Namibia
Republic of Zambia	Zambia
Kingdom of Lesotho	Lesotho

As in other BMSs, the Struman BMS is a computerised database comprising inventory, inspection, condition, budget, and maintenance modules [6]. *Figure 3-5* shows the flow of information through the five modules. The system process involves collecting inventory and field inspection data in paper format, which is then compiled to the BMS software. The inspection data is used to prioritise and optimise maintenance, repair, and rehabilitation (MR&R) actions within the budget allocated. This results in the implementation of feasible routine maintenance programs. All modules are interlinked; hence, each module informs the other. The inventory informs the inspection module with details about the bridge, the inspection module provide data for condition analysis in the condition module, which provide condition ratings and rank bridges in order of prioritisation. The associated cost for each remedial repair is provided in the budget module, which allows maintenance options to be chosen within the allocated budget.

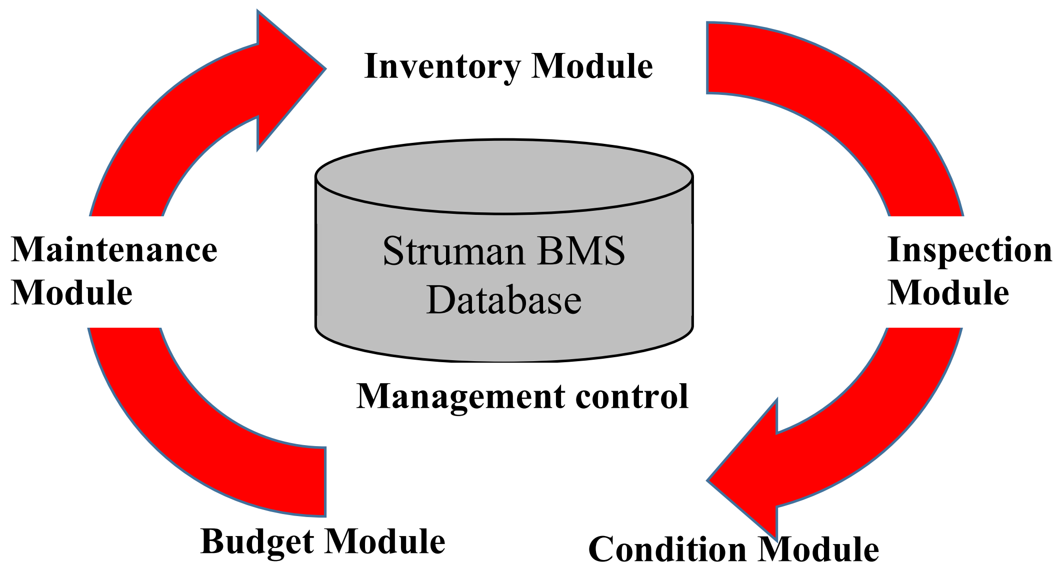


Figure 3-5: Flow of information in the Struman BMS (adapted from [5])

Inventory module – consists of unchangeable data of all bridges in the road infrastructure network, with comprehensive details on the bridge type and design, inventory photos, location and road configuration, construction and contracts, dimensions and geometry, traffic volume and previous works [4]. The information in this module can be obtained from design engineers, bridge drawings or measured onsite during field inspections. According to Nordengen [6], this information may vary between different agencies, e.g. National Road Authorities, Rail Authorities and City Council, as some agencies manage more complex structures than others.

Inspection module – records the input data to the Struman BMS, which is the visual inspection data. Inspection data are obtained by assessing and rating visible defects on bridge elements with respect to their condition of serviceability and safety. These details are recorded in standard inspection sheets onsite, which are later transferred to the BMS database together with the inspection photos.

Condition module – allows prioritisation of bridges within the network by using the inspection data for a specific period. This involves calculations of performance measures such as Condition Index and Priority Index [6]; the former indicates the condition of the whole structure while the latter is used to rank bridges for MR&R actions.

Budget module – helps bridge engineers allocate yearly budgets to identified maintenance and repair work. As per TMH19 [50], the repair quantities are estimated during field inspections, taken as the extent of the defect measured (in unit area, length or each item). The estimated total quantities assigned to all defects on the bridge elements are thus used to estimate the total cost of remedial activities for that specific bridge. The benefit-cost ratio is then used to optimise the budget by comparing the estimated cost with the relevancy of the defects to the bridge as a unit [6]. This ensures maximum benefit is obtained by first repairing or maintaining the element that poses risks to road users and the one with lower cost.

Maintenance module – keeps a detailed record of all maintenance activities that have been performed and completed. A preventative maintenance component also exists, typically done in routines and informs the bridge manager of the work to be done on specific structures if the bridge maintenance crew needs to perform any more remedial work [4].

Management control – also known as the administration module, allows the users to modify the BMS parameters, such as dealing with any changes in the main modules. This module is encrypted to limit access.

3.3.3.2 Assessment approach in the Struman BMS

Bridge inspection is the crucial component of any BMS, which helps accumulate necessary information on the condition of the elements making up the bridge structure. The accuracy of these condition assessment procedures contributes significantly to the quality of the system output, which ultimately defines the success of the BMS.

The bridge inspections are carried out at a network level as per the *TMH19 (Technical Method of Highway- Manual for the Visual Assessment of Road Structures)* [50]. These involve visual inspections, rating and quantifying visible defects using the DERU (Degree, Extent, Relevancy and Urgency) rating system. This rating system is known as the defect-based rating system, and it rates defects on a scale of 1 to 4, as illustrated in *Table 3-8*. The degree (D) is described as the severity of the defect, extent (E) as the area over which the defect occurs on the bridge element, relevancy (R) as the consequence of the defect in terms of the serviceability of the bridge and safety of the users, and urgency (U) as the need to repair. During inspections, only elements with defects are rated, and the bridge only gets one rating of the worst defect on any of the elements, which is typically the defect with high relevancy or highest degree for the same relevancy [6,50].

Table 3-8: Details of the DERU rating system [50]

Rating	Degree (D)	Extent (E)	Relevancy (R)	Urgency (U)
X	Not applicable			Make safe
U	Unable to			Record only
0	None			Monitor only
1	Minor	Local	Minimum	Routine
2	Fair	More than local	Moderate	< 10 years
3	Poor	Less than	Major	< 5 years
4	Severe	General	Critical	ASAP*

*ASAP – As soon as possible

The defects that pose a high risk to the safety of users or critical to the structural integrity of the bridge are usually rated as "Make Safe" (see *Table 3-8*), as it requires urgent attention. Appropriate measures have to be implemented in a short period after the inspection. Defects on bridges are generally rated on the 21 predefined bridge elements (see *Table 3-9*), of which the rating represents the effect of the defect on the safety of users, structural integrity and serviceability of the bridge. The rating then allows these defects to be placed in order of priorities for remedial works.

When carrying out the assessment, one inspection sheet is required for each bridge with a standardised remedial activity list worksheet to summarise the items needing repair, and a separate photographic record sheet. Each visible defect on the inspected item should receive one applicable remedial activity from the list. In addition, inspectors are required to estimate the quantity of repair work. This is used to determine the repair cost of the defect by multiplying the estimated quantities with corresponding unit rates of the activities [5]. Furthermore, the photos of all present defects are recorded in the photographic record sheet, with a brief description of the defect indicated. If cracks exist, inspectors must draw a freehand sketch of the crack pattern or capture a defect photo. All the inspection data are then stored in the Struman BMS inspection module and used for condition analysis.

Table 3-9: Bridge inspection items with assigned weight [50]

Bridge Inspection items			
General Items	**Weight**	**Support Items**	**Weight**
1. Approach embarkment	2	12. Pier protection works	1
2. Guardrail	0	13. Pier foundations	5
3. Waterway	2	14. Pier and columns	5
4. Embarkment protection	2	15. Bearings	3
5. Abutment foundations	5	16. Support drainage	1
6. Abutments	5	17. Expansion joints	2
7. Wing/ retaining walls	4		
8. Surfacing	1	**Span Items**	
9. Superstructure drainage	1	18. Longitudinal members in the deck	5
10. Kerbs/ Sidewalks	1	19. Transverse members in the deck	3
11. Parapets/ Handrails	3	20. Decks & slabs	5
21. Miscellaneous items			0

3.3.3.3 Struman condition assessment indices

The condition analysis is done in the condition module of the Struman BMS, which prioritises bridges in order of needs for repair and rehabilitation using inspection data. The DERU inspection rating allocated to each bridge element is used to place bridges in order of priority. To evaluate the condition of structures, the Struman BMS uses two rating systems; condition rating system and priority rating system, which are typically calculated for each structure as Condition Indices (CIs). The CIs are numerical ratings of an asset depending on its structural integrity [24]. According to the TMH19 [50] and TMH22 [51], the following CIs based on the ratings allocated during visual inspections are calculated.

3.3.3.3.1 Structural Average Condition Index

The Structural Average Condition Index (SACI) is used to rank bridges by considering the overall condition instead of maintenance need [44,46]. This is done by averaging the ratings of all defects found on each bridge. The system is based on the degree and extent of defects only, obtained during the visual inspections. The SACI ratings range from 0 to 100, with 100 indicating a bridge in perfect condition and 0 indicating the worst condition.

1. To determine SACI, the CI value (CI_{ij}) for each of the sub-items (j) of the 21 bridge inspection items (i) in *Table 3-9* is firstly calculated using Equation 3.2.
2. The CI value of the inspection item (CI_i) is determined by summing all CI values for the sub-items (CI_{ij}) and dividing them by the number of sub-items *(n),* as shown by Equation 3.3.
3. The SACI is then calculated as the ratio of the sum of weighted CI values (product of inspection item CIs with respective weight) to the sum of weights as in Equation 3.4.

$$CI_{ij} = 100 - 100\frac{(D+E)}{(b_{ci})} \qquad \text{Equation 3.2}$$

$$CI_i = \frac{\sum_{j=1}^{j=n} CI_{ij}}{n} \qquad \text{Equation 3.3}$$

$$SACI = \frac{\sum_{i=1}^{i=N}(CI_i \times w_i)}{\sum_{i=1}^{i=N} w_i} \qquad \text{Equation 3.4}$$

Where D and E are degree and extent rating for inspected sub-item j of inspection item i and b_{ci} is $D_{max} + E_{max} = 4 + 4 = 8$. CI_{ij} is the condition index for each sub-item j of the considered inspection item i, n is the number of relevant sub-items in inspection item i, CI_i is the condition index of each inspection item i, N is the number of inspection items (21 for the bridge) and w_i is the condition weight for inspection item i.

3.3.3.3.2 Structural Priority Condition Index

The Structural Priority Condition Index (SPCI) is used to identify bridges with critical defects that need urgent treatment [50]. The bridge is given high priority when it has the greatest need to repair. The SPCI is used to develop repair strategies for structures within a budget constraint; structures with high SPCI are mostly the priority though their SACI might be low [53]. The SPCI is calculated following the Structure Deduct-Points Method (SDPM) [46,47] using the allocated DER rating values. The DER rating for the worst defect on the bridge sub-item (e.g. slab 1 or slab 2) is used as an input to calculate the deduct points for that sub-item. The sub-item with the worst condition (highest deduct points) determines the deduct points for the item inspected (e.g. inspection item 20 - Decks and Slabs in *Table 3-9*). The deduct points of the inspection items are then weighted based on the impact the item has on the overall bridge condition. Items are weighted using the scale of 0 to 5; the defined weights for the 21 items of a typical bridge are presented in *Table 3-9*. Five of the highest weighted deduct points are then subtracted from 100 to determine the SPCI for the whole bridge. The bridge is rated using one of the five condition categories presented in *Table 3-10*, with SPCI ranging from 0 (worst condition) to 100 (best condition).

1 To calculate SPCI, firstly, the deduct points of inspection sub-items (dp_{ij}) is determined by Equation 3.5:

$$dp_{ij} = \frac{75(k_d\,.D + k_e\,.E)R^a}{(D_{max}\,.k_d + E_{max}\,.k_e)4^a} \qquad \text{Equation 3.5}$$

Where: *D, E*, and *R* are degree, extent, and relevancy rating for inspection sub-item *j* of item *i;* k_d and k_e are degree and extent coefficient with a provisional default value of 1 for k_d and 0.25 for k_e; *a* is the relevancy exponent with a default value of 1.5.

2 Secondly, the weighted deduct points for the items are determined using Equation 3.6.

3 Thirdly, SPCI for the bridge is then determined by subtracting the deduct points of the five inspection items with the highest deduct points using Equation 3.7.

$$DP_i = \frac{w_i}{w_{max}} \times dp_{imax} \qquad \text{Equation 3.6}$$

$$SPCI = 100 - [DP_{i1} + a.DP_{i2} + b.DP_{i3} + c.DP_{i4} + c.DP_{i5}] \qquad \text{Equation 3.7}$$

Where: DP_i is the weighted deduct points for each inspection item *i,* w_i is the weight for inspection item *i,* w_{max} is the highest weight with a default value of 5, and dp_{imax} is the deduct point of the worst condition in the sub-items of that particular inspection item. The DP_{i1} to DP_{i5} is the deduct points for the five selected inspection items descending from the

1st highest to the 5th highest value. The *a, b* and *c* are defined contribution factors with 30%, 10% and 5%, respectively.

Table 3-10: Condition categories for rating bridges (as defined in TMH22 [51])

Condition category	Index range	Condition description
Very good	85 – 100	New structure and no problem expected.
Good	70 – 85	The structure is still in a condition that only require routine maintenance to retain its condition.
Fair	50 – 70	Structure with the clearly evident deterioration that would require preventative maintenance or renewal of isolated areas.
Poor	30 – 50	Structure needs significant renewal or rehabilitation to improve its structural integrity
Critical	0 – 30	The structure is in imminent danger of structural failure and requires substantial renewal or upgrading with less than 10% of EUL remaining.

3.4 Approaches to bridge inspections in various BMSs

Bridge inspection is the condition assessment procedure and is considered the most important component of a BMS. High-quality bridge inspection data provide quality input for a BMS and allow meaningful results generated from this system. The assessment involves registering defects and damages that affect the safety and reducing bridge serviceability by increasing the degree of deterioration [6]. Hence, bridge inspections aim to report on the physical condition of bridges. Periodic inspections are thus necessary to ensure the bridge's safety and long-term viability, and they are considered part of a continuous condition assessment process [45]. The information obtained from bridge inspections assists bridge managers to decide when to perform maintenance, which procedure to utilise and ensuring that inspections are completed on time. This section covers the bridge inspections practices used in different countries, specifically South Africa, the USA, Canada, and the UK.

3.4.1 Inspection type and intervals

The inspection type and intervals vary from country to country and with individual BMSs. Generally, detailed visual inspections are carried out on a routine basis, principle or detailed inspections are essentially scheduled, and special inspections occur when deemed necessary or when problems arise, e.g. accidents and natural disasters. Overall, inspection types differ due to variation in purpose, inspected bridge elements and the assessment tool or technique used.

Table 7-1 of Appendix A shows the type of inspections with respective intervals for countries including South Africa, the USA, Canada and the UK [11]. The South African practice includes three types of inspection, *Monitoring*, *Principal* and *Verification,* with two additional repair related inspections [44,48]. It should be noted that only the principal inspections are used to determine the condition of the bridges in the South African BMS. It focuses on the observed defects of some bridge elements and not the overall condition of the bridge, which distinguishes it from other BMSs [4]. According to Nordengen [5], bridge inspections usually take place on an interval of 5 years, though on special occasions, frequent inspections are done, e.g. when an unexpected failure occurs. Critical defects are monitored for deterioration and attended to in the five-yearly cycles of inspections to prevent further deterioration.

An approach related to South Africa is followed in the UK by a typical BMS, Highways Structures Management Information System (HiSMIS); the only difference is the practise of *special inspections*. The special inspections investigate critical defects identified during the principal inspections and often involve material sampling and the application of non-destructive techniques [11]. The details on the use of the mentioned methods are discussed in Section 3.4.4.

The USA regulations defined seven types of bridge inspections, with about 95% being routine inspections done after two years [11]. On the other hand, Canada practises six types of inspections based on the USA approach; however, they carry out detailed condition surveys, which enhances the distinction between the two. All inspections in these countries have intervals ranging from 2 years to 6 years.

3.4.2 Inspection personnel

Bridge inspections are performed by different personnel and have different requirements in various countries. In South Africa, inspections are carried out by qualified and experienced personnel either from the bridge authorities (agencies) or appointed consultant firms. The inspections are led by Bridge Network Managers, who coordinates all BMS activities and deliver the training course with some input from the BMS developer (CSIR). As per TMH19 [50], Senior Bridge Inspectors and Bridge Inspectors are the personnel allowed to carry out visual assessments on bridges, and they are accredited through the Committee of Transport Officials (COTO). Some agencies have Team Leaders who organise the inspection activities and do all admin work. The Inspection Specialists and Underwater Bridge Inspectors are also required on an ad hoc basis [11]. All inspectors are required to adhere to the following [50]:

- Must be registered as a Professional Engineer or Technologist.
- Senior Bridge Inspectors must have a minimum of 15 years (for Professional Engineers) and 20 years (for Professional Technologists) experience in bridge management from design to construction, maintenance and repair.
- Bridge Inspectors must have a minimum of 5 years (for Professional Engineers) and 10 years (for Professional Technologists) experience in bridge design and maintenance.
- Must attend a two-day training course, which typically covers various subjects, including bridge management, inventory data capture, typical structural defects, assessment methodology and site inspections

One advantage associated with inspectors is that mostly they are given to inspect bridges designed by their respective firms, making it easy for them as they are familiar with the structure.

In the USA, bridge inspections are carried out by qualified personnel illustrated in *Table 3-11,* with their requirements as per the FHWA manual [11]. It should be noted that inspection team members carry out the inspections without the established qualifications required. Hearn [11] noted that about 70% of identified inspection team members in the State's DOTs are regular staff members without training. However, they work under the supervision of registered Team Leaders.

Bridge inspections in the UK are organised by Area Managers from the Highways Agency, who set up policies and standards and appoint contractors to do the work [11]. The contractor's personnel typically include Team Leaders, Bridge Inspectors, Underwater Inspectors and Specialists. This is similar to the South African management structure. However, UK Highways Agencies do not set requirements for any personnel, except for the Team Leaders who are required to have a Chartered Engineer certificate (equivalent to PE) [11]. It is the responsibility of Team Leaders to ensure contractors appoint personnel that are qualified for the job. However, the performance of contractors is only based on the quality of inspection work delivered, with no emphasis on qualifications and experiences.

Table 3-11: Bridge inspection personnel in USA [adapted from Hearn [11]]

Bridge inspection personnel	Description	Requirements
Program manager	In charge of inspection admins, policy development, inventory, training and reporting.	Must complete bridge inspection training. Must be a registered PE* or with 10 years of bridge inspection experience.
Team leader	Leaders of inspection teams, hence plan, perform and report on field inspections.	Must complete bridge inspection training. Must be a registered PE or have a professional certificate by NICET* (with exceptions of experience) or engineering graduate with 2 years of experience or individual without formal education with 5 years of experience.
Inspection team members	Inspect under the direction of the Team leader	No requirements for inspection team members.
Load rater	Responsible for bridge load rating.	Must complete bridge inspection training. Must be a registered PE or with bridge inspection experience.
Underwater bridge inspectors	Responsible for inspections of submerged elements of bridges.	Must complete an FHWA***-approved course in underwater bridge inspection

*PE - Professional Engineer
**NICET - National Institute for Certification of Engineering Technologies
***FHWA – Federal Highway Administration

The Canadian Agency, such as Ontario bridge inspections, are managed by Reginal Managers with Team leaders and Senior Structural Engineers. All personnel must have an engineering degree, be registered as a PE or certified as Civil Engineering Technologist and have 5 years of bridge inspection experience. The Ontario regulations also require all bridge inspectors to attend a 3-day course every 2 years, as in-house training. Special inspections are usually outsourced to consultant films [49].

3.4.3 Inspection Quality Control and Assurance

Quality Control (QC) and Quality Assurance (QA) are the responsibilities of agencies dealing with bridge inspections. They involve activities such as reviews of inspection programs, teams, training, and reports. Generally, program managers are the ones directly involved with the QC and QA activities. In South Africa, Senior Bridge Inspectors validate bridge inspections by assessing a representative of 2% of all assessed structures each year [6]. If large discrepancies are found, reinspection is implemented. This is a similar practice in Canada; approximately 2% of inspected bridges are verified each year [49]. In the USA, the Team Leaders review all inspection reports as per FHWA requirements [11]. The BMS makes spot checks on bridge data in the UK, and verification of inspections reports is considered when performing detailed inspections for repair projects preparation [41]. In addition, the agencies of these four countries review inspection reports from appointed consultant firms or contractors, of which the records are kept and may impact future awards or tender allocations.

3.4.4 Inspection techniques

Inspection techniques for concrete bridges are still evolving, with visual inspection as the commonly accepted practice. As shown in *Table 3-12*, both countries reviewed (South Africa, USA, Canada and UK) practise visual inspections. Other techniques that some countries practise include measurement, instrumentation, NDT, destructive tests and condition surveys. Though visual inspection is also a non-destructive method, it should be noted that the term "NDT" used in this dissertation refers to methods other than visual inspection.

Table 3-12: Current monitoring practice

Country	Method of monitoring					
	Visual inspection	Measurement	Instrumentation	NDT	Destructive tests	Condition surveys
South Africa	X					
USA	X	X	X	X	X	
Canada	X	X		X	X	X
UK	X			X	X	

South Africa employs visual inspection as a technique to assess both the extent and degree of deterioration. The same technique is used to monitor the deterioration of defects specified as critical during the principal inspections. It should be noted that the assessment manual THM19 [50] did not define any other method; hence the feasibility of using NDTs, measurements or instrumentations need to be considered.

In the USA, three monitoring methods are used; according to Hearn [11], about 50% of the States DOTs use visual inspection, 70% use measurement methods, and 30% use instrumentation. Visual inspections are used to monitor known defects from routine inspections. Measurements are carried out to determine differential movement, deflection, elevations, settlement, crack widths and growth, deck grades, and element rotations. And instrumentations basically involve the use of acoustic emission and strain gauges for fracture-critical members. These are all used to determine any significant change in deterioration of monitored defects, resulting in a change in the condition of such specific bridge element. Notably, the above-mentioned measures do not include rebar corrosion-related parameters such as corrosion potential, corrosion rate and chloride concentration, which are essential in monitoring RC structures.

Furthermore, the USA Bridge Inspectors Reference Manual (BIRM) [54] specified various NDT methods and destructive testing methods based on structural material types, i.e. steel or concrete (see *Table 7-2* of Appendix A), and each state has its specific manual on bridge inspections based on this manual. The BIRM manual provides a comprehensive guide on how all agencies need to conduct their bridge inspection programs. However, most states do not directly address the types and applications of NDTs in their respective inspection manual. According to Lee et al. [55], only 0.2% (8) of 52 states address the application of NDT methods, of which only specific situations where NDT is necessary are briefly mentioned. In these manuals, the common and well-established NDT methods are dye-penetrant, magnetic methods, and ultrasonic testing. Dye penetrant is specifically used to detect flaws in steel members; the magnetic method detect damage to rebars in RC (if the location of rebars is known), and ultrasonic testing allows detection of cracks, cross-section loss and measure member thickness in both steel and concrete members. Among these methods, only ultrasonic testing detects sub-surface defects; the dye penetrant and magnetic methods are insufficient at detecting invisible flaws far away from the concrete surface. It should be noted that these NDTs are specified for use during damage and special inspections [56].

Though the BIRM specified methods that can be used for bridge inspection, most state agencies avoid using NDTs due to their difficulty in use and lack of experience on their applications [56]. Regarding reinforcement corrosion-related NDT methods, the BIRM specified the use of pachometer, half-cell potential, chloride content test, carbonation depth, and concrete strength (see *Table 7-2* of Appendix A). However, the manual did not provide details on the procedures, frequency of use and data interpretations. In addition, state agencies typically use conventional methods, including visual inspection, chain drag, covermeter, rebound hammer, and half-cell potential; according to Moore [57] 78, 65, 43, 39, and 22% of the state DOT uses the mentioned NDT techniques, respectively. It should be noted that though these conventional methods are used during field inspections to determine the deterioration level, their data are not used in the Pontis BMS (only visual inspection results are used for condition ratings).

The inspection techniques in Canada involve general visual inspections, measurements, and condition surveys [49,50]. Measurements are carried out for crack openings, movements or deflections, which are typically attributed to settlement. This continues until the problem is fixed or when the movement becomes stable. Condition surveys basically require measuring defects' extent, deck condition assessment using GPR and thermograph, and load-carrying capacity assessment. It should be noted that though it is defined by the OSIM standard [49] to use these techniques, only traditional destructive methods are currently practised [58].

In the UK, visual inspections are used to record only visible deterioration. Material sampling such as core sampling and NDT techniques are also used during principal and special inspections. The material sampling involves the coring of concrete samples, which are used to determine concrete strength and examined to check the presence of alkali reactions. The NDTs involve measuring concrete cover depths and half-cell potential. Ahmed [58] indicated that though NDTs are defined in the UK, they are hardly used.

It is indicated that visual inspection is the common practice in all the four countries reviewed. Though the technique has the benefits of being cost-effective and straightforward, it is not sufficient to investigate invisible defects. Though NDT and destructive testing are useful, agencies found them to have limitations. The NDT are quick and effective; however, they require experts in applications and interpretation of results. On the other hand, destructive testing is time-consuming, resulting in a limited number of samples to be assembled and tested during inspections. The inspector also has to verify the reading with the onsite inspection results.

3.5 Findings and the research gaps

This chapter concludes by emphasising the following findings:

1. A BMS is the most effective tool available that allows transportation agencies to manage their bridges effectively from conception to end of service life, at a project-level and

network-level. Most BMSs comprise the main components, namely inventory, inspection, condition, maintenance and budget, forming a fully integrated system. Each BMS differs in terms of architecture, principles and assessment methodology. However, they all have a similar objective, to assess the risk of failure and prioritise bridges for repair within limited budgets.

2. Bridge conditions obtained during inspections in the form of numerical ratings provide input data to BMSs. The condition rating varies from one BMS to another, generally on a scale of 1 to 5 (except for the NBIS rating used in the USA, ranging from 0 to 9). The rating represents the defect condition detected on the overall bridge or its elements during inspections. However, this rating is only based on visible defects, which might not be adequate for assessing reinforcement corrosion. In addition, elements in good condition are not rated (i.e., in the Struman BMS), which might result in overlooking the possibility of corrosion being in the initiation stage. As discussed in Chapter 2, defects due to reinforcement corrosion typically show on the concrete surface when significant damage has already occurred. Hence, condition rating obtained from monitoring techniques results may provide more objective and accurate bridge condition assessment and enables monitoring of deterioration progress.

3. The most used performance measure is the Bridge Condition/Health Index, which acts as the acceptable and simplified way of reporting the inspection results to decision-makers, elected officials and bridge users or the public. This index also helps track the bridge condition over time and serves as a basis for allocating resources to bridges within a network. In the USA and Canada, this index is based on the remaining economic value of bridges, which is associated with the uncertainty of estimating the failure cost; hence this cause variability in the replacement value for bridges. South Africa uses the Structural Average Condition Index, which provides a comprehensive depiction of the condition of the bridge by utilising both degree, extent, and relevancy of the defect on a bridge element in the calculation. However, this index is only applicable to structural elements of the bridge, considering only the structural condition (e.g., based on factors such as bridge age, environment and inspection carried out) and neglecting other performance measures such

as bridge functionality (e.g., based on the bearing capacity of the bridge). Integration of different performance measures can be essential to the decision-making process.

4. In the Struman BMS, the repair of the bridge elements is not based on perceived needs of bridges, but on the systematic approach. All structure items have already built-in weighting factors in the prioritisation algorithm of the condition module. With chosen/known as essential items assumed to have a more significant influence on the priority index, e.g., the abutments, piers, and decks with the highest weight of 5 (see *Table 3-9*). It is also challenging to estimate the impact of the condition of an individual element on the bridge as a whole; estimating the impact of a group of elements might be more practical.

5. Most BMSs rely on visual inspections as the primary data source to determine the condition of a bridge. Though the use of NDTs has been specified in the inspection manual of some countries (USA, Canada and UK) they have hardly been used for condition assessment in the BMSs. Visual inspection is however associated with various limitations. Most of all, it focuses on visible defects, which does not adequately address the assessment of reinforcement corrosion. There is thus an opportunity to refine the condition assessment of bridges in the BMSs by including reinforcement corrosion monitoring methods to supplement visual inspection that is already used. Hence, these monitoring techniques can be integrated into the BMS to improve the bridge condition assessment process.

6. Currently, in the Struman BMS, principal inspections are the minority in assessing bridge conditions. Monitoring is only considered on critical defects or when unexpected failures occur. There is a lack of integration of appropriate monitoring systems in the management of bridges, especially monitoring of reinforcement corrosion which is the widely noticed deterioration that affects the durability of RC structures.

7. In addition, the Struman BMS has no clear guidelines that relate the visual inspection results to appropriate repair actions; it depends on the engineer's judgement. Therefore, appropriate monitoring technologies for reinforcement corrosion need to be identified, which will allow suitable remedial repair methods to be chosen for assessed defects.

Chapter 4

4 Monitoring of reinforcement corrosion

As described in Section 2, reinforcement corrosion has been the widely noticed durability problem in the marine environment. The increasing number of RC structures deteriorating before reaching the end of their service life increases maintenance and repair costs. Appropriate monitoring methods are thus needed to evaluate the corrosion of steel reinforcement within the structure during the initiation and propagation periods before severe corrosion damage.

Corrosion monitoring (assessments) is done to detect, measure, and predict corrosion damage on a structure [21]. These methods are essential in evidence-based decision making of maintenance and repair of RC bridges as they help engineers understand the cause and extent of damage on the structure. In addition, the integration of these methods into management decision-making systems could help attain quality assessment for deteriorating and ageing RC bridges.

Bridge inspection is an essential aspect of any BMS, and its variable of condition ratings are set as a basis for the prioritisation of bridge needs. As bridge inspection has evolved, so have the associated inspection (or monitoring) techniques and methodologies. The inspection technologies have advanced and become increasingly available to bridge inspectors. However, their use in BMS related activities is still at the research stage and has not been used practically.

This chapter discusses the corrosion monitoring techniques currently used or available for use in RC bridge inspections, emphasising their principles and applications. The chapter provides a review of different monitoring methods grouped into three categories: visual inspection, non-destructive testing (NDT) and remote monitoring methods. *Figure 4-1* shows the three monitoring categories for RC bridges. The principles, advantages and limitations of these methods are summarised in *Table 7-3* of Appendix A.

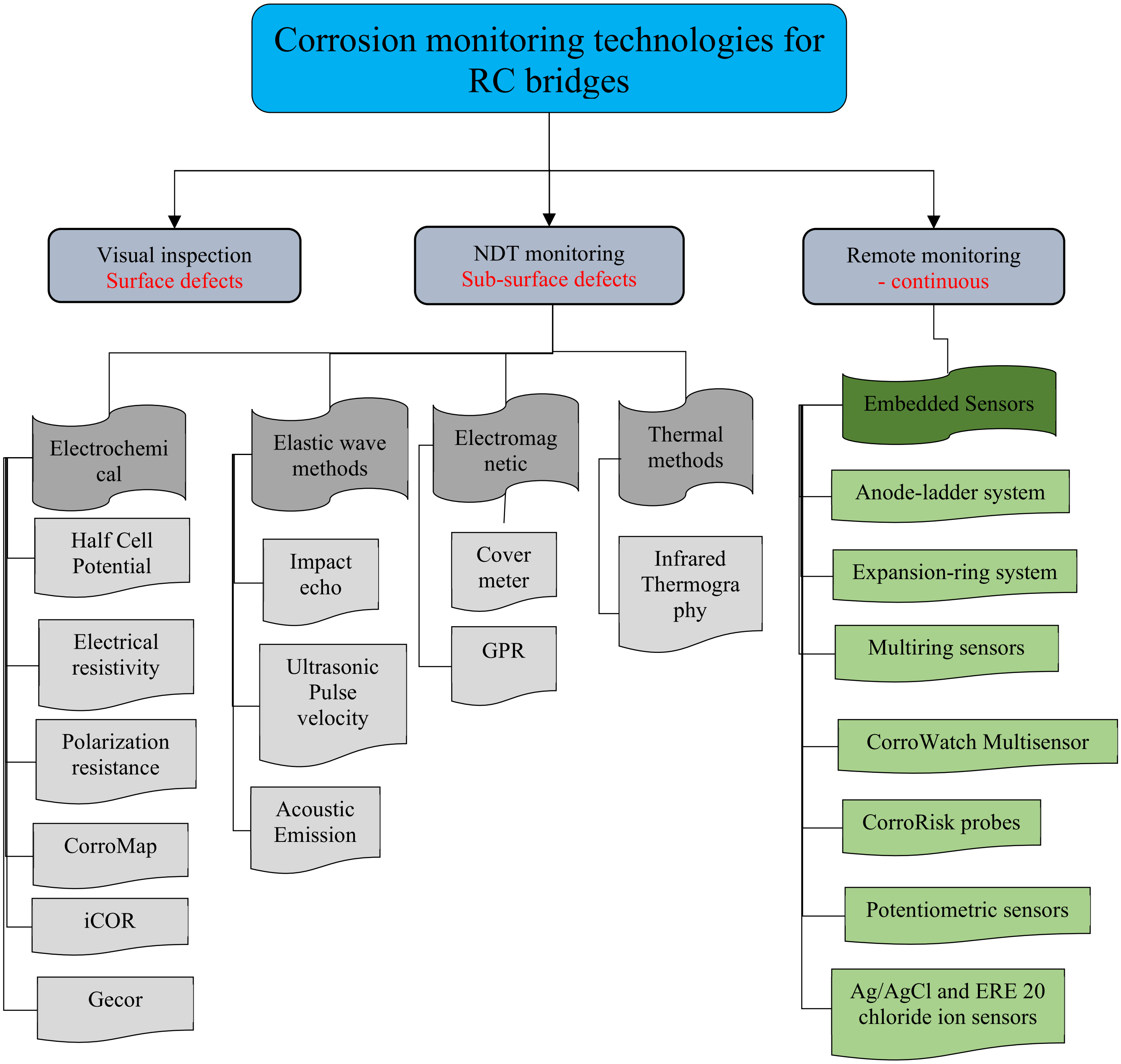


Figure 4-1: Corrosion monitoring technologies for RC bridges

4.1 Visual inspection method

Visual inspection is an assessment process used to identify visible corrosion defects outlined in terms of their appearance, cause, and location on the element being inspected. It is also used to determine the degree of repair needed and help decide whether the structure requires more testing or interventions. Its principle involves observing the cause and current condition status of defects by inspectors who may be trained or have experience in this area. This is still the dominant method used to assess bridges as it is simple, least expensive and does not require special equipment. As discussed in Section 3.5, it is the commonly used method of assessment in BMSs.

Reinforcement corrosion defects, known as damage indicators, are usually evident on the concrete surface in terms of cracks, surface damage (delamination, spalling), and rust stains [59]. Although the concrete condition typically provides a good indication of the deterioration process in terms of these damage indicators, in the case of reinforcement corrosion, deterioration defects only manifest on the surface of the concrete when significant damage has already occurred. Visual inspections may allow identification of the cause of damage, but do not allow detection of internal flaws such as voids, internal cracks, internal delamination, and corrosion before visible damage occurs [45,54,56]. In addition, visual inspections do not quantify the damage, nor do they specify its effect, thus hindering the design of proper maintenance procedures aimed at preventing advanced corrosion damage.

Another problem associated with visual inspections is that they provide subjective information, meaning that their evaluation results depend on the experience and judgement of those conducting them [55]. This affects bridge management decision-making as the accuracy of results provided by the bridge inspectors are relied upon even for funding decisions in the maintenance program. Nevertheless, visual inspection still provides valuable information on the condition of bridges.

4.2 Non-destructive corrosion monitoring methods

There are more advanced methods that have been introduced to supplement visual inspection because of its drawbacks. NDTs are one of the advanced methods that do not affect the integrity of a bridge element during testing and provide reasonably accurate and reliable data [58]. Evaluations with these methods are carried out by experienced inspectors on structures still in service to provide the degree, extent and severity of deterioration at a specific time. The condition status obtained during the assessment can also predict the remaining service life. The following sections discuss the NDTs used to monitor parameters involved in the reinforcement corrosion process. These are categorized according to their principal process, as illustrated in *Figure 4-1.*

4.2.1 Electrochemical methods

As discussed in Chapter 2, reinforcement corrosion affects both the reinforcing steel and the concrete. Various methods have been developed for measuring the degree of reinforcement corrosion [61]. Some methods focus on determining the rebar condition and others on the concrete condition. Among these methods are electrochemical methods for evaluation of corrosion rate. Corrosion rate measurements can be used to predict future maintenance of RC structures once corrosion has initiated. Some of the considered parameters for the measurements include corrosion potential, concrete resistivity, and polarization resistance. These parameters can thus be monitored with surface electrodes/sensors for periodic measurements and with embedded sensors for remote/continuous measurements.

4.2.1.1 Half-cell potential measurement

Half-cell potential (HCP) or open circuit potential (OCP) is an electrochemical method used to assess the risk of steel corrosion by measuring the potential difference between the rebar and an external reference electrode (RE) with a high impedance voltmeter, as shown in *Figure 4-2* [28]. The principle uses a RE (half-cell-which is a device consisting of metal in a solution of its ions) such as a saturated-calomel electrode (SCE), copper-sulfate electrode (CSE), or silver-chloride electrode (Ag/AgCl) [19]. When connected to another metal in a solution of its own such as steel in concrete pore solution, the potential difference will be induced between the two half-cells, measured with a voltmeter. The voltmeter usually indicates a negative value of potential, which is used to indicate the likelihood of corrosion activity.

The HCP measurement is usually performed using portable, lightweight equipment and requires a connection to the reinforcing steel. When testing, the concrete surface is dampened with water to increase electrical conductivity. The RE is moved on the concrete surface directly above the reinforcement to assess the corrosion potential at different locations. Following a spaced grid pattern, equipotential contour maps are obtained. This potential mapping signifies corrosion conditions at specific investigated areas, and a steeper gradient on the maps indicates a higher risk of corrosion at that position [21]. An accurate grid of measurement points is thus essential to ensure the precise identification of actively corroding spots.

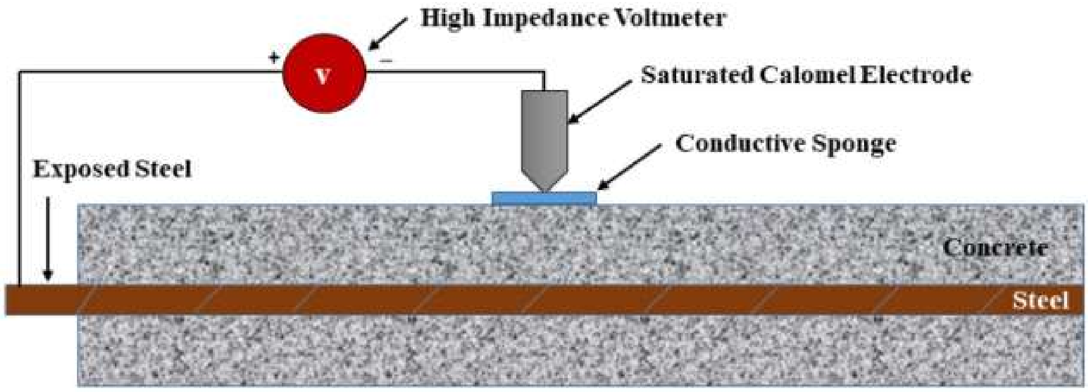


Figure 4-2: Schematic representation of HCP measurement [62]

Apart from using a surface electrode for measurements, it is often desirable to embed REs for permanent measurement and monitoring of the steel reinforcement potential, especially in inaccessible areas. RE such as metal-metal oxide (MMO), manganese oxide (MnO_2), activated-carbon and pseudo-reference electrode have shown good stability when used in concrete [57–59]. These studies validate embedded sensors for corrosion monitoring applications in RC. In addition, advanced climbing robots and flying drones are recently used to assess corrosion potentials of inaccessible elements [61].

The interpretation of results for this method is based on the recommendations by the ASTM C876 [65] standards. As shown in *Table 4-1*, the absolute values of corrosion potential are used for evaluating the likelihood of corrosion occurring in the assessed area. The potential measured when steel is passive is relatively positive between 0 - 200 mV and changes to more negative typically −350mV when the steel is actively corroding [21]. It should be noted that there is no distinct relationship between corrosion potential and the corrosion rate. The HCP method only provides qualitative data for locating high corrosion risk areas before evident damage appears at the concrete surface. Furthermore, the measurements are affected by numerous factors, including the availability of oxygen, cover depth, concrete condition (resistivity, presence of cracks), the composition of the pore solution, and the condition of the reinforcing steel [13,57,61].

Table 4-1: Corrosion risk based on HCP measurements, as per ASTM C876 [56]

Likely corrosion condition	Half-cell potential (HPC) values			
	Silver/silver chloride (mV)	**Copper/copper sulphate (mV)**	**Saturated calomel (mV)**	**Standard hydrogen (mV)**
Low (<10% risk corrosion)	> −100	> −200	> −80	+120
Intermediate corrosion risk	−100 to −250	−200 to −350	−80 to −230	+120 to −30
High (<90% risk of	−250 to −400	−350 to −500	−230 to −380	−30
Severe corrosion	< −400	< −500	<−380	−180

4.2.1.2 Concrete resistivity measurement

Electrical resistivity of concrete (ER) is the measure of the ability to resist the flow of electric current through concrete. For corrosion to occur, the electric current passes through the concrete pores from the anode to the cathode; the concrete resistance influences this process. Thus, the corrosion process is typically slower in high resistance concrete compared to low resistance concrete.

ER is determined using the one, two, or four-probes method, where the latter is mostly preferred for on-site and laboratory measurements. *Figure 4-3* shows the Wenner four probes method, commonly used [13,62]. This method provides an indication of how far the concrete supports corrosion in cases where corrosion occurs. It is non-intrusive and does not require connection to the embedded steel in concrete. The method uses a portable equipment comprising equally spaced four probes, which provides electrical contact with the concrete surface [2]. The test principle involves impressing a known current 'I' between the external probes while measuring the potential drop 'V' between the inner probes, which is then used to calculate ER as illustrated by Equation 4.1 [57,63].

$$(\rho) = \frac{2\pi a V}{I} \qquad \text{Equation 4.1}$$

Where (a) is the inner probe distance (in cm), and '2πa' is the geometrical factor, which depends on the geometry and arrangement of the electrode and the specimen being tested for a specific structure.

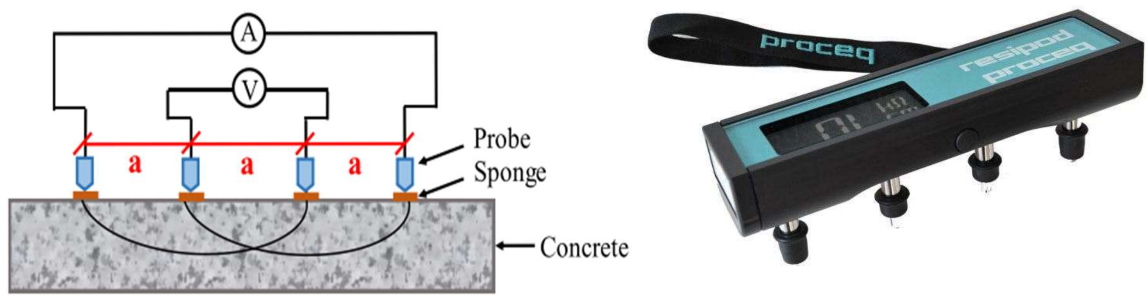


Figure 4-3: Schematic drawing of the four-probe method resistivity measurement (left) and equipment used to measure resistivity (right) [68]

ER measurement is widely accepted; it is used to interpret corrosion rate or HCP measurements. Some authors found ER to be inversely proportional to the corrosion rate of steel in concrete [13,28,57]. Others argue that ER becomes a controlling factor in dry conditions as its influenced by the pore solution composition and concrete porosity [69]. ER influences the corrosion rates in concrete when the rebar is corroding, and hence it provides a general indication of the corrosion risk as indicated in *Table 4-2*. The corrosion risk is high when the concrete resistivity is lower than 10 kΩ.cm and negligible when greater than 100 kΩ.cm [70]. In addition, RILEM recommends that low resistivity areas are more susceptible to chloride penetration than areas with high resistivity [70]. This is because resistivity, like chloride ingress, is related to the concrete's pore structure.

Table 4-2: Corrosion risk based on concrete electrical resistivity [13,28]

Resistivity (kΩ.cm)	**Corrosion risk**
Greater than 20	Low
10 to 20	Moderate
5 to 10	High
Less than 5	Very high

Concrete resistivity has also been used to indirectly assess the concrete quality and its characteristics, such as electrochemical chloride ion diffusion and the degree of saturation [13,64]. From a concrete durability point of view, concrete resistivity is a good indicator of chloride ion diffusivity and moisture transport. The electrical resistance tends to decrease with an increase in moisture content and chloride concentration, of which can be attributed to the ionic and moisture conductivity of the electric current through the concrete. Low resistivity areas indicate the possibility of chloride penetration, as such concrete resistivity is related to the susceptibility of chloride ingress into the concrete [27]. The electrical resistivity of concrete is thus related to the service life stages of a structure, the initiation period (during chloride penetration) and the propagation period (corrosion rate).

Furthermore, concrete resistivity measurement is influenced by various factors. Exposure conditions such as moisture content and temperature increase the ionic transport in the pore solution of concrete, which results in decreasing concrete resistivity. Concrete composition (w/b ratio, type of binder and aggregate size) and the concrete curing conditions affect the pore solution composition and concrete porosity [61]. For example, the use of blended cement (fly ash or ground granulated blast furnace slag) refines the concrete microstructure, hence increasing resistivity. The presence of cracks influences the resistivity measurement and leads to under or overestimation of results, as it is inconsistent with the homogeneous and isotropic nature of concrete. The presence of rebars also affects the resistivity measurement by potentially causing short-circuiting, which reduces current flow within the concrete [61]. Thus, it is always recommended for readings to be taken away from the rebars for accurate results. Other factors include concrete cover depth, surface contact of probes, direction and spacing of probe when testing [62].

However, the resistivity measurement is associated with some challenges, such that it is not conclusive on its own and has to be used in conjunction with other test methods such as HCP measurements (described in Section 4.2.1.1) and corrosion rate measurements (described in Section 4.2.1.3). Furthermore, the method provides details on the capacity of concrete allowing corrosion, but it does not affirm whether corrosion has occurred and to what extent is the active corrosion [21].

4.2.1.3 Linear polarization measurement

Linear polarization resistance (LPR) measurement is an electrochemical technique used to determine the actual steel reinforcement corrosion rate in RC structures [71]. This technique monitors the time-dependent corrosion activity of the reinforcing steel embedded in concrete. The principle used by this test method involves correlating the HCP of corroding steel to the externally applied current, i.e. proportioning the applied current to the change in potential to obtain the corrosion rate [68]. Its set-up typically includes auxiliary electrode and reference electrode placed on the concrete surface and a potentiostat connected to the working electrode (corroding steel), as shown in *Figure 4-4* [13,21]. The auxiliary electrode can be either titanium bars or strips (Ti/MMO), and commonly Ag/AgCl or MnO_2 reference electrodes are used.

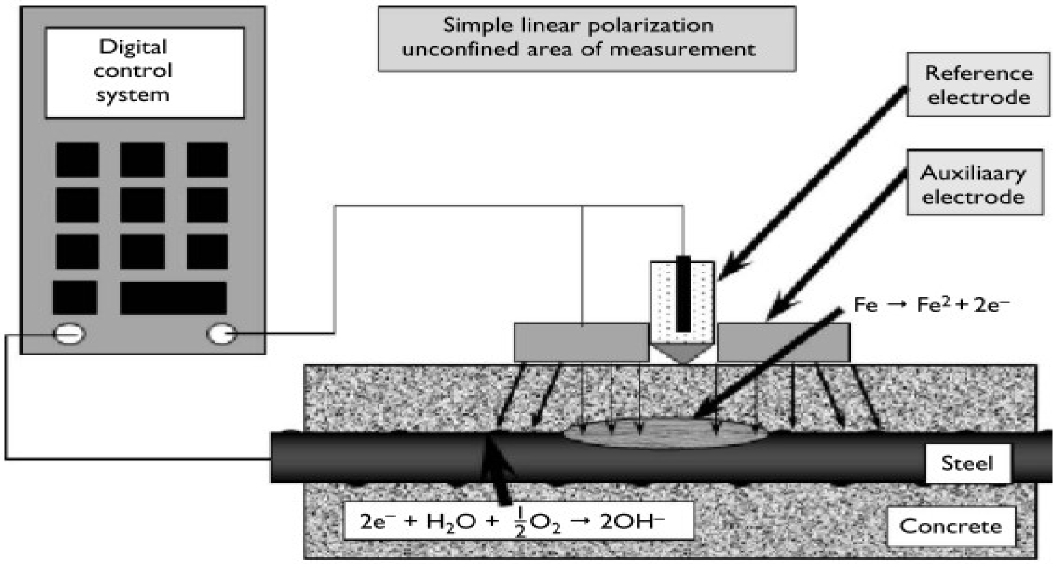


Figure 4-4: Schematic of the polarization resistance measurement device [21]

Fundamentally, steel corrosion generates anodic zones on the steel surface in concrete where the steel is oxidized and cathodic zones where oxygen is reduced. In chloride-induced corrosion where the anode and cathode are clearly separated, their potentials are the same as corrosion potential (E_{corr}), and the current densities are equivalent. The LPR technique involves applying a slight potential shift *(ΔE)* of about ± 10 - 20 mV to the reinforcing to produce a perturbation from its equilibrium potential (E_{corr}) so that the resulting change in current density may be measured *(ΔI)*. The relation between *(ΔE)* and *(ΔI)* defines the polarization resistance (R_p), which is used to calculate the corrosion current density (i_{corr}) by using the Stern-Geary Equation 4.3 [18,68]. The formula is applicable for non-uniform corrosion processes such as pitting corrosion.

$$R_p = \frac{\Delta E}{\Delta I}$$ Equation 4.2

$$i_{corr} = \frac{B}{A * R_p}$$ Equation 4.3

Where (R_p) is the polarization resistance, as illustrated by Equation 4.2, (B) is the Stern-Geary constant varying from 26 to 52 mV depending on the passive (52 mV) or active (26 mV) condition of steel [28], and (A) is the surface area of the polarized steel.

Due to the unconfined flow of current from the auxiliary electrode, the polarized surface area of steel can be laterally distributed over an unknown larger area of steel [28]. This poses a constraint on the structure such that the area of the working electrode is greater than that of the counter electrode, resulting in loss of electrical signal when testing due to larger differences between the two. To overcome this limitation, Feliu et al. [73] introduced a second auxiliary guard ring electrode (shown in *Figure 4-5*) surrounding the inner auxiliary electrode to limit current flow during the experiment within a limited area. This method provides a value of the corrosion rate of the steel being monitored, which is measured instantaneously.

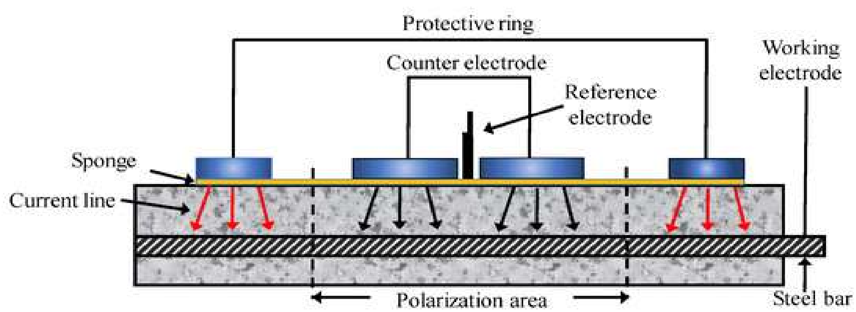


Figure 4-5: Polarization area by guard ring method [68]

One of the latest technologies using the guard ring technique is the Gecor 8^{TM} developed by NDT James Instruments [74]. This technology is distinguished from others as its applicable in very wet or submerged structures. It is easy to use, with a programming feature that automatically evaluates and provides graphic displays of the contour mapping, and stores data in its database. Some technologies take LPR measurements without the guard ring, e.g., CorroMap developed by Force Technology [75]. This handheld equipment has an algorithm that automatically calculates the polarization resistance and corrosion rate values. It displays contour mapping as an output with subsequent measurements in an MS Excel format that is transferable.

Most LPR technologies are considered non-intrusive and only require a connection to the reinforcement without imposing other damage to the structure. Giatec recently developed a wireless technology for measuring corrosion rate without a connection requirement to the steel; the Giatec iCOR [76]. This advanced technology does not only detect corrosion rate but measures corrosion potential and electrical resistivity. This technology is explained in detail in Section 4.2.1.3.1. Other alternatives include embedding electrochemical sensors at critical positions on the structure – allowing long-term corrosion monitoring and assessing inaccessible areas [28]. Karthick et al. [64] compared the performance of embedded sensors to surface-mounted electrodes for corrosion monitoring in RC. They found that the embedded system accurately predicts corrosion rate by 2.89 times lesser than the surface mounted system. The embedded system showed I_{corr} values of the embedded steel as 1.76×10^{-4} mA/cm^2, compared to 5.10×10^{-4} mA/cm^2 for surface mounted system [64].

The LPR results can be evaluated using the general criteria shown in *Table 4-3* [14]. However, to obtain reliable results, the testing and interpretation should be conducted by experienced personnel. One limitation associated with this method is the inherent uncertainty in converting polarization resistance to corrosion density, which is based on the determination of the polarized steel area. The LPR measurements are also relatively prolonged and influenced by temperature, concrete resistivity and relative humidity, and different devices typically give different corrosion rate values [77].

Table 4-3: General criteria used to evaluate corrosion rate [21]

Corrosion current (I_{corr}, μA/cm²)	**Condition of the rebar**
< 0.1	Passive condition
0.1 – 0.5	Low to moderate corrosion
0.5 – 1.0	Moderate to high corrosion
> 1	High corrosion rate

4.2.1.3.1 Giatec iCOR

Giatec iCOR is an advanced technology developed by Giatec Scientific Inc. for wirelessly evaluating the corrosion of RC structures. It should be noted that information given hereafter is based on the company's (Giatec Scientific Inc.) information and not scientific evidence. The iCOR technology is applicable for both laboratory and field measurements. Its advanced sensors are used to measure corrosion rate, concrete resistivity, corrosion potentials, temperature, and relative humidity. The device has an advantage in eliminating the need for connection requirement to the embedded steel in concrete when testing corrosion rate or concrete resistivity, which is the case in other commercially available electrochemical technologies. In addition, it comes with a Tablet (see *Figure 4-6*) that wirelessly collects, analyses and stores data, and provides fast, automatic reports of the results [76].

The Giatec iCOR follows the same principle as other technologies of using LPR to obtain corrosion rate, the Wenner probe method to determine ER, and ASTM C876 for HCP measurements [78]. As per the Manual [78], rebar sizes and cover depth are required as inputs in the software to define the grid for measurements. An advanced algorithm in the device's software automatically analyses different properties of concrete and steel reinforcement. The software produces results in MS Excel raw data format or contour maps as outputs for data analyses for any selected parameters. This allows easy reporting, exporting and sharing of results, and subsequent determination of areas of high corrosion risk and areas with active corrosion.

Figure 4-6:Giatec iCOR measuring unit: iCOR device (left) and Data Recording Unit (right) [76]

The Giatec iCOR is the only product available that can wirelessly measure corrosion rates [76]; hence, there is no standard available yet that these measurements can comply with. However, Giatec provides technical specifications that can be followed, and colour-coding is used to interpret the results, as shown in *Table 4-4* [78]. Nevertheless, creating new standards for corrosion rate measurement is the hope for the future of various researchers.

Typically, it is recommended to supplement corrosion rate measurements with HCP measurements, concrete resistivity, and cover depth. However, with the iCOR, all measurements are done simultaneously, allowing straightforward interpretations of results. These measurements vary from one structure to the other, as they are influenced by parameters such as temperature, moisture, cover depth, concrete properties, availability of oxygen, presence of cracks and delamination, as mentioned in the previous sections.

Table 4-4: Interpretations of results when using Giatec iCOR [78]

Parameter measured	Criteria	Classification	Color code
Corrosion rate (µA/cm²)	< 1	Low	Green
	1 – 3	Moderate	Yellow
	3 – 10	High	Orange
	>10	Very high	Red
Concrete resistivity (kΩ.cm)	>20	Very high	Green
	10 – 20	High	Yellow
	5 – 10	Moderate	Orange
	<5	Low	Red
Half-cell potential (mV/CSE)	>−200	<10% risk of corrosion	Green
	−200 to −350	Uncertain	Yellow
	<−350	>90% risk of corrosion	Red

4.2.2 Elastic wave methods

Elastic wave methods are useful when additional information is required for determining rebar corrosion damage. These methods are used to estimate mechanical properties and heterogeneous characteristics of concrete resulting from reinforcement corrosion. This is because the elastic waves are reactive to damage such as internal cracks, voids, delamination and corrosion products, which results from reinforcement corrosion [79]. There are three major wave-based methods: Impact Echo (IE), Ultrasonic Pulse Velocity (UPV), and Acoustic Emission (AE). IE and UPV are based on wave propagations produced from external impacts on the solid surface, and AE is based on the wave propagation exerted from the material itself. This section describes the three methods in detail.

4.2.2.1 Impact Echo

The IE method is based on the propagation and reflection of elastic waves in concrete (solids). Its principle involves introducing a low-frequency stress wave using a mechanical impact on the concrete surface, as shown in *Figure 4-7*. The induced waves propagate through the concrete and are reflected by internal cracks, voids or changes in material characteristics (the so-called echoes from these sources) [56,74]. Near the impact point, the reflected waves cause displacements recorded and converted into frequency data by the receipt transducer (piezoelectric sensor). The resulting displacement-time curves are analysed in the frequency domain for anomalies. The frequency domain allows ultrasonic waves to undertake multiple reflections within the concrete member. This frequency analysis is carried out using the fast Fourier transform technique to obtain amplitude spectrum; peak amplitudes in the waveform are related to the depth of defects or thickness of concrete elements as illustrated by Equation 4.4 [77].

$$d = \frac{C_p}{2f} \qquad \text{Equation 4.4}$$

Where: *d* is the distance from the impact point to the defect or edge of the concrete element, C_p is the speed of the waves, and *f* is the frequency of reflected P-waves.

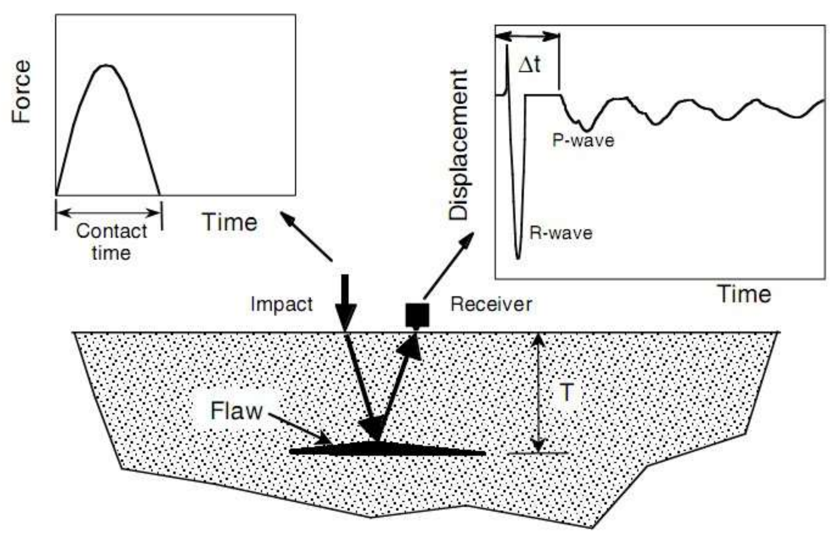


Figure 4-7: Principle of impact echo method [60]

Observed from various studies, IE is used for different applications in RC (1) detect and locate flaws such as internal cracks, voids and delamination, (2) determine the thickness of concrete elements, and (3) measure crack depth [54,56,75,76]. The mentioned parameters might be corrosion-related and need to be considered for assessment. Using this method during inspection is fast and requires less time; hence it can be used for quality control and routine checks in maintenance programs. However, defect detection accuracy depends on the duration of the impact, and the test is less reliable on bridge decks with asphalt overlays [59]. Hence, the number of studies tempting to monitor reinforcement corrosion with this technology are limited.

4.2.2.2 Ultrasonic Pulse Velocity

The UPV method determines concrete properties by measuring the propagation time of ultrasonic pulse (compression stress wave) through the concrete. Its principle involves generating an elastic wave and monitoring the propagation time between the emitting source and the receiver, both on the concrete surface [13,54,74]. The emitter and receiver are typically piezoelectric and can be arranged in three ways: (1) placed directly opposite each other, (2) diagonally to each other, or (3) on the same face and separated by a known distance. The latter is mostly preferred as concrete elements are typically easily accessible on one face during inspections. The receiver is moved along the surface with the fixed emitter, and the transit time is recorded. Pulse velocity is then calculated automatically with the obtained time at a known distance (Pulse velocity = distance/time). The variation of wave velocities in terms of concrete uniformity, presence of flaws, and internal cracks characterize the concrete quality, which is assessed following the guidelines in *Table 4-5* [81].

Table 4-5: General guidelines for predicting concrete quality based on UPV [81]

Pulse velocity (km/s)	Concrete quality
>4.5	Excellent
3.5 – 4.5	Good
3.0 – 3.5	Medium
<3.0	Doubtful

In heterogeneous materials such as concrete, the pulse velocity is considered a function of density, elastic modulus and Poisson’s ratio. This is because the non-uniform consolidation of concrete can lead to variation in density, and the variation in mix proportions or curing makes elastic properties differ [77]. Thus, measured pulse velocities are related to the concrete quality and its continuity.

Researchers have attempted to relate pulse velocity measurements to reinforcement corrosion and obtained promising results. Zaki et al. [79] found that amplitude attenuations correlate well with corrosion damages. Defects such as internal cracking caused by reinforcement corrosion results in wave attenuations and a decrease in UPV. Li et al. [82] attempted to detect corrosion damage and found that UPV measured by the first wave peak could describe the reinforcement corrosion process, from formation of corrosion products to the visibility of corrosion damage indicators on the concrete surface. As the corrosion damage level increases, the relative variation for the first wave peak value of UPV increases first and then decreases. This condition occurs due to an increase in corrosion products from the progression of the corrosion process, which results in debonding between the reinforcing steel and the concrete. This leads to great reflection of the first wave and reduction in direct transmission of waves [79]. This study was laboratory-based, and using this approach in practice could be difficult as it involves estimating the corrosion damage level of the steel rebar. Advanced studies are thus needed for better improvement.

In the context of reinforcement corrosion, this method has been studied for specific applications. It is used to detect internal flaws such as cracks, voids, delamination and other defects that might result from steel degradation. It is also used to estimate concrete strength, determine concrete member thickness and measure surface crack depth [60]. This method is well established with procedures specified in ASTM C597-09 [83], BS EN 12504-4 [84], and IS 13311-1[81]. It also allows fast measurements as its equipment have onboard data storage and processing abilities. However, it is less reliable in detecting shallow defects, and the results are influenced by pulse attenuation, concrete composition and aggregate sizes.

4.2.2.3 Acoustic Emission

The AE method is based on transient stress waves caused by a rapid release of energy within a material such as concrete [68]. This technique has been used for structural health monitoring (SHM), especially monitoring of reinforcement corrosion. This method is non-intrusive, and it is considered a good complementary method to other elastic wave-based methods like UPV and IE. In this method, the rapid energy released in RC is produced from localised sources such as internal crack growth, void closure, plastic deformation, generation of corrosion products and other material degradation [54,64]. This all results from the corrosion process, where the increased volume of corrosion products induces stresses on the concrete surrounding the steel, leading to the formation of cracks and other defects, thus generating sound waves. Signals emitted by the localised sources can be continuous signals (rapidly occurring emission) such as plastic deformation or burst signals (individual emission) such as cracks in concrete [58]. These emitted elastic waves are detected and converted into electrical signals by AE sensors on the concrete surface. The signals are then amplified, processed and recorded by the data acquisition system (see *Figure 4-8*) to obtain the information about the material emitting the emission.

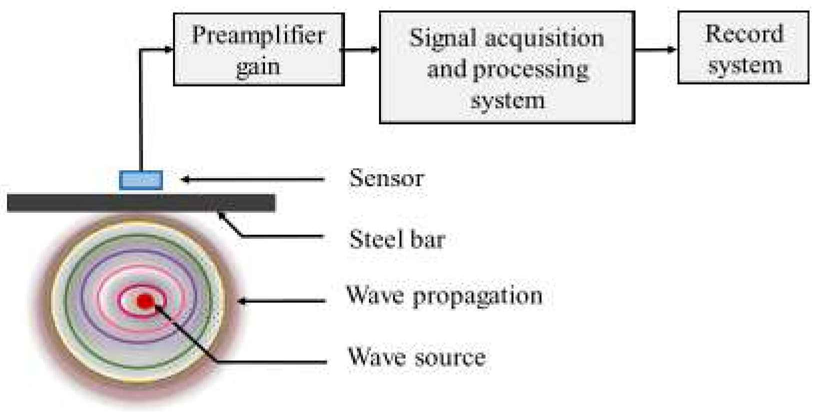


Figure 4-8: Principle of Acoustic Emission [68]

AE has recently emerged as a powerful tool for remote monitoring ongoing corrosion damage such as cracks formation and propagation in RC structures. Because of its high sensitivity, it can detect active cracks at their earlier stage before they become visible, making it suitable for long term monitoring [85]. It can help identify different sources and stages of corrosion damage [68]. Through AE parameter analysis, Kawasaki et al.[86] found two corrosion stages; the corrosion onset and nucleation of corrosion-induced cracking. The same corrosion stages were monitored by Li et al. [87], of which Luo et al. [68] specified that steel passivation occurs at a frequency of 20 to 30 kHz, and the concrete cracking stage ranges from 30 to 50 kHz on average. In addition, a correlation between increasing AE parameter and the corrosion current density was established by Idrissi & Limam [88].

So far, this system has been used mostly in laboratory settings with limited fieldwork. There is no critical standard either on its procedures, installation or interpretation of results in terms of corrosion. However, threshold values of AE technologies have been reported in literature ranging from 40 to 60 dB [89]. AE signal outputs have massive data and require extensive data interpretation and analysis to obtain meaningful results. Abdelrahman [85] suggested a filtering technique and considering pre-existing damage in the AE Intensity Analysis to solve this limitation in the field application. AE method is susceptible to interference by other waves such as noise waves or wave reflections [68]. This leads to unnecessary elastic waves detected by the sensors and subsequent misinterpretations of results. Environmental conditions such as seasonal rain and temperature are other factors that influence AE activities [85]. AE activities tend to increase with a decrease in temperature, and moisture and repeated wet/dry cycling from rain events accelerate the corrosion process, hence increasing the energy released from localized sources.

The AE technique can detect damage resulting from reinforcement corrosion at early stages in the propagation period; this provides early warning and allows preventive maintenance to be carried out prior to severe damage. This method can also be useful in supplementing visual inspection to monitor the behaviour of observed defects. This could reduce the frequency of inspection, because usually when a defect has been observed on a certain member, recommended actions involve increased inspection on the defective location.

4.2.3 Electromagnetic methods

4.2.3.1 Cover measurement

Adequate cover depths are essential to ensure sufficient durability of RC structures, as it ensures sufficient depth to reinforcements. Cover measurements are carried out to locate the depth, size, and location of reinforcement. In corrosion-affected RC bridges, the cover measurements are used to identify areas that are prone to corrosion. As inadequate cover typically increases the corrosion risk by providing easy access of corrosion agents (chlorides, moisture, oxygen, or carbonation) to the steel [21]. Thus, the protection of reinforcement depends on the thickness of the concrete cover and its quality.

Covermeters are used to perform this measurement non-destructively by using an alternating magnetic field to locate the steel and other magnetic materials [2]. The device used consists of a probe and an indicator unit (see *Figure 4-9*) where the probe induces an alternating current on the concrete surface using the electromagnetic induction principle. The presence of reinforcing steel or other magnetic material in concrete causes magnetic field changes detected by the receiver on the device, thereby providing information on the reinforcement [77].

The use of covermeters has been established and standardised by BS 1881: Part 204 [90]. The literature found that a minimum cover of 40 mm is required for structures exposed to the marine environment, with an optimum recommendation of 50 mm [14]. The ACI 357 also specified a minimal cover of at least 50 mm for RC members subjected to seawater, which is only applicable to submerged and atmospheric-exposed structural elements. However, SANS 10100-2 recommends a cover of 65 mm for members in contact with seawater [91]. Though the specification of cover depths varies from code to code, they all aim for structures to have an adequate concrete cover that delays or slows down chloride ingress to the steel level.

Figure 4-9: Use of covermeter for cover depth and measurement of rebar location and size [92]

Covermeters are commonly used for condition assessment of reinforcement corrosion-affected structures as they have the advantages of being portable, lightweight and easy to use. Their ability to detect locations of reinforcements helps other methods such as concrete resistivity and HCP measurements. Apart from this, they are also used to establish concrete cover depths, which is an essential parameter for reinforcement corrosion. However, the accuracy of cover depths is affected by bar size and bar spacing, and the maximum penetration depends on the meter design [77]. Though, a maximum cover depth of 120 mm is always recommended for magnetic covermeters [14].

Furthermore, cover depth is one of the input parameters required for the service life prediction of RC structures. With the known surface chloride concentration, cover depth, and diffusion coefficient, one would determine the time required for chloride content at the steel level to reach the chloride threshold, which indicates corrosion initiation. In addition, relating cover depth to factors such as chloride penetration depth indicates the sections most vulnerable to corrosion [2].

4.2.3.2 Ground-penetrating radar

Ground-penetrating radar (GPR) is an electromagnetic method used to detect the presence and location of defects and steel reinforcement. GPR is also proficient at the evaluation of concrete cover thickness. Hence, it has been widely used for bridge monitoring, especially bridge decks [56,89–93]. The GPR system consists of three units; antenna, control and display units, as illustrated in *Figure 4-10* [89,90]. For example, to determine the condition of a bridge deck, the antenna is dragged manually over the inspected surface, which transmits an electromagnetic impulse into the concrete, which is reflected to the antenna and produce an output signal. The impulse reflects at the interface with materials of different dielectric properties such as reinforcing steel, chlorides, moisture or other anomalies. Furthermore, the output signal is proportional to the amplitude of the reflected impulse [93]. Hence, strong reflections are typically attributed to sound concrete, and high-amplitude attenuations are associated with concrete corrosion damage.

The GPR profile (example see *Figure 4-11*), which is the output of each scan, is typically analysed to evaluate the conditions of the bridge deck. According to Dabous [93], there are two ways to analyse the results; numerical and visual methods. The numerical method follows the ASTM D6087-08 in calculating deterioration based on the reflection amplitudes at top rebars, of which variations in values are used to analyse the internal condition of the inspected element. The visual method is based on a visual assessment of the GPR profile by an analyst to identify locations of signal attenuation and factors affecting the profile pattern. However, the numerical method ignores essential information such as changes in bar spacing, polarisation effects, surface moisture or attenuation of image; hence, visual analysis is preferred for accurate mapping defects [90,91].

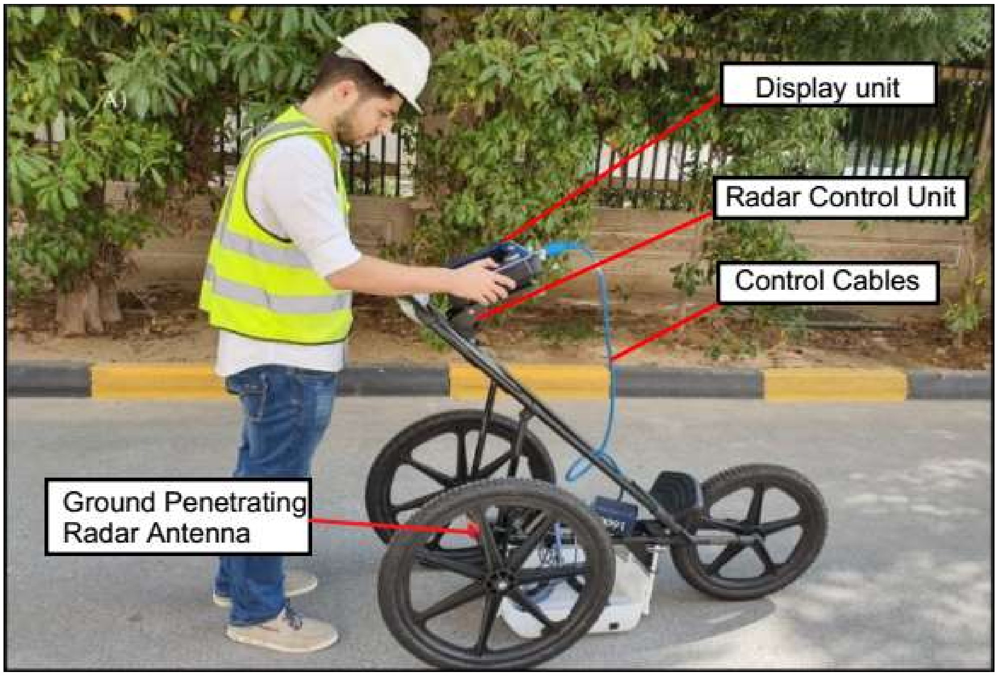


Figure 4-10: Experimental set-up of the GPR equipment [94]

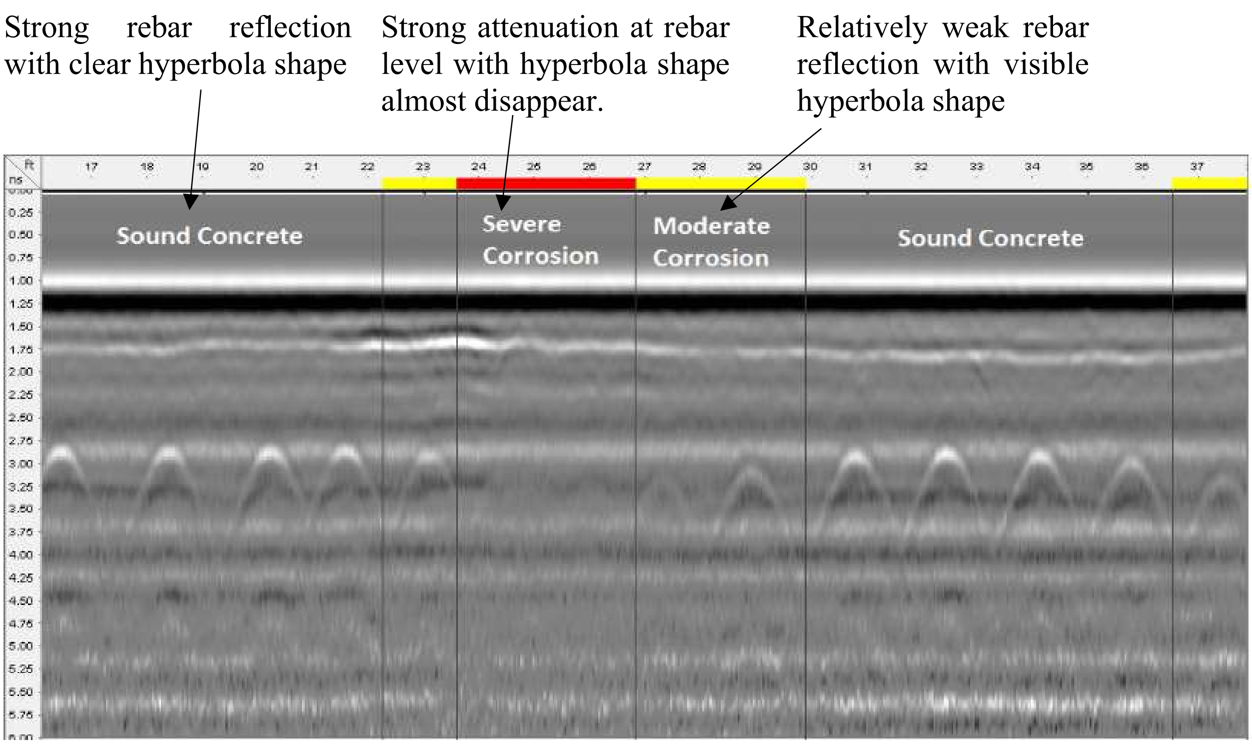


Figure 4-11: Example of a GPR sample data profile [60]

Visual analysis of GPR data has been applied practically. In the past, the individual radar waveforms were analysed visually, which was time-consuming and impractical for bridge inspection. Tarussov et al. [95] later proposed corrosion mapping in RC using GPR profile (A scan/B-scan) image analysis, where visible anomalies are marked based on deterioration criteria. This method was however subjective, and analysts usually battle in defining the sound concrete from deteriorated areas. Recently, an approach to eliminate subjectivity was applied to the numeric and visual analysis methods by Dinh [60]. With the use of the K-means clustering technique, Dinh [60] defined threshold values for condition categories, which are used for corrosion maps with decibel scales to determine the relative level of rebar corrosion. The condition categories with their definitions are illustrated in *Figure 4-11*. Nevertheless, visual analysis is an excellent approach for interpreting GPR data as it identifies corrosion-induced damage in a much more accurate way and eliminates anomalies that are not related to structural defects.

Though bridge decks are usually surfaced with overlays or membranes, GPR is effective with or without asphalt overlays [55]. The method can cover large areas of measurement in a short period of time. GPR results can also be used in the BMS condition assessment to provide bridge condition ratings [93]. The output results of this method which are deterioration maps, provide an outline of corrosion affected areas. These maps are analyzed to obtain percentages of defective areas on the bridge being assessed. The condition of the bridge is then rated based on the assigned total defective quantity of the bridge elements following guidelines as per the Pontis Inspection Coding Guide shown in *Table 4-6*.

Table 4-6: Condition state based on the defective area [47]

Condition state	**Description**
1	No spalls, delamination, or temporary patches on top surface
2	Combined areas of defects is 2% or less
3	Combined areas of defects is more than 2% or less than 10%
4	Combined areas of defects is more than 10% or less than 25%
5	Combined areas of defects is more than 25%

Recently, various researchers have proven that GPR can be used for the early evaluation of reinforcement corrosion in existing RC structures, especially bridge decks [92,93,95]. Zaki et al. [96] found that GPR can detect rebar corrosion at an early stage during the propagation period, before corrosion damages become visible on the concrete surface. Larger wave transit times and lower amplitude zones were associated with increased chloride content and the presence of corrosion products. Solla et al. [97] agreed with what Zaki et al.[96] found, and associated various corrosion-related anomalies to the GPR signals. This study identified corroded rebars, concrete detachments, voids, delamination, areas of higher moisture, and salts content. Corroded rebars were characterized by prominent hyperbolic reflections with stronger amplitude spectrums and positive signal polarity. Hyperbolic reflections near the surface, higher amplitude and signal attenuation were related to delamination, concrete detachment and presence of moisture. The GPR signal can thus be related to changes within the concrete during the whole corrosion process, particularly from the formation of corrosion products to internal cracks formation and propagation, as shown in *Figure 4-12*.

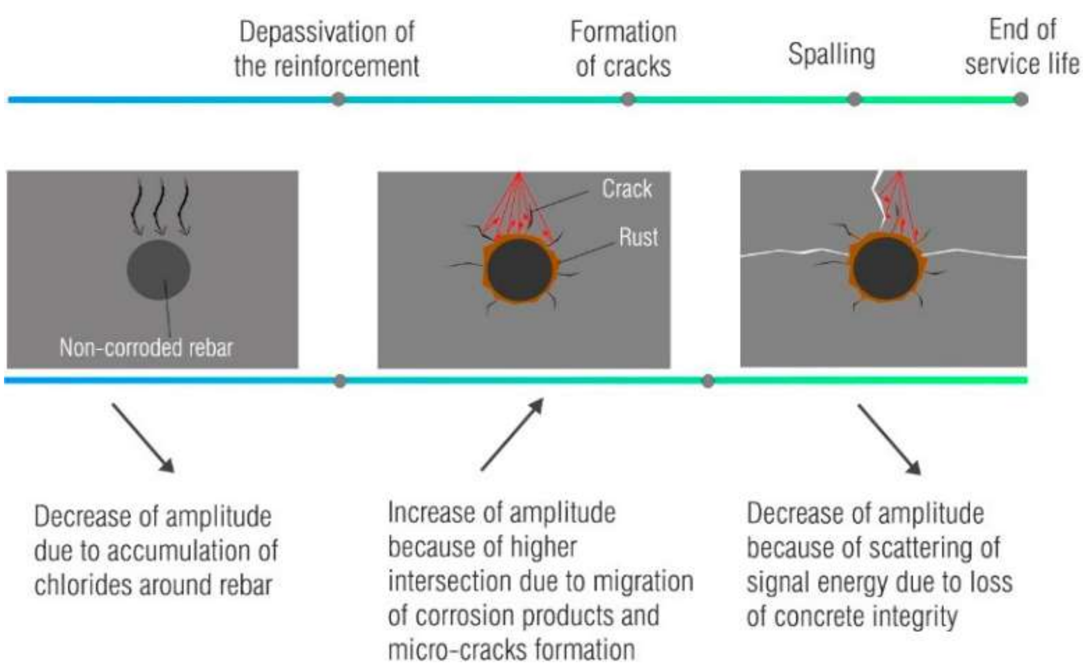


Figure 4-12:Changes in GPR signal during the corrosion process [99]

4.2.4 Thermal method

4.2.4.1 Infrared thermography (IRT)

Infrared thermography (IRT) is a technology used to detect thermal radiation released from materials such as concrete. IRT can be passive or active, the former exposes specimens to artificial heating prior to testing to induce temperature differences, and the latter uses natural heat from solar heating or ambient temperature changes [97,98]. The passive approach is commonly used. The IRT principle of application in RC involves detecting the temperature gradient within the concrete under heat exposure, and because defects have lower thermal conductivity than concrete, they disrupt the heat transfer [101], which causes localised differences in surface temperature. The IR camera captures the emitted radiation and converts it into electronic signals, which are processed to visualise images in terms of colour contrast with different intensities. The images are enhanced and stitched together using custom made codes to form a mosaic thermogram of the entire assessed areas. The mosaic thermogram is then segmented to identify thresholds based on which condition maps are constructed with delineated delamination severity categories.

In the past decades, the IRT method has gained interest in bridge inspection and evaluation (especially on bridge decks and soffits). It has been preferred because of its high speed and coverage at testing, reliable, accurate, and cost-effectiveness [59]. It does not require direct access to the element being inspected as the instrument capture images from a distance using appropriate optical lenses. This eradicates the limitation of traffic disruption and lane closures on bridges when doing the assessment. This method has been used to detect sub-surface defects (such as delaminations, internal voids and cracks) within concrete elements [75,97]. Up to date, IRT has been practised in laboratory and field setting to detect reinforcement corrosion-induced defects [97–100].

Some studies have attempted to examine the IRT applicability in the laboratory setting by measuring the extent of corrosion in RC. Kobayashi and Banthia [103] detected corrosion in RC using induction heating and IRT, which was based on the principle that corrosion products have poor heat conductivity (see *Figure 4-13*) [68] and exhibit diffusion of heat from the heated rebars to the concrete cover. Concrete has a thermal conductivity of 2.7 W/m/°C and corrosion products 0.07 W/m/°C. In RC with uncorroded steel, the higher thermal conductivity of concrete causes the generated heat (through electromagnetic heat induction) in the rebars to diffuse immediately to the concrete cover, which results in high temperature on the concrete surface and lower temperature rises on the embedded bars. However, in corroded RC case, it is the opposite; the corrosion products block the thermal diffusion from steel to concrete, resulting in temperature rise in rebars (as illustrated in *Figure 4-14*) [103].

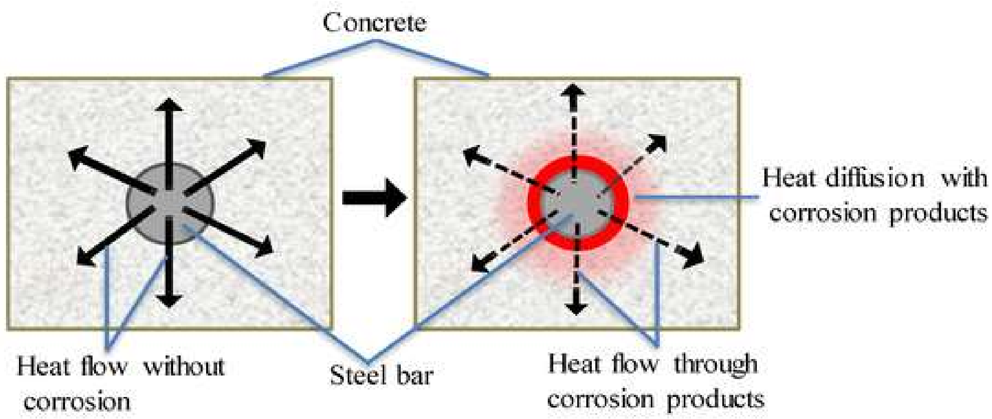


Figure 4-13: Effect of low heat conductivity of steel corrosion products [68]

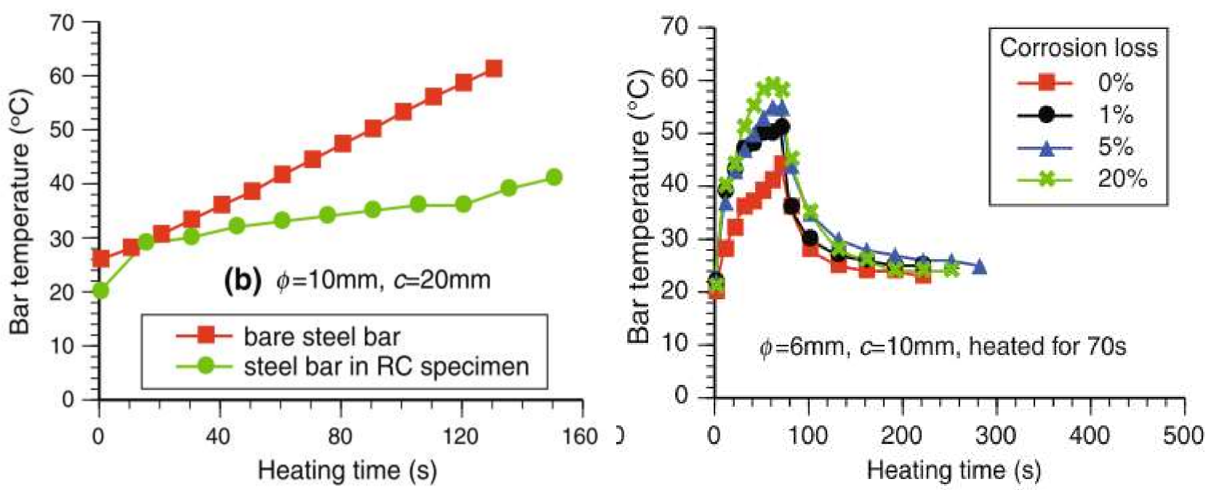


Figure 4-14:Temperature development in steel bar embedded in concrete; compared with bare steel (left) and at various targeted corrosion values (right)[103].

Baek et al. [102] practised a similar experiment and found a correlation of heating rate and infrared intensity with corrosion level. The heating rate and peak infrared intensity tend to increase with an increase in corrosion level. This approach is limited to laboratory settings, its use on in-service structures is questionable as the heat induction to the steel can be difficult and would require breaking the concrete cover for access.

In other studies, IRT has been used to detect rebar corrosion-induced delaminations on concrete bridges decks and other bridge surfaces, following the standardized procedures in ASTM D4788-03 [104]. Omar & Nehdi [100] used the IRT to detect delamination on a deck and soffit of an in-service RC bridge. It was found that the magnitude of thermal contrast correlates with the depth of delamination. An increase in delamination thickness increases the maximum thermal contrast. The potential application of unmanned aerial vehicle (UAV) IRT for the detection of subsurface delaminations was explored by Omar & Nehdi [101] on RC bridge decks. The UAV IRT allowed fast, shallow delamination detection, and delaminated areas were found to have a higher temperature than sound concrete. However, the UAV system can only carry a lightweight IR camera due to its limited payload, and weather fluctuation such as high wind speed affected the UAV stability and image quality.

It was also found that IRT measurements are influenced by various factors [93]. Surface-related factors such as dirt, oil spills, moisture content, rust staining affect the emitted thermal radiation on the concrete surface by causing temperature variation. Environmental factors such as wind speed, humidity and ambient temperature can also affect the test results. Dabous & Feroz [94] indicated that delamination defective areas are more distinct at higher ambient temperatures and humid atmospheres.

The IRT results indicate the extent of deterioration and cover large testing areas in a short period [68]. IRT can provide both qualitative and quantitative results of subsurface defects such as delamination and can be used to assess areas that are difficult to access. Detecting delamination in the initial stages of the propagation period (before spalling occurs) is crucial as it ensures serviceability, safety and optimised maintenance and repair needs. However, it has limitations, such as the techniques' sensitivity depending on weather conditions; it tends to work best when used in the morning or late evening when there is no reflection of sunlight [21]. The defects become difficult to detect when at greater depths, and the test needs to be carried out by highly trained individuals.

4.3 Remote monitoring systems

As discussed in the previous Section 4.2, NDT methods are able to provide semi-qualitative and quantitative information on the extent and progress of reinforcement corrosion in RC structures. These methods can detect early damage in concrete caused by reinforcement corrosion, providing sufficient time for appropriate maintenance and repair interventions to be taken. However, the NDT methods require access to the inspected bridges, with some structural elements usually being inaccessible or difficult to access. Furthermore, measurements need to be taken at regular intervals, which leads to more funds being spent on the arrangement of access (e.g., using cranes) and traffic interferences when doing the tests. It is for these reasons that remote or permanent monitoring may provide a preferred solution.

Remote monitoring systems comprise sensors installed on the bridges and connected to a data acquisition system. These systems have no interference with traffic, except during installation, and they save time as the measurements do not have to be taken in-person, as is the case when using conventional systems. The systems are typically recommended for bridges that are difficult to access for regular inspections. The data obtained from these systems help with planning, optimising and implementing required interventions to prevent premature deterioration of bridges.

Remote monitoring systems are used in new and existing structures where they are embedded prior to concrete casting in new structures and drilled in holes in existing structures at various depths within the concrete cover. This is considered during the initiation and propagation phase of deterioration in the structure's service life. Monitoring systems such as Anode-ladder systems, Expansion ring systems, CorroWatch and CorroRisk monitor the time to corrosion initiation. Multi-ring electrodes provide an indication of moisture content and potentiometric sensors measure chlorides in RC. Hence, these monitoring systems primarily monitor the ingress of the main corrosion parameters (i.e., chloride concentration, oxygen, and moisture) prior to depassivation and during corrosion propagation. The aforementioned remote monitoring systems for corrosion-related parameters used in RC structures are discussed in the following sections.

4.3.1 Time to corrosion monitoring systems

Time to corrosion monitoring systems, as the name implies, monitor the time to corrosion initiation. The methods monitor the ingress of aggressive agents (i.e. chlorides) and indicate the depth of critical chloride content that can depassivates the reinforcing steel and subsequently initiate corrosion. These methods can be used in both new and existing structures. They allow assessment of areas difficult or without access, hence may reduce overall costs of inspection, maintenance, and repair. However, though these methods directly indicate the critical depth of chloride content, they do not provide the absolute chloride content (chloride content measurements) [105].

The principle of these methods employs a macrocell reinforcement corrosion approach, which involves the measurement of current flow between separate anode and cathode areas [106]. With these methods, several anodes are placed at different depths within the concrete cover in relation to the concrete surface. The anodes are made of the same composition as the reinforcing steel; hence, they have similar corrosion behaviour. This is done to ensure they corrode at the same time. The anodes depassivates one after the other once in contact with sufficient chlorides [105], which allows the time to corrosion initiation to be determined continuously. The onset of corrosion of the anodes is determined at any time, provided the cover to reinforcement is known. This is obtained by automatically measuring the electrical currents, voltages, electrical resistances, and temperatures of the embedded sensors. There are four methods that use the afore-mentioned principle: Anode-ladder system, Expansion-ring system, CorroWatch and CorroRisk, and are discussed in the following sections.

4.3.1.1 Anode-ladder system

The anode-ladder system (ALS) was developed in 1986 by the Institute for Building Materials Research in Germany for long-term chloride-induced corrosion monitoring in RC [107]. The system has been in use worldwide since 1990 on RC structures situated in the marine environment and has gained vast experience from on-site installations. This system is embedded in new RC structures before or during concrete placement. It is installed by being placed between the reinforcement cage and the concrete surface, preferably with the outer anode at 15 mm and the inner anode near the reinforcement.

The ALS comprises the equipment shown in *Figure 4-15*, of which detailed specifications on the application, operations, maintenance and quality control are provided by Sensortec GmbH [107]. The system comprises:

- Anode ladder (AL) - Ladder-element with six single anodes and a temperature-sensor,
- Cathode bar - A 40 cm long platinum oxide-coated titanium bar with 8 mm diameter that acts as a counter electrode,
- Cable connection – connecting the sensors to the terminal box,
- Terminal box – stores the sockets for the measuring plug,
- Connection to the reinforcement (CR) – A black steel bar with cable connection to additionally monitor the corrosion behaviour of the reinforcing steel

As previously discussed, the ALS indicates the critical depth of chloride content as six steel anodes that form up the anode ladder (see *Figure 4-15*) depassivates [105]. The system measures potential voltages, electrical current, concrete resistance and temperature (using the PT 1000 temperature sensor) between the six single anodes (A1-A6) [107]. The depassivation of the anodes is usually related to a significant increase in electrical current combined with the decrease in potential using the thresholds shown in *Table 4-7*. These limits are only applicable to non-submerged structures. However, to monitor the corrosion risk in the long term, regular measurements (potential voltage, electrical current and concrete resistance) are taken using a controlled computer at the office about one to two times a year.

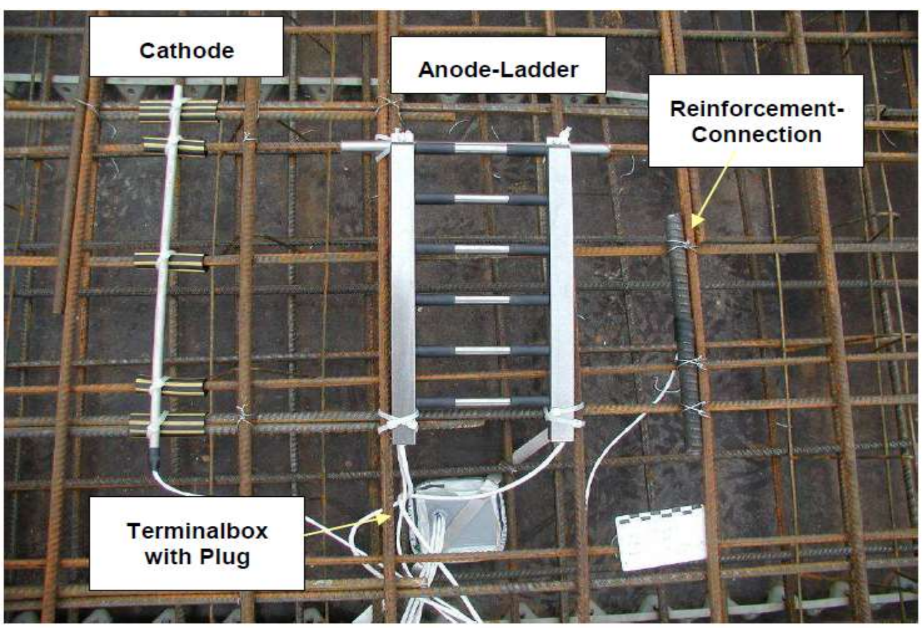


Figure 4-15: Anode-ladder system equipment [107]

Table 4-7: Limit values for anode-ladder result interpretations [107]

Parameter measured	Specified limit values
Potential voltage (mV)	−150 mV
Electrical current (µA)	15 µA at 5 sec after coupling
	1.5 µA in long-term
Concrete resistance (kΩ)	Range from 1 – 10 kΩ

The system has been used successfully to monitor corrosion in new structures in countries like Germany, Austria, Denmark, Hong Kong, Japan, Croatia, Egypt, Switzerland, The Netherlands, Sweden, Taiwan and Australia [105]. In one of the projects in Switzerland, 25 anode-ladder systems were installed in a multi-storey car packing deck to observe the ingress of chlorides [108]. The sensors were placed at high expected chloride exposure sites, which were attached to the upper reinforcement layer prior to casting, as illustrated in *Figure 4-16* (left). The results for this study indicated that the chloride concentration at the steel level was below the threshold, and the corrosion initiation was recognised early in three years after construction, which is shown by the drop in potential and increase in corrosion current (see results of anode A1, year 2010 in *Figure 4-16,* right). Corrosion initiation was detected first at Anode 1 closest to the concrete surface (A1) in 2010 and secondly on Anode 2 (A2) in 2015, while all other anodes remained passive throughout the monitoring time.

The ALS is durable for corrosion monitoring, comprising materials compatible with concrete and corrosion-resistant. Hence, it is typically designed for a service life of more than 100 years [107]. However, the anode-ladder systems are associated with limitations such that the sensors should not be placed in positions where poker vibrators are used, and there should be no other electrical contact within the system apart from the reinforcement connection. In addition, the cathode part of the system cannot be used in submerged structures as it needs to be installed near the concrete surface, where sufficient oxygen is available.

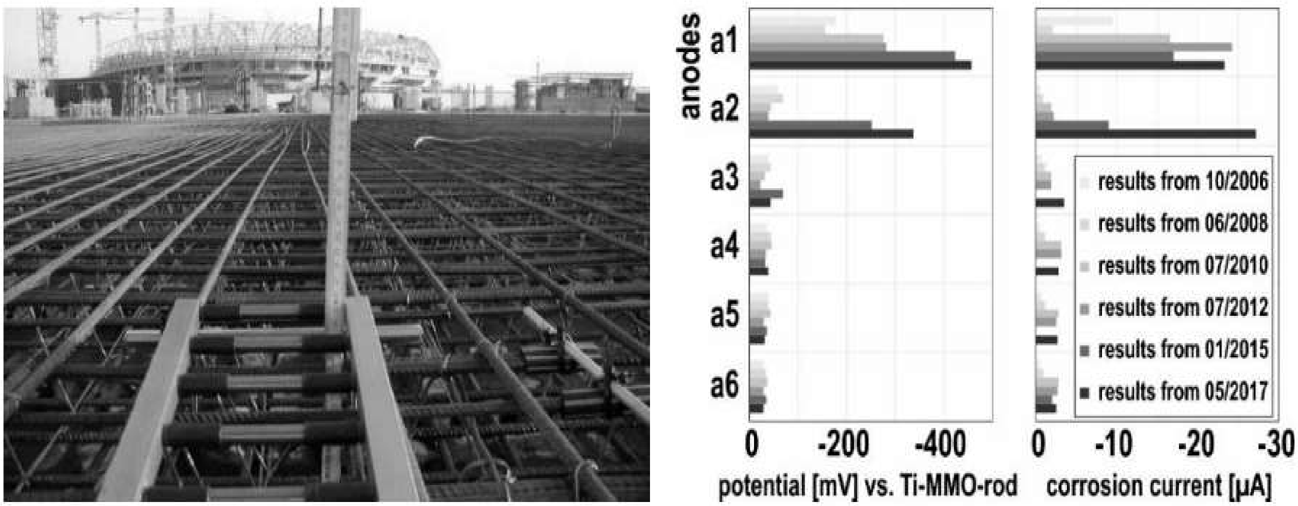


Figure 4-16: Installation of the anode-ladder system (left) and time-dependent development of potential and corrosion currents of an anode ladder (right) [108]

4.3.1.2 Expansion ring system

The expansion ring system (ERS) is used for corrosion monitoring in existing RC structures, i.e. in components of bridges, tunnels or parking structures exposed to seawater. The system determines the time to corrosion initiation in structures where corrosion has not initiated yet. In structures where the propagation period has begun (when the critical depth of chloride content is reached), the corrosion potential and resistance provides the indication of corrosion risk.

This system comprises the expansion-ring anode, a titanium oxide cathode bar and a temperature sensor [109]. The expansion-ring anode comprises six circular steel anodes and the sealing rings (to prevent moisture or chloride ingress) as shown in *Figure 4-17*, which are arranged at different depths of 1-cm-steps from the concrete surface below one another (see *Figure 4-18*). This expansion-ring anode and the titanium cathode bar are installed into drilled holes, typically 5 to 15 cm apart. The expansion-ring anode has a nut on its top part, which allows it to be tightened within the drilled hole. Hence the tension between the sensors and the concrete needs to be ensured for high durable connection and to avoid induced cracking on the concrete. Raupach [103,104] provided the procedures for installing this system into existing RC structures, as illustrated in *Figure 4-19*.

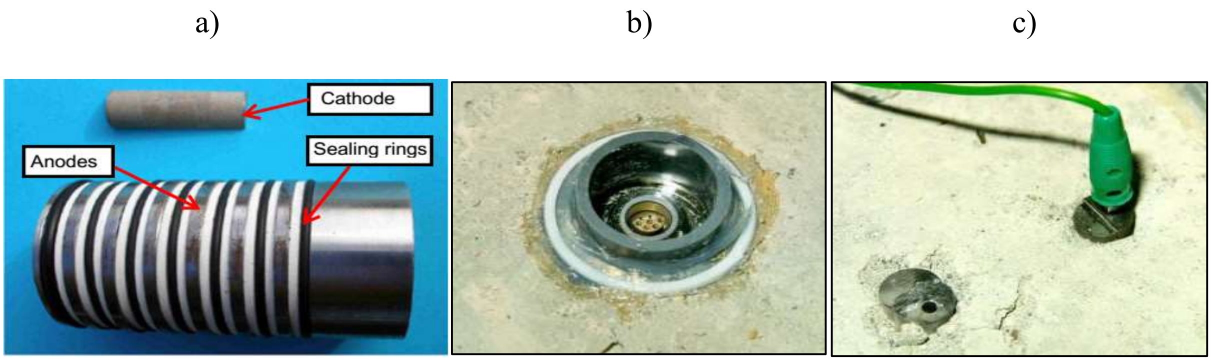


Figure 4-17: An expansion ring system; prior to installation (b), an expansion ring anode after installation (b) and the cathodes (c) [106,107]

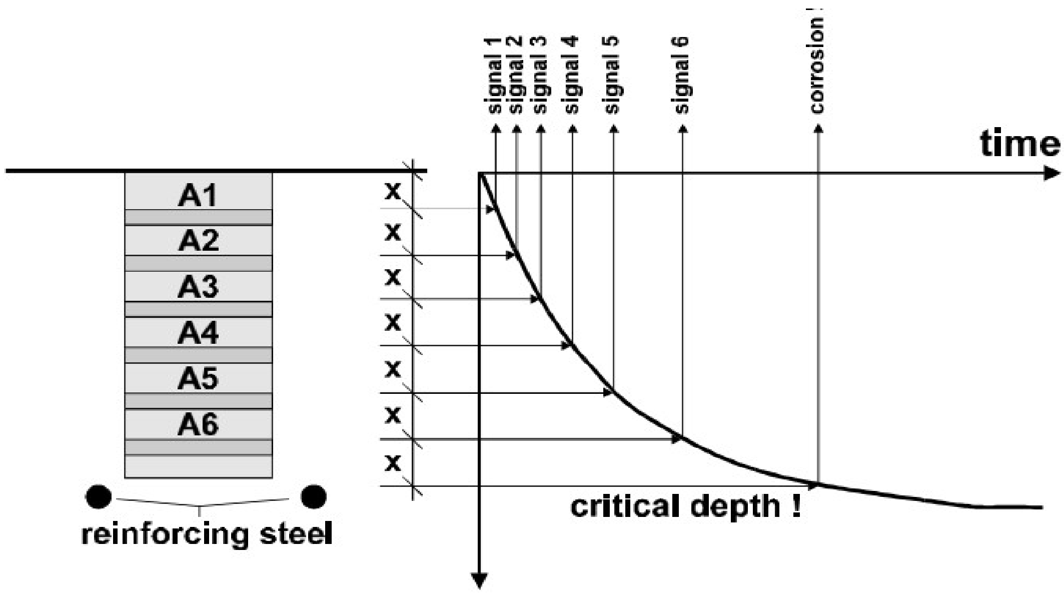


Figure 4-18: Arrangement of sensors to determine the time to corrosion [105]

Measurements are taken automatically by a controlled computer or by using a hand-held instrument (CANIN-LTM) plugged onto the socket integrated on the head of the expansion-ring anode. When the socket is not in use, it is sealed with a protection cap to prevent it from corroding. The protection can also be enhanced by the rust formed after the depassivation of the first anode [106]. These measurements use the specified thresholds shown in *Table 4-8*, which apply to unsubmerged structures only. Measurements are done one to four times a year, and as long as the critical chloride content has not reached the surface of the outer anode (A6), all electrical currents between the anodes and the cathode bar are usually minimal [105].

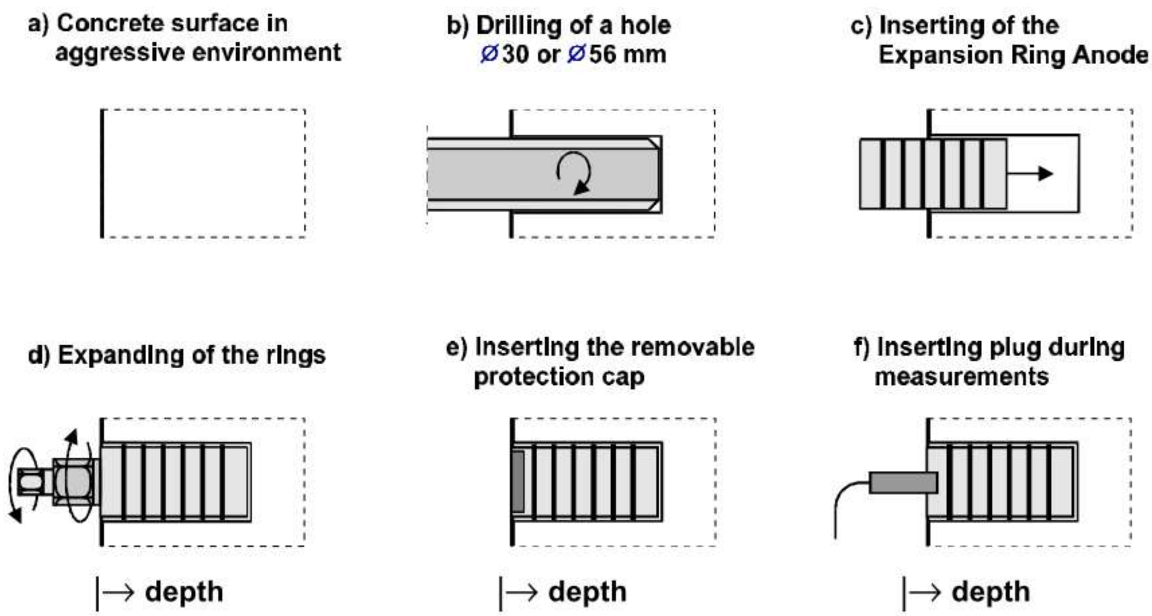


Figure 4-19: Expansion-ring anode installation and measurement procedures [105]

Table 4-8: Limit values for expansion-ring system result interpretation [111]

Parameter measured	Specified limit values
Voltage (mV)	−400 mV
Electrical current (µA)	10 µA at 5 sec after coupling
	1.5 µA in long-term
Concrete resistance (kΩ)	< 100 kΩ (usually range from 1 – 10 kΩ)

Extensive experiments in the laboratory and on-site have been carried out using the ERS to investigate its practicality and applicability. For example, this monitoring system was used in an RC bridge in Germany under the Smart Structures project for three years [111]. Time-dependent measurements were taken for electrical currents at 5 seconds after coupling, and voltage for all anodes against the cathode; the concrete resistance between neighbouring anodes and temperature as illustrated in *Figure 4-20*. Considering the specified limits in *Table 4-8*, it was found that the first ring of the expansion-ring anode was corroding; hence the critical depth was determined as 1-2 cm. In the last measurements, the concrete was dry, which was shown by increased resistances with reduced currents and voltages.

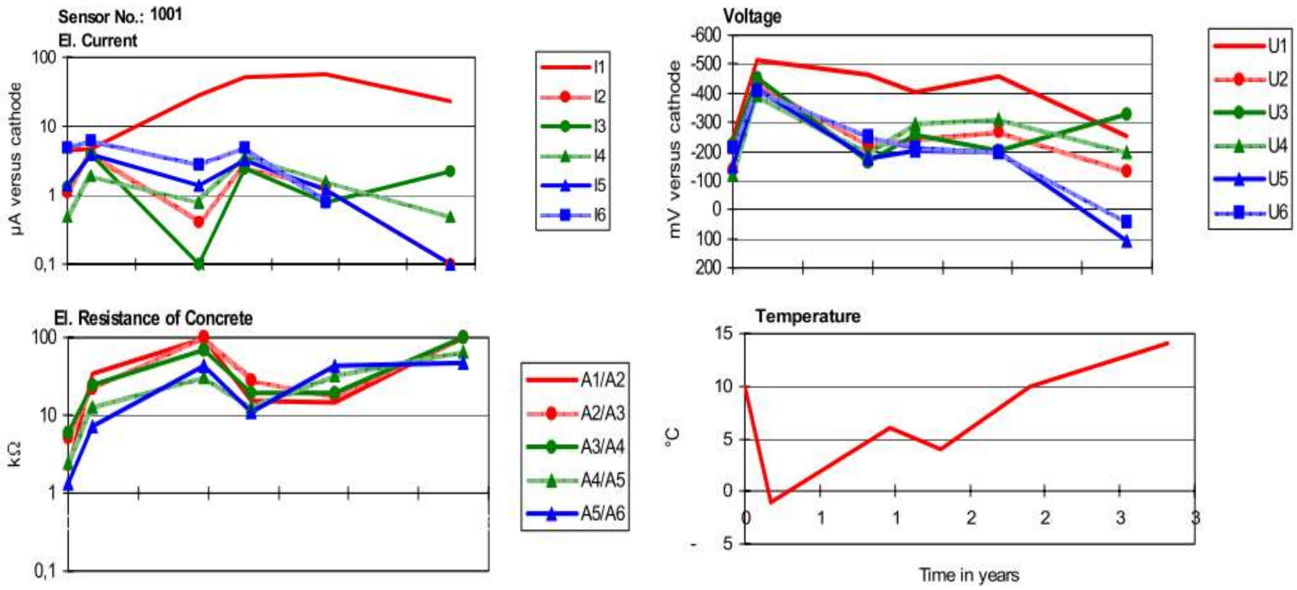


Figure 4-20: Measurements from an expansion-ring system in an RC bridge [111]

The system cannot be installed near the end or at the edges of concrete elements, and it is not suitable for use on submerged elements [109]. Nevertheless, Sensortec developed a technology that uses the same principle for application on submerged elements. That technology is referred to as the Drill Core Anode [112].

4.3.1.3 CorroWatch and Corrorisk sensors

CorroWatch (CW) and CorroRisk (CR) sensors are monitoring technologies developed by the company Force Technology to provide early warning on corrosion initiation in RC structures. Force Technology has been involved in the steel corrosion problems in concrete since the 1970s and offers a wide range of specialities in the aspects of concrete condition assessment. The CW sensors are applicable in new structures of which are cast in the cover concrete, whereas CR sensors are installed in existing structures exposed to aggressive environments or inaccessible for inspection. These two systems are explained hereafter.

The CW is a multisensor system comprising four black steel anodes and a ring cathode, as shown in *Figure 4-21* [75]. During installation, its anodes are placed at different defined locations from the concrete surface, which are flexible to be adjusted within the specified concrete cover depth. Though there are no appropriate standards followed when using this system, Force Technology provides specifications regarding the type, dimensions and size of the equipment and cable connection details.

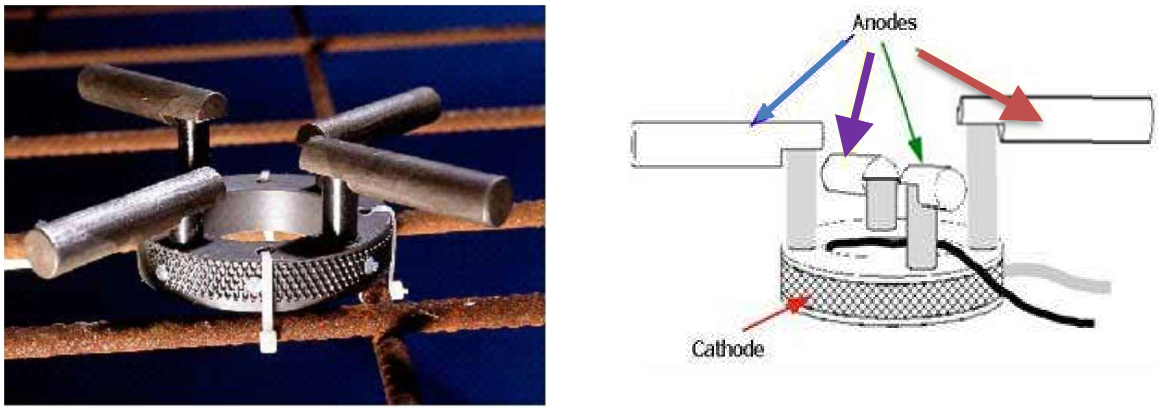


Figure 4-21: CorroWatch sensor with four anodes and a cathode [75]

Measurements in this system are carried out continuously using a remote monitoring modem or automatic data logger attached to the system. The sensors measure corrosion parameters such as the current or potential of individual anodes against the cathode at various depths over a period of time. It should be noted that the current measured in this system is the galvanic currents flowing between the black steel anodes and the cathode, and not corrosion currents. Corrosion initiation is then indicated by the increase of currents and more negative potentials. These measurements are interpreted against the commonly used corrosion current and HCP thresholds outlined in previous Section 4.2.1.1.

CW sensors have been demonstrated to monitor corrosion risk in inaccessible existing RC structures such as bridges and tunnels in marine environments. One of the recent applications is the long-term field corrosion monitoring in the supporting structures of the China Xiamen Xiangan Subsea Tunnel [113]. The real-time data of the installed anodes for six years on corrosion current (I_{corr}), potential (E_{corr}) and temperature (T_{corr}) indicated the feasibility of CR for on-site corrosion monitoring. The supporting structures were found to be safe as all I_{corr} values of the first anode were within the limit of 0.2 A/cm^2 (0 ~ 0.05 A/cm^2). The E_{corr} and T_{corr} values fluctuate between −0.5 to 0.5 V and 20 to 35 °C, respectively, as shown in *Figure 4-22*. The fluctuations of E_{corr} values were attributed to instantaneous changes in the vicinity of rebar and concrete and the T_{corr}

values to seasonal changes. Furthermore, these changes were more susceptible to anodes near the concrete cover and the rebar surface.

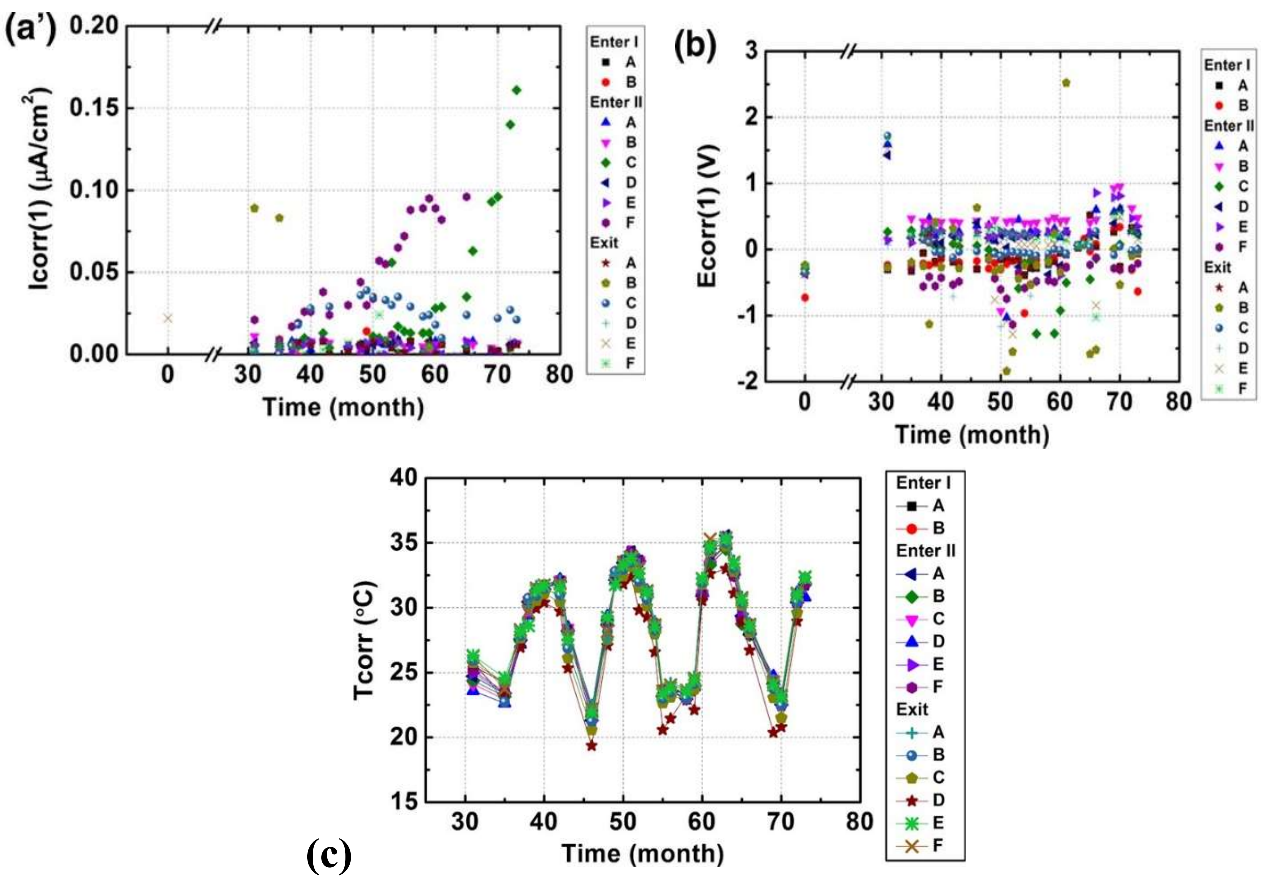


Figure 4-22: Variation of I_{corr} (a), E_{corr} (b) and T_{corr} (c) with monitoring time at different locations [113]

CR sensors also detect reinforcement corrosion before it initiates. The system consists of metal nails with an activated titanium mesh cathode and a manganese oxide (MnO) electrode. The metal nails are of the same material as reinforcements to ensure that they corrode similarly when sufficient chlorides reach the level of individual anodes [67]. Like CW sensors, CR sensors are drilled in the concrete cover between the surface and the outermost layer of reinforcement at various well-defined depths with the cathode in the middle, as shown in *Figure 4-23* [75].

To predict the reinforcement corrosion initiation, the current or potential between the individual anodes and the cathode is measured with a specially designed datalogger (CAMUR II). When corrosion starts, the current will increase significantly, and the potential decreases. These measurements are evaluated similarly to CW sensors. However, all anodes in this system need to be connected to the monitoring box, accessible on the concrete surface.

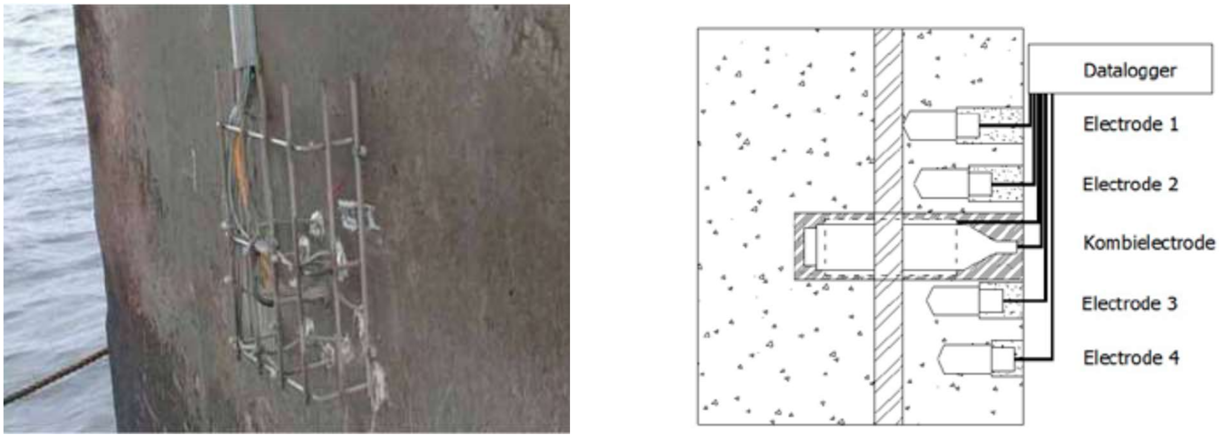


Figure 4-23: CorroRisk installation (left) and design (right) on a marine bridge pillar [75]

4.3.2 Moisture content monitoring systems

Apart from monitoring the critical chloride content depth or the time to corrosion initiation, other conditions necessary for the acceleration of corrosion can be monitored. One of these conditions is moisture, which stifles the corrosion rate when lower than the critical limit value [114]. This parameter can be indirectly measured through electrical resistivity as it indicates moisture content in the concrete [115]. A significant drop in electrical resistivity indicates the vulnerability of the concrete to corrosion sustaining agents, i.e., chloride ions, oxygen and moisture.

Conventionally, the moisture content is determined periodically by measuring the weight loss of segments from core samples extracted from different depths on the in-service structure. This method is destructive and time-consuming. New technologies have been developed that can indirectly measure moisture in concrete and are available on the market. The Multi-Ring Electrode (MRE) system is one of the technologies used for more than 20 years to monitor in-depth electrical resistance in new and existing structures [110,112]. Sensortec develops the system, can be installed before concrete placement in new structures and drilled and anchored with mortar in existing structures. These sensors need to be connected to a measuring and recording device, from which data can be obtained automatically. Measurement nodes can also be used for automatic data acquisition.

The MRE system consists of nine stainless steel rings spaced 5 mm apart, with 2.5 mm plastic insulators between them [111,112]. As a function of distance to the concrete surface, the MRE measures the electrical resistance between any adjacent rings and provide a resistance profile across the sensor depth, as shown in *Figure 4-24*. These resistivity readings are converted to moisture profiles using concrete-specific calibrated curves (a relationship between electrical ER and moisture content based on the type of concrete and temperature).

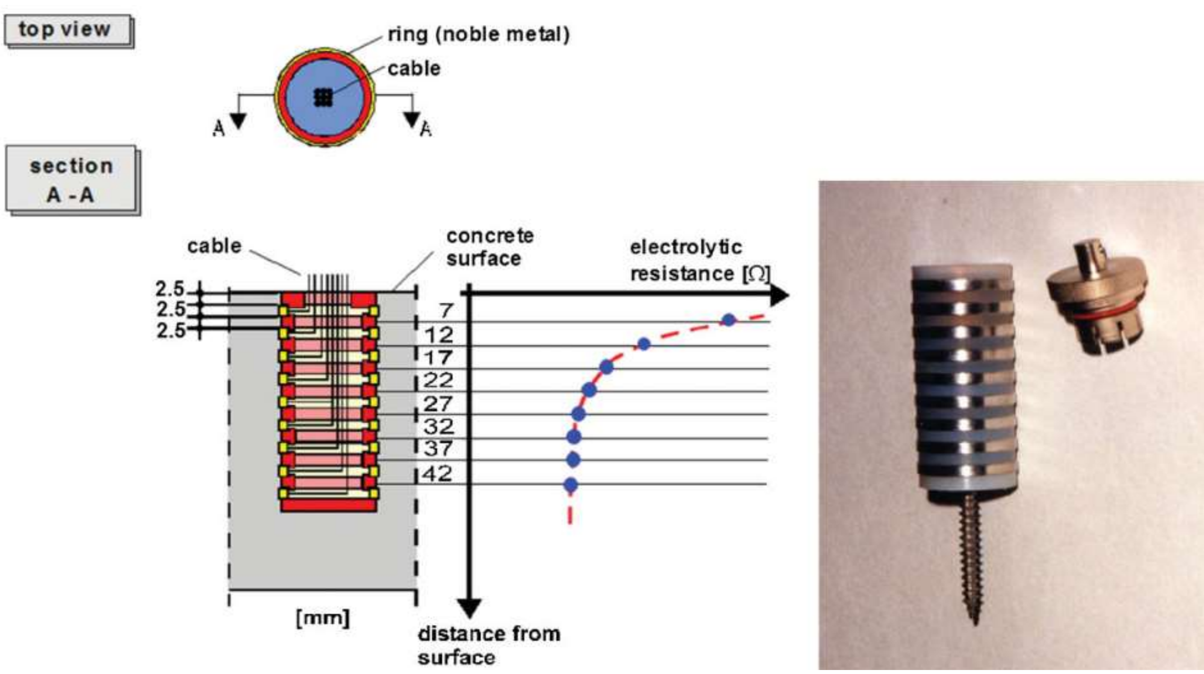


Figure 4-24: The Multi-Ring Electrode sensor - top and section view [114]

Babler et al. [113,114] reported on the first experimental results of using the MRE technology in the laboratory. They found that when drying the concrete is expected depending on the climate condition, the increase in resistance of every ring of the MRE reflects the moisture content. It was observed that resistance at all rings of the MRE increased continuously as shown in *Figure 4-25,* with the ring near the concrete surface (5 – 7.5 mm) drying out faster than the rings located deeper (30 – 32.5 mm). For the past 20 years, this technology has been explored on different projects; however, no advancement has been done since its first use in 2000.

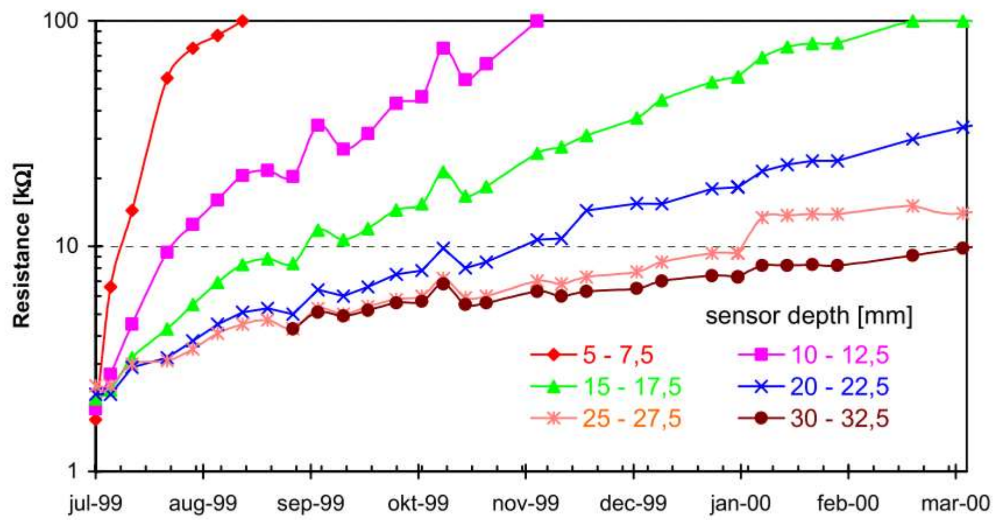


Figure 4-25: Resistance readings at MRE during exposure at room temperature [113,114]

Moisture content governs the initiation to reinforcement corrosion and the progression of corrosion after depassivation. If moisture content is reduced in RC, reinforcement corrosion is stifled. Hence, continuously monitoring the moisture at an early stage will help control reinforcement corrosion. The resistivity data obtained using the MRE can also be used to evaluate the rates over time for such parameter, based on sensor depth e.g., after one year of data collection [114]. In addition, using the MRE for monitoring moisture is beneficial as it provides the ER measurements, in case the NDTs (such as Wenner-four probes Resipod) are decided not to be used on a certain project. However, this system has a limitation of considering the properties of the anchoring mortar used in installation of the system in existing structures in the interpretation of the results.

4.3.3 Chloride content monitoring systems

As discussed in Chapter 2, free chlorides in concrete pore solution are responsible for reinforcement corrosion initiation. This makes free chloride concentration one of the crucial parameters that need to be monitored in RC structures. The concentration of chlorides at the steel level determines if the corrosion has initiated or not; hence, it influences the durability of RC. Conventionally, chloride content is measured destructively and involves testing core samples taken from the existing structure, resulting in chloride concentration obtained at different depths, as shown in *Figure 4-26*. This method is generally time-consuming, expensive and samples are sometimes not representative of the whole structure [35]. However, it is the commonly used method for condition assessment of structures situated in the marine environment.

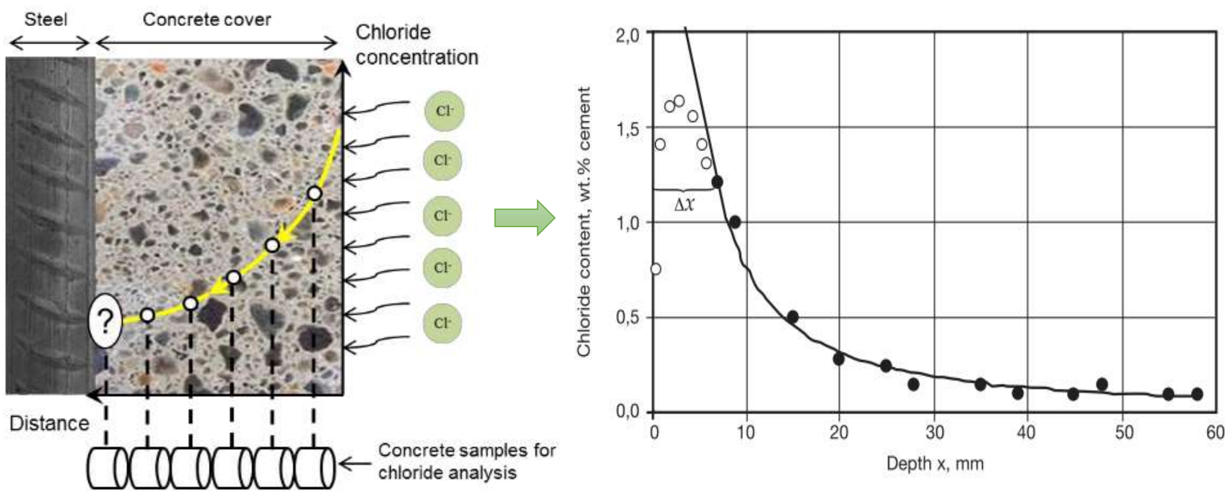


Figure 4-26: Conventional method of measuring chloride content; extracting samples at various depths to obtain chloride profile [2,116]

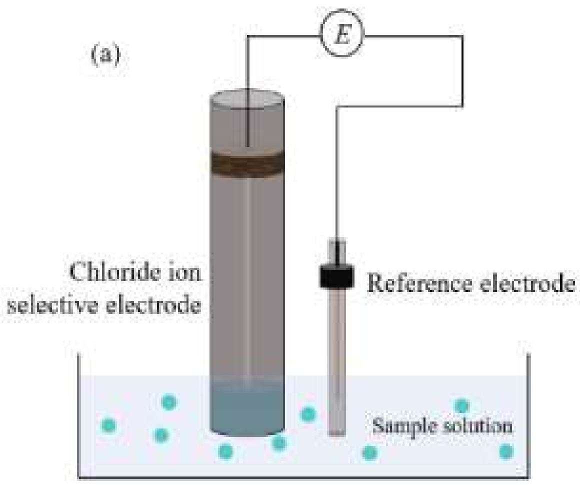
Recently, new methods have been explored that can non-destructively measure and allow continuous monitoring of chloride concentration in the concrete. Potentiometric sensors or Ion Selective Electrodes (ISE) are some of the systems that have been used to determine free chlorides in RC [116–118]. These embeddable sensors are suitable for installation in existing structures, especially in RC elements that are difficult or inaccessible for inspections. In the field, the system comprises the embedded ISE (usually silver/silver chloride electrodes (Ag/AgCl) supplemented by a reference electrode (RE) and connected with a potentiometer [122] as shown in *Figure 4-27 (a)*. The ISE sensor acts as a working electrode whose potential can be measured and the RE as a counter electrode with a fixed potential. The potential difference between the two electrodes can thus be associated with the dissolved ion concentration, such as chloride ions.

The change in potential measurements between an ISE sensor and RE (*E* in V) is estimated using the Nernst equation illustrated by Equation 4.5. This established a linear relationship between the chloride activity and the potential difference between an ISE sensor and RE. Hence, allows the estimation of chloride activities (a_{cl}^{-} in mol/dm^3) [119,120]. The chloride concentration (C_{cl}^{-}) can then be determined using the chloride activity coefficient (γ_{cl}^{-}) in Equation 4.6.

$$E = E^0 - \frac{RT}{nF} In[a_{cl^-}] \qquad \text{Equation 4.5}$$

$$a_{cl^-} = C_{cl^-} . \gamma_{cl^-} \qquad \text{Equation 4.6}$$

Where E^0 is the standard electrode potential, R is the gas constant (which is 8.3144 J/(K.mol)), T is the absolute temperature (K), F is the Faraday constant (which is 96.487 KJ/(V.mol)), and n is the number of electrons involved in the reaction.



(b)

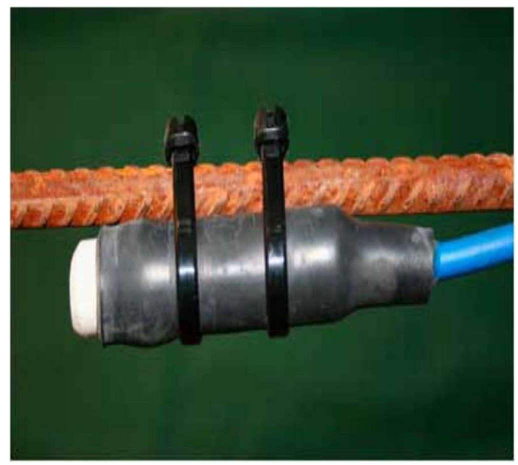


Figure 4-27: An ISE system (a) and ERE 20 reference electrodes (b) [122]

The Ag/AgCl-ISE has been a widely used and commercially available sensor that can detect chlorides non-destructively in concrete [118–121]. This sensor consists of a silver wire (Ag) coated with a silver chloride (AgCl) layer. In these studies, the Ag/AgCl-ISE sensor is characterized by high sensitivity, stable and robust, easily gaining its potential back after being polarised, has the opportunity for wireless detection and requires low-cost preparation. Its performance has been evaluated in high alkaline environments, long time exposure to high chloride concentrations and its reaction to pH changes.

The applicability of this sensor has been tested in the laboratory setting by Angst et al. [120], Seguí Femenias et al. [121] and Tian et al. [124] on mortar specimens. Angst et al. [120] found that Ag/AgCl ISE have a good long-stability in a high alkaline environment such as concrete, hence can be successfully used to measure chloride ion activity. The authors indicated that in the absence of chlorides, the potentials are affected by pH, though the effect is revocable once in contact with chloride. Seguí Femenias et al. [121] agreed to use these sensors as an early warning for corrosion initiation by placing them at various depths within the concrete cover. Furthermore, the sensors detected chloride ions under dry-wet cycle conditions, which was more sensitive than the anode ladder system [124].

Other potentiometric sensors include the ERE 20 reference electrodes (see *Figure 4-27, (b))* developed by Force Technology [75]. These sensors are used to monitor the presence of chlorides in the concrete embedded in the cover concrete of new or existing structures. The electrode is a half-cell of manganese dioxide (MnO_2) in a chloride-free alkaline solution and measures corrosion potential, of which data are remotely monitored by modem. This sensor is designed for a service life of 30 years, it is commercially available and has been used in various corrosion monitoring projects.

4.4 Discussion

Knowing that the reinforcement corrosion process of a typical RC structure consists of three stages; initiation, propagation and acceleration stages, various methods have been looked at that can provide early detection of corrosion damages before they become severe. This is because reinforcement corrosion damage becomes evident on the concrete surface only after significant damage has already occurred.

Research on advanced technologies that enable the detection of corrosion at its early stages has been increasing and evolving since the 1990s. Of the reviewed publications (see Reference list in Section 8) on corrosion monitoring, 77 % have been published in the last ten years, as shown in *Figure 4-28*. Monitoring methods categorised into visual inspection, NDTs and remote corrosion monitoring methods have been reviewed and discussed. Consideration was given to monitoring methods used to monitor and evaluate reinforcement corrosion in RC structures (specifically bridges). These methods can be incorporated into the inspection process to evaluate internal damage such as rebar corrosion, internal formation and propagation of cracks, and delamination.

Regardless of its limitations of being subjective and only detecting visible defects, visual inspection is still used in corrosion monitoring. It is the basis of condition assessments in BMSs, and effort should be made to reduce the subjectivity of its measurements. In addition, when other corrosion monitoring methods supplement visual inspection, its limitations are eliminated.

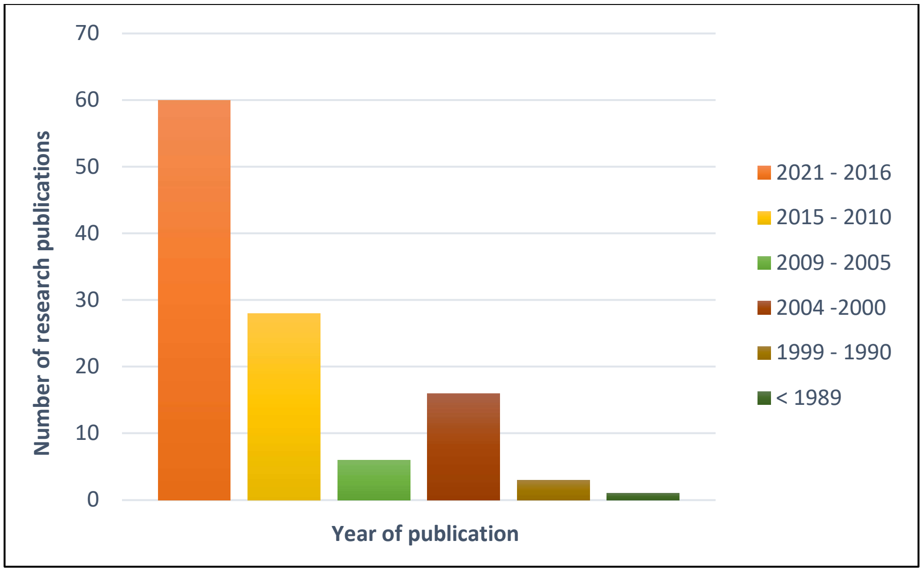


Figure 4-28: Corrosion monitoring research trend (based on the references reviewed in this research)

NDT corrosion monitoring methods provide indirect measurements that are related to different concrete and rebar properties. They can take measurements without any harm to the structure being tested. These methods are found to be applicable for both laboratory and field assessments in new and existing structures. Due to advancements of being portable, the addition of new software and battery-operated tablets (or small computers), NDTs have become popular and available to the communities of engineers and researchers.

NDTs help detect both surface and subsurface defects associated with reinforcement corrosion in RC structures. For example, *Figure 4-29* shows how different NDTs can be used for the condition assessment of a bridge during its service life. As depicted, prior to depassivation of rebars, HCP provides an indication for likely corrosion activity of the actual rebar and the chloride content indicates if corrosion has initiated or not (if the chloride concentration is within or above the threshold limit). LPR determine the actual steel reinforcement corrosion rate and ER influences corrosion rates when the rebar starts corroding. GPR signal is related to changes within the concrete during the corrosion process, particularly from the formation of corrosion products to internal cracks formation and propagation. In addition, IE, UPV, IRT and hammer sounding determines delaminations, internal cracking and voids during the propagation period. The aforementioned methods can detect corrosion at an early stage during the corrosion initiation and propagation, and others (visual inspection) can detect corrosion damage indicators during the acceleration stage.

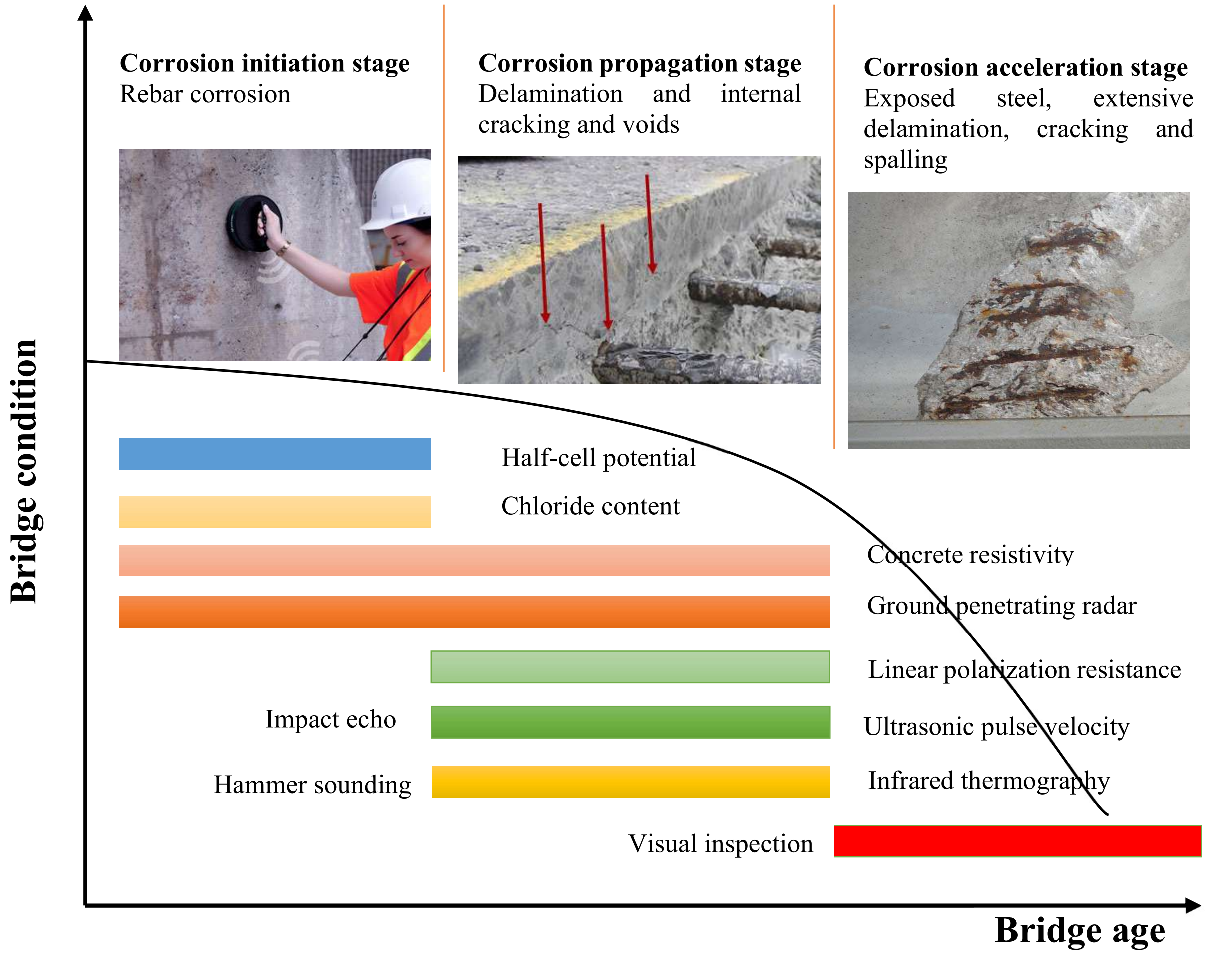


Figure 4-29: Bridge deterioration with time vs NDT methods (images adapted from [72,122])

In the last decade, these methods have evolved from simple methods such as hammer sounding to technical and complex methods such as elastic wave methods in concrete structures. The NDTs reviewed in this study include electrochemical, elastic wave, electromagnetic and thermal methods.

Electrochemical methods have been widely used to monitor corrosion of steel reinforcement in RC because of particular advantages (see *Table 7-3* in Appendix B). The HCP, ER, and LPR techniques have been the utmost used methods to measure the corrosion of reinforcing steel in concrete. All three methods are useful for detecting corrosion at an early stage. The HCP measurements indicate the likelihood of reinforcement corrosion in concrete, whereas the LPR indicates the corrosion rate. ER supplement HCP or LPR measurements, hence its used to help with their result interpretation. These techniques are mostly used under laboratory conditions and very few used in field conditions. This can be attributed to the partially destructive characters, such as the requirement of reinforcement connection in HCP and LPR measurements. Drilling holes for connection purposes during testing makes the whole process destructive and time-consuming.

The Giatec iCOR has a unique ability to perform 3-in-one measurements (corrosion rate, HCP, and ER). It is also wireless and does not require a connection to the steel when testing. Using this one device for all three parameters saves time and reduces the overall costs and resources needed to carry out these tests in the field. According to [76], this advanced technology also allows easy reporting, exporting and sharing results, enabling fast and efficient condition assessment and subsequent decision-making.

The elastic wave methods discussed in this study included the IE, UPV and AE. These methods are non-intrusive, quick, reliable and use lightweight, portable and simple equipment for their measurements. Following the wave propagation principles, these methods detect and locate flaws such as cracks, voids, honeycombing and delamination. However, the three methods differ in how the wave is introduced on the concrete surface during testing. Wave propagations are produced from external impacts in the IE and UPV method, whereas in AE, waves are exerted from within the concrete. The interest in using elastic wave methods for corrosion assessment is a new approach that has emerged in the past decade, and it is still developing. Out of the three methods, IE has limited studies on monitoring of reinforcement corrosion, UPV is still used for its conventional purpose of determining concrete quality, and more research is focused on AE for remote monitoring of ongoing corrosion damage such as cracks formation and propagation in RC structures.

The covermetre and GPR are electromagnetic methods, used to locate reinforcement and cover thickness. They are used for the same application and can all help mark out the measurement grid that is essential to other monitoring methods such as ER, HCP and LPR. However, GPR can only be used on horizontal structural elements such as decks and slabs. A covermetre on the other hand, is simple to use and easy to interpret.

Furthermore, GPR has a long history in condition assessments of RC structures of which its mostly used for locating reinforcement and determining cover thickness. It has overcome the limitation of assessing large areas that requires time for inspection. However, its use in the evaluation of reinforcement corrosion is still in progress. The results from laboratory studies show that the influence of the corrosion process on GPR signal is not fully illuminated yet, and there is no clear relationship reported between the laboratory studies and the field assessments. Hence, more investigation is needed in this regard.

IRT is the only method under thermal methods that applies to corrosion monitoring of RC. This method has been used conventionally to detect sub-surface defects. However, the IRT method has gained interest in bridge inspection and evaluation (especially on bridge decks and soffits). It has been preferred because of its high speed and coverage at testing, reliable and accurate. Because it does not require direct access to the element being inspected, it makes it an appropriate method for condition assessments of inaccessible areas. This also eradicates the limitation of traffic disruption and lane closures on bridges when carrying out the tests. Its application for early detection of reinforcement corrosion is still in the development stage and needs to be understood more through research.

On the remote monitoring systems, six systems have been reviewed: Anode Ladder System (ALS), Expansion Ring System (ERS), CorroWatch (CW), CorroRisk (CR) sensors, Multi-ring Electrode (MRE), and potentiometric sensors. *Table 4-9* shows a comparison of the six systems considering their application, principle and parameters measured. It has been observed that the ALS, ERS, CW and CR measures the critical depth of chloride content, thereby determining the time to corrosion initiation. These systems use the same principle of six anodes placed at various depths in the concrete cover corroding one after the other and based on the macrocell corrosion principle. They measure the same corrosion parameters; however, they do not provide chloride content measurement (except potentiometric sensors).

The measurement results for the ALS and ERS are interpreted based on defined threshold limits that are based on conventional concrete and environmental conditions in Europe and only applicable to unsubmerged structures. Sensors such as temperature and moisture sensors are sensitive to changes in the conditions in RC structures. For example, the Ag/AgCl is very sensitive to chloride ions and dry-wet cycles. Researches and careful calibration tests thus are needed to assess their applicability in South African conditions.

Table 4-9: Summarized comparison of the remote monitoring systems

System characteristics	ALS*	ERS*	MRE*	CW*	CR*	ISE* sensors
Application	-	-	-	-	-	-
Used in new structures	X	-	X	X	-	-
Used in existing structures	-	X	X	-	X	X
Principle	-	-	-	-	-	-
Anodes placed at various depths in the concrete cover	X	X	X	X	X	-
HCP technique (measurement of anode vs cathode)	X	X	-	X	-	X
Measured corrosion parameters	-	-	-	-	-	-
Potential voltage (V)	X	X	-	X	X	X
Electrical current (μA)	X	X	-	X	X	-
Concrete resistance (kΩ)	X	X	X	-	-	-
Temperature (◦C)	X	X	-	-	-	-
Moisture content	-	-	X	-	-	-
Chloride concentrations	-	-	-	-	-	X

*ALS – Anode Ladder System
ERS – Expansion Ring System
MRE – Multi-ring Electrode
CW – CorroWatch
CR – CorroRisk
ISE sensors –Ion Selective Electrode sensor

Observed from *Table 4-9,* some remote monitoring systems are applicable to new structures (e.g. ALS, MRE, and CW) and others in existing structures (ERS, MRE, CR, and ISE sensors). In new structures, the systems are installed prior to concrete placement during the construction stage. In South Africa, the application of these systems is recommended for any RC bridge that is to be built in the future, as currently, the construction of new bridges is very limited. In existing structures, the systems need to be installed or drilled into the concrete. These systems are essential as the majority of South African bridges have been in service for a number of years. Hence, the systems can be used to monitor deterioration progression, existing defects and repaired sections.

This analysis shows that both NDTs and remote monitoring technologies are common and well-established monitoring methods for corrosion related parameters. Most researchers focused on these methods. From different studies, various available technologies are used for measurements and diagnosis of reinforcement corrosion related parameters. However, there is no acceptance or agreement regarding which technology can accurately evaluate the corrosion rate or determine the risk of corrosion. It is thus recommended that integration of different methods for one purpose should be considered for experimental projects to ensure reliability with the measurements on corrosion condition of the embedded steel. Integration methodology needs to be practised to overcome their individual limitations.

Though more complex methods such as IRT have been researched, they have hardly been practically in the field due to their complexity. Some techniques have become normal practices as they have been used and installed in various projects for more decades. For example, HCP measurements are commonly practiced on RC structures suffering from corrosion, especially due to chloride contamination. Also observed from the discussion on each method, it is evident that most methods have been investigated in the laboratory setting, but the practicality and applicability of these methods to existing real structural elements are very limited. These assessments are often avoided due to environmental and weather-related problems. More studies are thus needed to evaluate the practicality and applicability of existing corrosion monitoring methods to ensure their acceptance in bridge inspections. Other methods also require a high degree of experts for accurate results; hence most agencies try to avoid such consequences.

However, in the Southern African context, corrosion monitoring methods applicability is very limited to visual inspection. Seemingly because these methods required more time to be accepted as reliable for use by different agencies, and visual inspection has been the traditional and norm method that is simple to be carried out. Consideration of detailed assessment e.g., using NDTs are only considered after certain events such as accident, flood or any unexpected damage. Hence, more work is needed in demonstrating the feasibility of corrosion monitoring methods to bridge managers and decision-makers.

The use of up-to-date monitoring methods for condition assessments allow earlier detection of problems, hence, sound maintenance and damage prevention. This simultaneously mitigates the costs needed to spend on maintenance and repair. In addition, monitoring methods enhance the speed and scope of condition assessment, provide reliable and wide-ranging data and reduce traffic interruption when taking measurements. Hence, will improve the bridge inspection process in BMSs.

Chapter 5

5 Considerations for integrating monitoring systems into the Struman BMS

5.1 Introduction

This chapter presents considerations for the possible integration of corrosion monitoring methods into the Struman BMS. The suggested approach uses inspection data obtained (periodically or continuously) from visual inspections and any selected corrosion monitoring methods specific for a particular project. In order to use data from selected monitoring methods in the BMS, condition ratings for all corrosion parameters measured need to be defined based on threshold limits accepted in the industry and/or literature. This will facilitate the development of Integrated Condition Ratings for the bridge elements under consideration. In this chapter, previous studies that used integrated methods for condition assessments (with some methods used in the BMS) are presented, followed by outlining the proposed methodology for integrating monitoring methods into the Struman BMS.

5.2 Previous studies on integrated methods

As discussed in Chapter 4, various methods can be used to monitor reinforcement corrosion in RC bridges, of which each technique can be used separately or to complement other methods. Each of these methods has its own strengths and limitations. *Table 5-1* summarises the advantages and limitations of the reviewed monitoring methods (see more details in *Table 7-3* of Appendix B). Combining different methods in assessments maximises the capability of each method and assists in compensating respective limitations. Hence, improving the validity of condition assessments of RC structures.

Table 5-1: A summary of advantages and limitations of monitoring methods for RC structures

Methods	Advantages	Limitations
Half-cell potential	• The equipment used are lightweight and portable. • Indicate the probability of corrosion • Not expensive	• It only evaluates corrosion risk and do not assess corrosion rate. • Its semi-intrusive as it requires connection to the rebar • It is difficult to establish a connection to steel when the concrete cover is large. • It cannot give reliable results with coated concrete surfaces or in the presence of overlays e.g., on bridge decks • Not applicable to epoxy-coated bars • Require lane closure
Concrete resistivity (Wenner configuration)	• Non-destructive, easy, portable and fast technique • Indicates areas of high corrosion risk • Provide insight on concrete durability	• Used in conjunction with HCP, hence cannot be used alone for assessment • Influenced by the concrete exposure conditions and its composition. • The presence of reinforcement in the test region can cause short-circuiting
Linear Polarization resistance	• The equipment used are lightweight and portable. • Indicate the instantaneous corrosion rate at the time of measurement	• Require electrical connection to the rebar • Not applicable to epoxy-coated bars. • Cover depth has to be less than 100 mm. • Testing and interpretation has to be performed by experienced personnel. • Determining the polarized area is difficult and time consuming.
iCOR	• Requires no connection to the rebar • Its wireless, fast and easy to use • It has a user-friendly app that can be upgraded	• Not applicable to epoxy-coated bars.
Ultrasonic Pulse velocity	• The equipment used are lightweight, portable and simple to use. • Provide an indication of the uniformity and quality of concrete • It can survey a large coverage and thick members • Estimate the size and nature of defects • Results are relatively easy to interpret	• Require expert analysis and careful data collection • Less effective for irregular shapes or very thin members. • Require access to both sides of the structure. • Results are influenced by moisture and the presence of reinforcement.
Impact echo	• It is quick, accurate and reliable • Only need access to one side of the structure • Detect cracks, voids, delamination and debonding • Determine thickness	• Reliability decreases with an increase in thickness, and accuracy depends on the impact duration • Less reliable on elements with overlays • Difficult interpretation of results
Acoustic emission	• A cost-effective and sensitive method that can detect and locate active defects such as cracks • Require no lane closure	• Passive defects cannot be effectively detected. • Provide qualitative results

GPR	• It is fast at testing • Can assess bridge decks with or without overlays • Inspection results are reproducible. • Provide good quantity estimates and general locations of defects. • Minimal traffic control • Commercial equipment and application softwares are well-developed.	• It does not give information on corrosion rate. • It is sensitive to moisture, metal objects and electrical conductivity • Less effective as concrete thickness increases • Require experts or trained operators for inspection and interpretation of results.
Covermeter	• It is fast, simple and portable	• Cannot detect reinforcement corrosion
Infrared thermography	• Does not require access to the inspected element. • Can assess a large coverage in a short time period • Portable, simple with easy interpretations • Minimal traffic interference	• It is sensitive to contaminants • The equipments are expensive • It is unable to detect defects at greater depths • Its ineffective on decks with overlays • Need adequate solar radiation to obtain the required temperature differentials • Need experts for interpretation of results.
Anode-ladder system	• Warn bridge owners to take proactive measures prior to damage depiction • Applicable in inaccessible areas • They have a long design life	• Not suitable for use in submerged elements • Do not provide absolute chloride content • Cannot be installed where poker vibrator are used for compaction • Need to be installed near the concrete surface
Expansion ring system	• Warn bridge owners to take proactive measures prior to damage depiction • Easy and fast to install • Applicable in inaccessible areas	• Not suitable for use in submerged elements • Do not provide absolute chloride content
CorroWatch and CorroRisk	• Applicable in inaccessible areas	• All anodes need to be connected to the monitoring box
Multi-ring electrode	• Its used in both new and existing structures • Easy and fast to install	• The concrete-specific calibrated curves need to be calibrated always • Considers the properties of the anchoring mortar used in installation, in the interpretation of the results
Potentiometric sensors	• Easy and fast to install • They are very sensitivity, stable and robust • Requires low cost for preparation	• -

A potential combination of different NDT methods for condition assessment has been reported in the literature. Dabous [89,122] integrated the IRT and GPR to help improve the identification and quantification of defects on bridge decks. Both IRT and GPR detect delamination early and on greater coverage. IRT can be reliable and used on elements with shallow cover, while GPR can provide information on elements with deeper cover. While IRT has higher reliability and is more accurate for delamination detection, GPR detects corrosive environment that extends beyond the limit of existing delamination [127]. This is crucial when implementing GPR for maintenance and repair decisions, as the data obtained can be related to the potential deterioration of the bridge deck. The integrated results of IRT and GPR was thus used to provide condition ratings of bridge decks and compared to traditional inspection practices (visual inspection and hammer sounding). These methods were found to be more efficient and timeous than visual inspection and hammer sounding.

Apart from the technical aspects, modern technologies are found to be timeous and fast during field inspections. This reduces the time spent on inspections and saves resources. Gucunski et al. [128] spent about 6 hours of survey time in a day using five technologies, of which UPV was the slowest, followed by IE (which was 50% faster), HCP and ER were two times faster, and GPR was about four times faster. Janku et al. [129] indicated that between GPR, IRT and UPV, IRT is advantageous as it could quickly scan the entire bridge without direct access to the structure. Other practical aspects (e.g., speed of testing, ease of application, lane closure requirements, etc.) of the reviewed monitoring methods are provided in *Table 7-5* of Appendix C.

It has been noted that using a single method for condition assessment is insufficient in detecting all defects, as no particular method is complete independently. Rathod & Gupta [130] showed that combining multiple methods to evaluate defects had significantly reduced errors and provided more accurate results than standalone tests. For example, using GPR, IRT, ER, UPV, and HCP individually provide percentage errors of 43, 51, 48, 46, and 39 in detecting defects, respectively. However, the percentage error dropped to 29 when all methods were used simultaneously. The use of multiple technologies for condition assessments was also found to accurately and objectively detect and quantify deterioration progression [131]. Gucunski [131] proved this by evaluating the corrosive environment and corrosion processes using HCP and ER, the concrete degradation using GPR and USW, and deck delamination using IE.

ER and HCP are fundamental methods for determining the likelihood of corrosion; these methods correlate well with other methods. A good correlation was found between ER, HCP and GPR [96,125]. According to Gucunski [131], the agreement between sound and deteriorated areas was 71.4% and 75.1% for ER/GPR and HCP/GPR, respectively. The areas with the lowest resistivity matched the zones with the highest attenuation on the GPR maps, and the highest GPR signal attenuation which indicates corrosion activities, agrees with areas of more negative potentials. This trend can be expected as the electrical properties of concrete influence both methods. For instance, ER and GPR are both affected by concrete conductivity.

Other studies on integrated monitoring methods are illustrated in *Table 5-2*. As illustrated, it can be observed that the use of multiple NDTs on existing structures for defect detection and condition assessment has evolved in the past decades. Most of these integrated methods assessed a specific part of the bridge, the bridge deck, which is considered the most important structural element that can easily deteriorate.

Table 5-2: Summary of studies on integrated methods used for condition assessments

Integrated methods	Condition of structure	Objective of the study	Refe-rence	Year
NDT corrosion monitoring methods				
GPR and IRT	Existing structures	Detect anomalies	[97]	2019
GPR, IRT, and UPV	Existing structures	Comparative study	[129]	2019
GPR, IRT, UPV, HCP, and ER	Laboratory specimen	Defect detection	[130]	2019
GPR and IRT	Existing bridge decks	Comparative study	[127]	2018
GPR and IRT	Existing bridge decks	Comparative study	[132]	2018
GPR, IE, UPW, HCP, and ER	Existing bridge decks	Condition assessment	[131]	2017
ER, GPR, UPW, IE (RABIT system)	Existing bridge decks	Condition assessment	[133]	2015
ER, GPR, and UPW	Laboratory and existing structures	Measurement variability	[134]	2012
VI, HCP, GPR, IE, and CD	Existing bridge deck	Early defect detection	[135]	2011
GPR, IE, UPW, HCP, and ER	Existing bridge decks	Condition assessment	[128]	2010
GPR and IRT	Existing bridge decks	Defect detection	[126]	2009
GPR, IRT, and IE	Laboratory specimens	Defect detection	[136]	2007
Remote corrosion monitoring methods				
Ag/AgCl ISE, ALS, MRE and temperature sensor	Laboratory specimens	Chloride, moisture and temperature monitoring	[124]	2020
Ag/AgCl ISE and Ir/IrO$_2$ sensors	Laboratory specimens	Monitor pH and chloride concentration	[137]	2019
CW and ERE-20 electrode	New structure	Monitor corrosion	[113]	2017
ALS, ERS, CW and MRE	Laboratory specimens	Monitor corrosion	[118]	2002
ALS and ERS	Laboratory specimens	Comparative study	[106]	2001

*VI – Visual Inspection | HCP – Half-cell potential | CD – Chain Drag
GPR – Ground Penetrating Radar | ER – Electrical Resistivity | ALS – Anode Ladder System
IRT – Infrared Thermography | IE – Impact Echo | ERS – Expansion Ring System
UPV/E – Ultrasonic Pulse Velocity/Echo | UPW – Ultrasonic Pulse Waves | CW – CorroWatch
RABIT – Robotics Assisted Bridge Inspection Tool | Ir/IrO$_2$ – Iridium/ Iridium Oxide Electrode (pH sensor)
Ag/AgCl ISE – Silver/ Silver chloride Ion-Selective Electrode (Chloride sensor) | MRE – Multi-Ring Electrode

Apart from NDT methods, integrated sensor systems are also developed to monitor the corrosion of reinforcing steel. For example, Duffo and Farina [63] developed an integrated sensor system that can monitor parameters including HCP measurements, corrosion rate, concrete resistivity, oxygen availability, chloride ion concentration and temperature within the structure. The integrated system encompasses different electrodes embedded as one unit in concrete, connected to a software that attains and analyse data. This system can be embedded in new and existing RC structures with a wireless system that generates alarm warnings of damages within the concrete. The system showed capability to be used in both laboratory and field installations, with promising results and expected trends [63]. Other sensor systems that have been integrated are shown in *Table 5-2*. These integrated monitoring methods have been mostly based on laboratory settings, and few have limited use in real-life structures.

5.3 Integrated monitoring methods in the BMS

Worldwide, BMSs use numerical condition ratings to assess the condition and model deterioration of bridge elements and determine their needs for repair and maintenance. Currently, condition ratings for RC bridges are commonly based on visual inspections. However, because visual inspection is associated with various limitations, attempts have been made to complement it with other monitoring methods to assess the condition of bridge elements and assign appropriate condition ratings. For example, researchers have developed ways of utilising quantitative and qualitative data from NDT methods for condition assessments in BMSs. These methods are believed to inform the BMS on the corrosion initiation process, extent and degree of deterioration of bridges [138]. They also provide a better understanding of damage and deterioration that is not yet visible. However, condition ratings have to be redefined for these methods to work in the BMSs, as according to Hearn [139], data from NDT methods are often too detailed to be used directly in the BMS. NDT data need to be interpreted first to yield condition states or ratings that are currently being used in existing BMSs. Literature on this approach is very limited, and only a few studies have elaborated on the integration of NDT methods with visual inspections (see *Table 5-3*).

Table 5-3: Various studies on the integration of NDTs in the BMS

Studies	Methodology	Monitoring techniques	Limitations
Hearn and Shim [138]	They redefined NDT data to Integrated Condition States (ICS), where bridge elements are rated on a scale of five based on the service life stages. The attributes of service life stages include exposure, vulnerability, attack, and damage. The NDT methods determine condition states for bridge elements by measuring the five attributes. Subsequently, a binary yes/no assessment based on one threshold limit for the stages was defined.	Visual inspection, Sonic, Ultrasound, HCP, LPR, IRT, GPR, ER, UPV, covermeter, concrete permeability, and chloride content	• Each NDT was limited to two condition states (e.g., condition state 3 and 4, whether the bridge element is vulnerable or attacked) and one deterioration process (e.g., corrosion activity). • The method provides the severity of damage but not the extent, which could be because NDT methods only determine the condition and not quantity.
Dihn [39]	Developed a condition assessment system for concrete bridge decks based on an appropriate NDT technology. He integrated visual inspection with GPR technology by developing a k-means clustering** model to determine the GPR thresholds based on the deterioration level. The deterioration levels were defined as sound, moderate and severe corrosion. The method to obtain condition rating (the bridge deck corrosiveness index) for bridge decks based on the GPR output was developed.	Visual inspection and GPR	• The study is based on bridge decks only. • Parameters such as weather or moisture are not taken into consideration for the calibration of thresholds. • The true baseline data was not used, hence the analysis method based on correlation of A-scans has not been fully developed.
Pailes [140]	Use multiple NDTs for condition assessment to identify deterioration states and provide a complete condition assessment. The NDT results were quantitatively compared to determine the relationship between these methods. This comparison helps establish the threshold identification for each method (ER, HCP or GPR), which was based on the statistic approach. The results of each method was then broken down into descriptions regarding the damage state of the bridge deck, with condition states defined as severe delamination, delamination/lateral cracking, active corrosion, corrosive environment, and sound deck, with rating of 4 to 0, respectively. The data from the multiple NDTs were fused (combined) to provide condition ratings to bridge decks.	ER, HCP, GPR, IE and CD	• The study is based on bridge decks only. • Fusing data from multiple NDT requires the results from each method to be normalised, so as to obtain the same scale. • Thresholds were only identified for methods with continuous scale such as ER, HCP, and GPR, and not for IE and CD. • Coring was required to verify the accuracy of the developed thresholds for ER, HCP, and GPR. • The condition ratings did not provide the accurate representation of the deck condition, as it did not include the IE and GPR data.
Alsharqawi et al. [136, 137]	Developed a Quality Function Deployment (QFD) model based on visual inspection and GPR evaluation to provide consistent condition ratings. The visual inspection and GPR data are analysed and quantified surface and subsurface defects are fed into the QFD model. The method considers dependence between defects and allows flexible weighting. K-means clustering**	Visual inspection and GPR	• The study is based on bridge decks only. • The QFD model does not assess the structural importance factors of the different defects.

	technique was then used to determine threshold values for GPR condition categories (Good, Fair, Poor and Critical). The final output of the model is ratings represented by an integrated condition index.		
Akgul [9]	Developed an integrated bridge rating based on combined visual inspection and NDT ratings. Established rating scales for each measured NDT parameters using defined threshold limits from literature. The summation of the worst NDT rating with respective weight factors for each parameter on the bridge elements define the integrated condition rating for the bridge. Defined weight factors for the proposed integrated rating were obtained using available literature on NDT frequency of use and Analytical hierarchy process (AHP) approach.	Visual inspection, penetration resistance, bar locator, UPV, and LPR.	• The AHP cannot correlate different defects • The percentage frequency of use of NDT used to determine weight factors is based on methods used by state agencies in USA only • The NDT-based reinforcement corrosion rating was only based on corrosion potential and concrete resistivity measurements, which seems to consider corrosion risk only, as concrete resistivity does not measure corrosion.

*NDT – Non-Destructive Testing
ER – Electrical Resistivity
CD – Chain Drag
IRT – Ultrasonic Pulse Velocity
GPR – Ground Penetrating Radar
HCP – Half-cell potential
IE – Impact Echo
LPR – Linear Polarisation Resistance

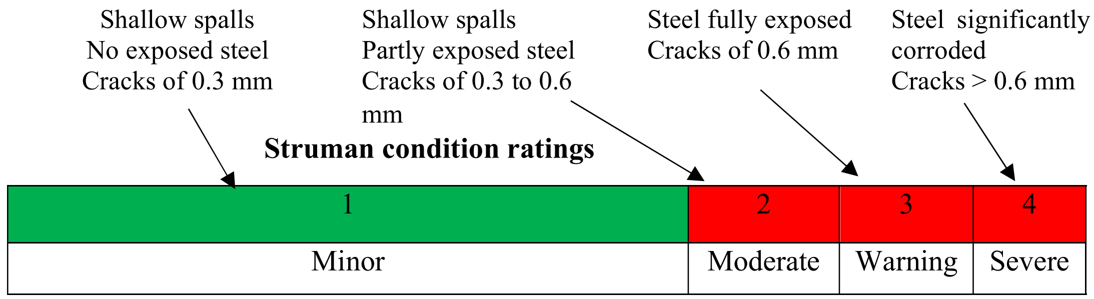
**k-clustering is a method for organising the data and summarising it through cluster prototypes. It solves the subjective determination of thresholds values.

Hearn and Shim [134,138] developed the first attempt of integrating NDT data into the BMS. Their method involves the use of NDT data in BMSs by using the Integrated Condition States (ICS) strategy based on the physical properties of bridge elements and materials. This strategy defines stages in the service life of bridge elements into four attributes: exposure, vulnerability, attack, and damage, which produce five condition states (see *Table 5-4*). The stages are based on parameters such as the presence of aggressive agents (e.g., chlorides), deterioration process activities (e.g., rebar corrosion) and the damage existing on the bridge elements (e.g., cracks, spalls or loss of steel section). The five condition states have a defined rating scale of 1 to 5, with NDT methods used to detect and measure the mentioned attributes. With each NDT method addressing a single attribute, the data is reduced to binary assessment (either the results are within or outside the threshold limit) for easier direct use in BMSs. The NDT tests results are compared to the threshold of such test to determine true conditions for bridge elements.

Table 5-4: NDT Integrated Condition States on a scale of 1 to 5

Rating	States	Description
1	Protected	Protected against agents that can cause deterioration
2	Exposed	No protection. Aggressive agents have not yet reached their threshold to initiate deterioration.
3	Vulnerable	No deterioration process is active, though aggressive agents are present and may activate the deterioration process at any time.
4	Attached	A deterioration process is active with small spalls and delamination
5	Damaged	Measurable and visible damage indicators (loss of steel section, large spalls and extensive cracking)

In comparison to the Struman BMS condition rating, Hearn's rating assesses the early deterioration of RC bridges as it considers the service life stages. For instance, in new bridges, the NDTs are used to evaluate the protection system and ingress of chloride ions; bridges with medium ages are evaluated for active corrosion, and old bridges are evaluated for damage indicators such as cracks, delamination, spalls, and loss of steel section. The Struman BMS condition rating relies on spalls and cracks to define the condition ratings, and its these damage indicators that are attributed to reinforcement corrosion (including steel exposure). Compared to Hearn's method (see *Figure 5-1*), the Struman rating 1, which is considered in good condition, addresses the first four stages of service life and ratings 2 to 4 address the damaged stage only. This rating relies primarily on visual inspection, reporting on damage but not on early deterioration (vulnerability and exposure) which is typically not visible on the concrete surface. This is critical in structures exposed to chloride environments.



Hearn and Shim's integrated condition states

Condition states	Protected	Exposed	Vulnerable	Attacked	Damaged
Rating	1	2	3	4	5
Thresholds	$X_{pro} < T_{pro}$	$X_{pro} \geq T_{pro}$	$X_{vul} \geq T_{vul}$	$X_{att} \geq T_{att}$	$X_{dam} \geq T_{dam}$
	-	$X_{vul} < T_{vul}$	$X_{att} < T_{att}$	$X_{dam} < T_{dam}$	-

Where: X – the test data
T – threshold value
Four tests: $Test_{pro}$, $Test_{vul}$, $Test_{att}$, $Test_{dam}$ (protected, vulnerable, attacked and damaged, respectively) for four transitions among the five condition states

Figure 5-1: Comparison of Struman and Hearn's developed condition rating

Furthermore, Hearn's proposal does not establish the rating scales and quantitative threshold limits for the measured parameters within the service life stages of bridge elements. Rather, the condition ratings are allocated using one threshold limit based on which the NDT used for each attribute is given a binary Yes/No assessment between two condition states (see *Figure 5-1*). This seems inadequate as some measured parameters, e.g., electrical concrete resistance or corrosion rate, have standard ranges of threshold values and not just a single threshold value. The quantitative threshold definition is essential for each condition state to assign condition ratings to bridge elements based on the level and extent of deterioration.

Since Hearn initiated the idea of using NDTs in bridge condition assessments, attempts have been made in the past decade to improve the methodology. As observed from *Table 5-3*, some studies incorporated only one NDT method while others incorporated multiple NDT methods. Dinh [39] and Alsharqawi et al. [142] incorporated visual inspection with GPR to assess the condition of bridge decks. The combined application of the two methods improves the identification and quantification of defects on bridge decks. Regarding reinforcement corrosion, the two methods can detect surface and sub-surface defects but cannot detect when corrosion has begun nor the risk to corrosion.

Pailes [140] later developed a multi-modal NDT integration into the BMS, where he combined ER, HCP, GPR and CD to obtain a complete condition assessment of bridge decks. The integration was done because some methods detect defects that others cannot. Similar to Hearn’s method, Pailes defined five condition states based on the deterioration process, with associated ratings. Each of these condition states has a specific diagnosis method and defined threshold. However, distinctly from other authors, Pailes fused the NDT results instead of presenting them separately. The study indicated that multi-modal NDT (multiple NDT) provides a complete and in-depth condition assessment, with condition ratings indicating more damage identified on the structures than when using only visual inspection.

Recently, Akgul [9] improved Hearn’s research by developing an integrated rating using three NDTs: polarisation resistance, HCP, ER, penetration resistance (compressive strength), covermeter and pulse velocity to supplement visual inspection. He aimed to use these rapid inspection techniques to incorporate quantitative bridge inspection results and rank bridges for prioritisation in the BMS. This was practically applied on a network of 100 deteriorated RC bridges with multiple bridge elements such as decks, girders, cap beams, and abutments. This part of his research was unique as the testing applied to multiple bridge elements and not just to decks, like other studies. In his study, four rating scales with associated threshold limits were defined for: visual inspection, polarisation resistance, penetration resistance and pulse velocity. These measured NDT parameters obtained rating values of 2, 6, 3, and 2, respectively. An integrated rating of 3.15 was obtained for the network of 100 bridges, which corresponds to medium to severe level of deterioration.

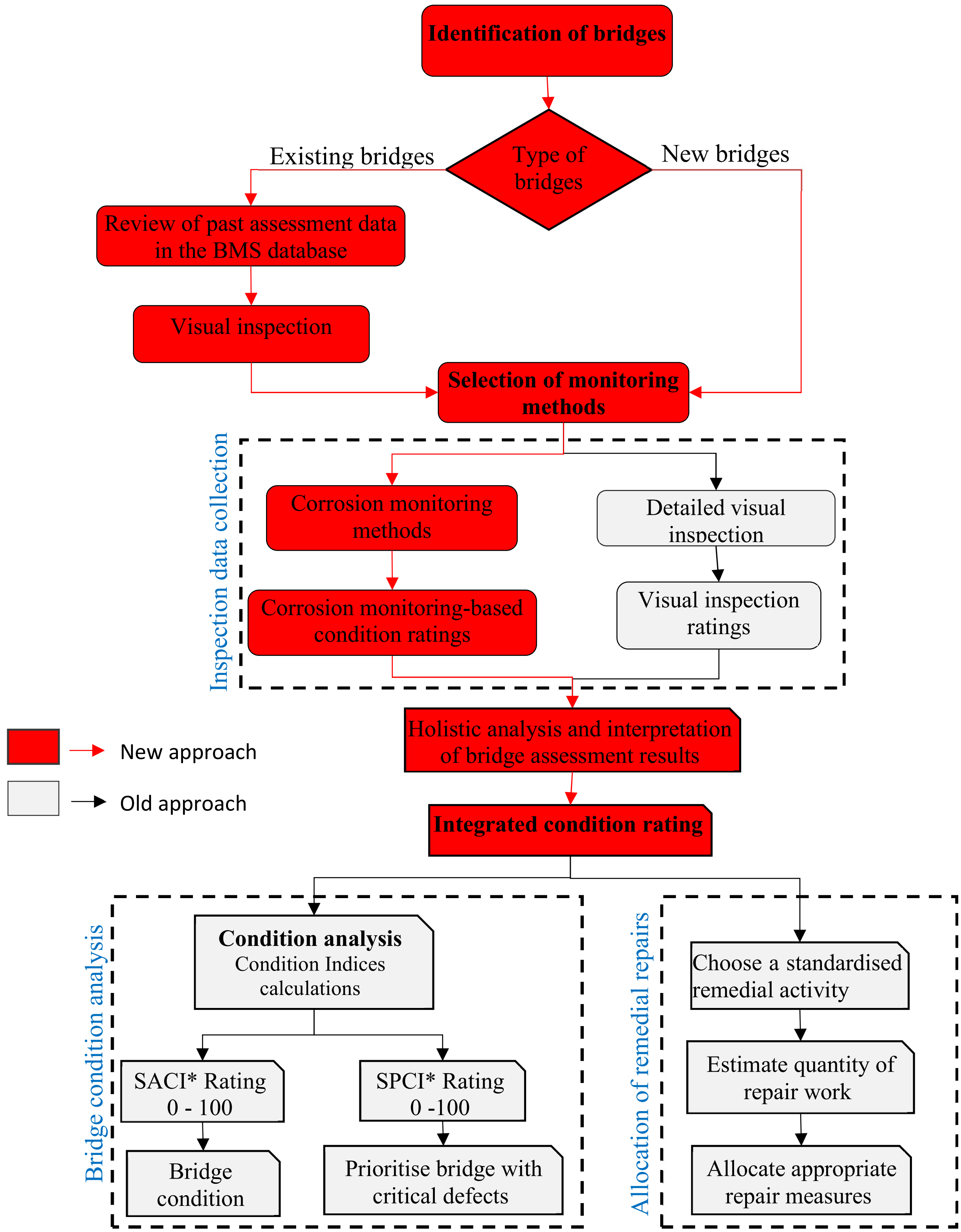
Approaches discussed previously of incorporating NDT data into the BMS have been attempted in the USA, for the Pontis BMS. In South Africa, no effort has been made in this regard. Hence, there is a need to develop monitoring-based ratings that can be directly integrated into the Struman BMS. A proposed methodology is discussed in the following section.

5.4 The proposed methodology

As discussed in Section 3.3.3, the Struman BMS is a complex database comprising inventory, inspection, budget and maintenance modules. This study focuses only on the inspection and condition component of the Struman BMS. Generally, the two components involve collecting inspection data through condition assessments and subsequent use of the data for allocating and prioritising bridges for maintenance and repair. Specifically, the condition of RC bridges is assessed on a network level by visual inspection, using the *TMH19* manual. Visible defects are rated using the DERU (Degree Extent Relevancy and Urgency) rating system on a scale of 1 to 4. These defect ratings are then used to obtain the bridge condition and allocate repair and maintenance measures.

The visual inspection method currently used in BMSs is insufficient as it cannot detect early damage related to reinforcement corrosion. Literature has shown that integrating visual inspection with multiple monitoring methods compensates for this limitation, improves accuracy, and provides a complete condition assessment. In addition, these methods provide quantitative information based on measured physical quantities of deterioration. It is thus desired to integrate these monitoring methods with visual inspections to allow early steel corrosion detection, hence sound maintenance and damage prevention.

Figure 5-2 shows the proposed integration process of corrosion monitoring methods into the Struman BMS. The proposed methodology follows the same approach that Struman BMS uses, except that instead of using visual inspection data alone, data from corrosion monitoring methods are incorporated for condition rating and analysis. The process begins by identifying bridges that need to be assessed (new or existing bridges), choosing monitoring methods specific for the identified bridge and intended purpose of inspection, and then collecting data. Selecting monitoring methods for new bridges is based on the needs and the type of data that need to be collected from bridges, and for existing bridges, it depends on the past assessment data and visual inspection. The obtained data from visual inspection and corrosion monitoring methods are then holistically analysed and interpreted to obtain integrated condition ratings, which are used for bridge condition analysis and allocation of remedial repairs.



*SACI – Structural Average Condition Index
SPCI – Structural Priority Condition Index

Figure 5-2: The proposed integration process of monitoring systems into the Struman BMS

5.4.1 Identification of bridges

In South Africa, all bridges on the national roads network are managed by the South African National Road Agency Limited (SANRAL). Bridges outside this network are managed by different agencies including, municipalities, transport authorities, and provincial governments. All these agencies implemented the Struman BMS. The first step carried out as part of the implementation process is to obtain inventory data of all bridges in their network. For inspection purposes, four criteria need to be considered in the identification of bridges:

1. The type of structure and material
2. Exposure environment
3. Age of the structure
4. Existing condition

The Struman BMS considers various structures for condition assessment. However, this approach applies to structures classified as bridges in the inventory list and constructed with reinforced concrete since RC bridges are common in South Africa. The exposure environment should be considered in the selection process as the proposed approach is related to bridges exposed to airborne chlorides in the marine environment. According to BS EN 206 [143], bridges in this region would be classified as XS1 (see *Table 2-1*). However, it should be noted that the proposed approach in *Figure 5-2* may not differ if the bridges are located in the inland environment.

Furthermore, it is critical to know the age of the bridges being inspected so as to know the type of monitoring method to use. This would help discern between new and existing bridges within the network as some monitoring methods such as expansion ring systems do not apply to new structures. The existing condition criterion is only applicable to existing bridges, with past assessment data available in the BMS database. If no past data is available, it is recommended that visual assessment be carried out on the specific bridges to know the up-to-date condition. The existing condition criterion is essential as it will save time determining locations for installing monitoring systems in existing bridges.

5.4.2 Selection of corrosion monitoring methods

Corrosion monitoring methods differ from one another in terms of principles and application. Some differ in principles but can be used for the same application. Each method has its advantages and limitations; however, different monitoring methods can be integrated to eliminate the limitations of each other (e.g. see *Table 5-2* and *Table 5-3*). Because some monitoring methods reviewed in the present study can be used for the same purpose or detect the same defects, selecting a method for each application is vital to save time and resources. The guideline process outlined in *Figure 5-3* can be followed when selecting a method.

Though most selection processes are typically based on cost-benefit analysis, the proposed method is based only on defined technical criteria. As shown in *Figure 5-3*, with RC bridges already identified for corrosion monitoring, the consideration is given to whether the bridges are new or existing to know the type of monitoring method to choose. Selection of monitoring methods for new bridges is based on the needs of the bridges to be assessed in terms of safety, serviceability and structural integrity. It is also based on the type of data that needs to be collected on a particular bridge, e.g. to know the time to corrosion initiation, the anode-ladder system can be chosen as a corrosion monitoring method. On the other hand, selection of monitoring methods in the existing bridges is based on the existing condition of the bridge, which could be obtained from previous assessment data or an up-to-date visual inspection.

Furthermore, input details on different corrosion monitoring methods are obtained from experts in bridge inspections for both new and existing bridges. In addition, technical criteria are defined for different monitoring methods based on the inspection requirements and the parameters involved in the deterioration process. *Table 5-5* shows some general criteria to be considered for corrosion monitoring methods. These criteria are open to improvement and can be adapted upon practical use of the monitoring methods. An evaluation of each reviewed corrosion monitoring method using the above-mentioned criteria is illustrated in *Table 7-5* of Appendix C. Based on the comparison of the characteristics of each monitoring method with the criteria defined, and the needs or existing condition of bridges, the appropriate methods can be selected for a particular bridge.

Visual inspection by default is included in the to-be-selected monitoring methods, as it is part of the old approach in the Struman BMS. In addition, it is recommended that chloride content measurements be taken both periodically and continuously for existing structures, as this research is focused on chloride-induced reinforcement corrosion. This method is used in most condition assessments of marine structures and should be considered for monitoring.

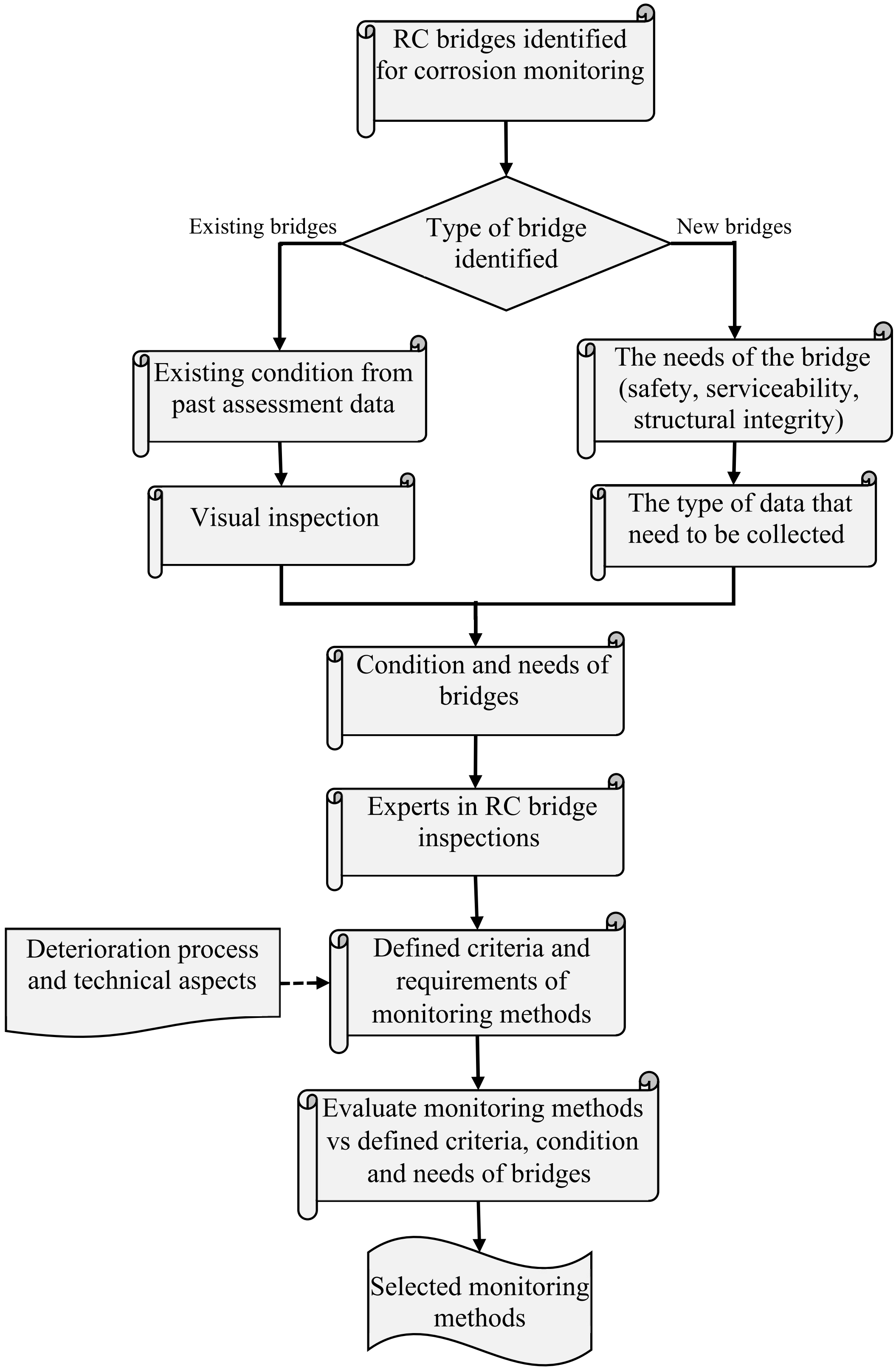


Figure 5-3: Selection guideline for monitoring methods

Table 5-5: General criteria for corrosion monitoring methods

General criteria for corrosion monitoring methods	
1. Nature of method (where on the bridge it's applicable)	2. Requirement of the lane closure
3. Types of defects detected	4. Maintenance requirement
5. Detection depth	6. Surface preparation during testing
7. Installation/setting up speed	8. Accuracy
9. Testing speed	10. Equipment cost
11. Results interpretation	12. Access required to the assessed element

5.4.3 Inspection data collection

Generally, regular inspections are needed to assess the condition of bridges. This is currently done approximately every 3 to 5 years, where inspection data is collected periodically through visual inspection and using the DERU rating system. The DERU rating system rates defects on a scale of 1 to 4 and only bridge elements with defects are rated. The assessment output is the overall bridge rating, indicated by the rating of the worst defect on any bridge element. This existing approach should still be used for visual inspection of visible defects in the proposed methodology.

The new approach incorporates selected monitoring methods for bridge condition assessment of invisible defects. Monitoring of RC bridges in this new approach is divided into two parts; periodic and permanent monitoring. The periodic monitoring includes NDT measurements of invisible defects using any of the selected NDT methods previously discussed (HCP, ER, LPR, UPV, IE, AE, GPR, cover measurements or IRT). It is also recommended that chloride content measurements be carried out as part of periodic monitoring. These periodic measurements must be taken at the same time when visual inspections are done to save time and resources. The permanent monitoring part includes installing sensor systems in new bridges during construction (using ALS, MRE or CW sensors) and existing bridges (ERS, MRE, CR or ISE sensor). The following aspects should be considered for successful data collection in the mentioned monitoring systems.

5.4.3.1 Monitoring system design

The system design for monitoring methods varies depending on their application and the type of monitoring under consideration. For data collection on existing structures, periodic monitoring methods require the selection of representative locations of measurements and the provision of required access if needed (with access equipment). The selection of measurement points depends on the priority of areas most susceptible to corrosion (e.g., bridge decks or areas with lower cover depth), areas deemed important from a structural integrity point of view or areas with existing moderate-poor conditions from previous inspections.

As mentioned previously, this research focuses on chloride-induced reinforcement corrosion, parameters such as chloride and cover depth measurements should be included in the periodic monitoring. Periodic chloride measurements are to be carried out on core samples taken from the existing bridges only. The locations for core samples need to be representative and not intrusive, e.g. if there is a reason to assume that a bridge abutment and deck are made from similar concrete, then the core samples can be taken from the abutment than the deck so that they do not cause any damage. Also, the abutment does not have much structural significance compared to the deck and it is easy to access. Core samples also need to be taken from the side of the bridges facing the ocean, as air-borne chlorides tend to concentrate more on these sides. Cover depth measurements should be taken on the critical structural elements such as slabs, soffits, beams, abutments, intermediate supports (e.g., pillars and piles) etc. The cover depth measurements are critical, easy to be carried out and only taken once, as they do not change. This informs the condition of the structure throughout its lifetime. In South Africa, for newer bridges, it might be the case that cover depth data is available from the quality assurance process after construction, as different agencies have now known its importance to structures. In existing bridges, cover depth data might still need to be established.

The design of periodic monitoring systems consists of components shown in *Figure 5-4*. Data must be collected in the TMH 19 inspection sheet, which would be compiled into the BMS database. If more than one NDT method is used for assessment, their data should be fused (combined) to obtain one deterioration map. For example, data from HCP, ER, GPR, IE and IRT can be fused to one deterioration map. According to Ahmed [58], fusing deterioration maps helps bridge engineers and inspectors better interpret the inspection results and accurately identify defective areas, as some methods can present clarity and visibility of the defective areas while others cannot.

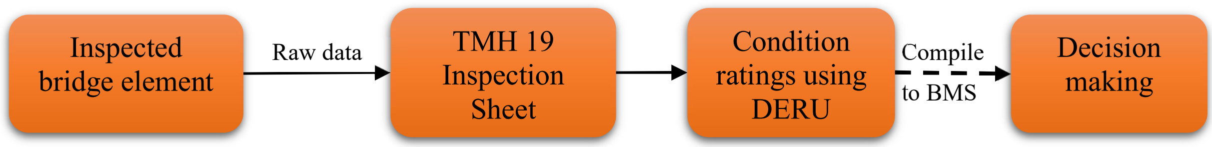


Figure 5-4: Schematic system design for periodic monitoring systems

On the other hand, the design of permanent monitoring systems follows the fundamental setup with components including sensor and instrumentation, data acquisition and analysis, condition ratings, and decision making (see *Figure 5-5*). This strategy is the same for both new and existing structures. The sensors capture raw data from the material (e.g., concrete or reinforcing steel) regarding its condition or physical measurements associated with environmental conditions or loads. The functionality of any monitoring system depends on the types and number of sensors used. The sensors in this approach need to be used in multiples (i.e., more than one sensor) for the whole bridge coverage. However, the number of sensors to be used on any bridge depends on various factors such as bridge length, critical points, vulnerability to exposure and installation spacing.

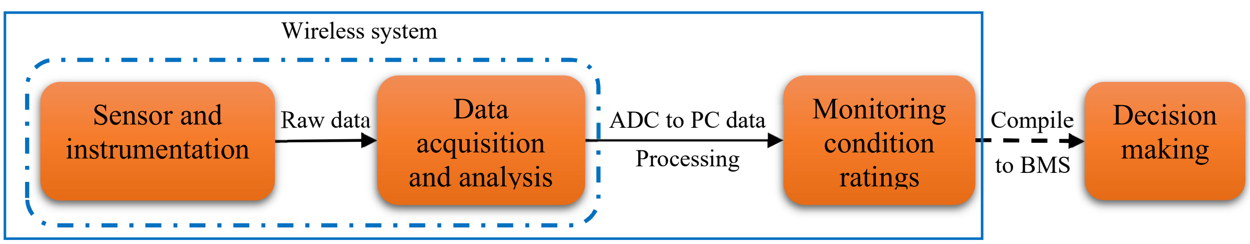


Figure 5-5: A system design approach for permanent monitoring system

The data acquisition and analysis component of the monitoring system allows real-time data collection, interpretation, analysis and storage of raw data from the sensors to assess the condition of bridges. Generally, for this to be possible, the sensor's electrical output is digitised by an analogue-to-digital converter (ADC) and connected to the central computer for further processing. The connection of the data acquisition sources is usually wired with cables, where all sensors are wired to data acquisition hubs mounted on bridges and the data collected wirelessly through an

Ethernet connection. This has been an approach that has been used traditionally and is currently still in use.

The cabled system is only advantageous when used in new structures due to ease of application. However, installing various hardware (e.g., power and data cables, conduit) on a network basis in in-service bridges is still a challenge and results in more labour and costs spent on the installation process. In addition, wired system costs increase with the increase in bridge length due to conduit and power connection [144]. This makes wireless systems relatively cheaper than wired systems; hence, it has become a practical approach to solving the aforementioned limitations. The wireless system also allows in-network data communication and processing, which is advantageous over wired systems. Furthermore, if used in South Africa, the wireless system would circumvent the theft of conduits and cables, which is currently critical for wired systems. Therefore, a wireless system can be used for this approach.

5.4.3.2 Monitoring system installation

Permanent monitoring systems need to be installed in both new and existing bridges. In new bridges, the chosen monitoring systems can be installed during construction, which should be placed per the specifications before concrete placement. The chosen monitoring systems can be installed in existing bridges by drilling into the concrete at selected locations. Critical points for installation should be determined based on:

- Locations on structural elements that are susceptible to reinforcement corrosion from chloride exposures, e.g., bridge decks, edge beams, the upper part of columns etc
- Structural elements that, if damaged, may compromise the safety of the whole bridge
- Areas that can be accessible for periodic maintenances

The number of positions for installing chosen monitoring systems depends on the number, size and condition of the considered bridge element. Longer and wider bridge elements may require more monitoring than short and narrow elements. All corrosion parameters have to be considered for measurement at each chosen location. For example, rebar potential, corrosion current, the time to corrosion onset, corrosion rate, moisture content, and temperature should be measured at one location. Three sensors are proposed in one location to consider variations of the measured parameters with the depth into the concrete. These sensors can be installed at different depths based on the actual concrete cover (e.g., 20, 40 and 60 mm from the exposed surface for a concrete cover of 60 mm). A single temperature sensor is proposed to be installed at each location. It should be noted that all sensors at one location should be connected to one data acquisition hub with a modem for transmission, which connects to the controlled PC. The installed systems should be marked on the as-built drawings or the bridge elements for ease of identification.

For critical and difficult locations to access during maintenance, cable connections with good shielding should be used, running from the sensors to easily accessible locations. A box should be placed at the end of the cables for protection, installed together with the data acquisition unit. The cables connection should be marked on the as-built drawings for identification.

5.4.4 Integrated condition rating

In the Struman BMS, the performance of bridges is determined using the DERU (Degree, Extent, Relevancy, and Urgency) rating system (see *Table 5-6*). As discussed in Section 3.3.3.2, the ratings are assigned by bridge inspectors to different bridge elements based on visible defects. To integrate corrosion monitoring systems into the Struman BMS, monitoring-based condition ratings based on the performance of material properties are proposed. The monitoring-based condition ratings are assigned to bridge elements based on the data collected on specific elements. These condition ratings provide the severity (Degree) of the measured corrosion parameters. The extent (E) and Relevancy (R) rating of the measured corrosion parameter will still be based on the 1 to 4 rating illustrated in *Table 5-6*. Combining the DER rating from visual inspections and monitoring-based parameters provide the Integrated Condition Ratings (ICR) of bridge elements.

Table 5-6: Details of the DERU rating system for the Struman BMS

Rating	Degree (D)	Extent (E)	Relevancy (R)	Urgency (U)
X	Not applicable			Make safe
U	Unable to			Record only
0	None			Monitor only
1	Minor	Local	Minimum	Routine
2	Fair	More than local	Moderate	< 10 years
3	Poor	Less than	Major	< 5 years
4	Severe	General	Critical	ASAP*

*ASAP – As Soon As Possible

The monitoring-based condition ratings are based on the corrosion parameters measured and assessed by different monitoring methods. These parameters include corrosion potential, electrical resistance, corrosion rate, chloride content, moisture content, presence of cracks and delamination (see *Table 7-4* in Appendix B). In order to establish condition ratings for the different parameters measured, their threshold limits need to be defined. These thresholds are based on the measurements obtained on the deteriorating bridge elements or referenced from the literature. This will ensure that the physical condition of the bridge element is considered. It should be noted that only thresholds of the parameters measured by monitoring methods reviewed in this study will be discussed.

Corrosion risk is one of the factors considered for monitoring-based condition ratings. The risk of reinforcement corrosion occurring in RC bridges can be determined using corrosion potential. HCP measurement is an accepted and improved method for condition assessment. The measurements are quite easy to do and do not need equipment. Though, electrical resistivity measurements are needed to interpret the results of this parameter. The thresholds for the two parameters (corrosion potential and electrical resistivity) have been defined in the literature, with defined criteria being the low, moderate, high and very high-risk level of reinforcement corrosion [21,60]. In order to obtain one rating for corrosion risk, these two parameters should be combined using Akgul's approach [9].

Akgul [9] graphically combined the threshold values for these two parameters and plotted the measured corrosion potential and resistivity readings on the graph. He defined the corrosion potential threshold for low, medium and high risks as > -200, -200 to -350, and < -350 mV, respectively, and for concrete resistivity as > 50, 50 to 20, and < 20 kΩ.cm, respectively. This was done to determine corrosion risk levels, defined by the $c_{i,j}$ coefficients in *Figure 5-6*, with the corrosion risk zones (low, medium and high) based on the corrosion potential and resistivity reference values. The subscripts for corrosion risk levels ($c_{i,j}$ coefficients), '*i*' indicates the risk level for corrosion potential from 1 as the high-risk to 3 as the low-risk level, and '*j*' indicates the risk level for concrete resistivity, with 1 as the low-risk and 3 as the high-risk level. The dashed lines in the graph show the corrosion risk zones for both parameters. This same method is proposed in this study to determine the corrosion risk rating for RC bridges. However, it should be noted that factors affecting the two parameters (corrosion potential and concrete resistivity), such as type of binder type and relative humidity, to name a few, should be considered when taking measurements.

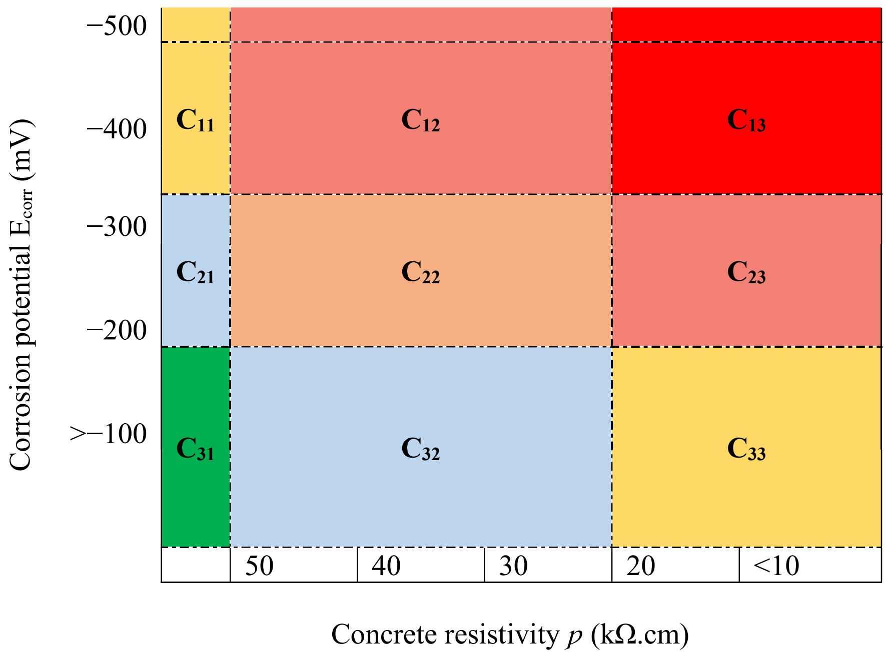


Figure 5-6: Corrosion risk zones based on integrated corrosion potential (using copper/copper sulfate electrode) and concrete resistivity (based on Akgul's approach [9])

Akgul's model only considered the corrosion potential thresholds for copper/copper sulfate electrodes, which form the graphical representations of the corrosion risk zones shown in *Figure 5-6*. In addition, this study considers the standard reference electrodes typically used; silver/silver chloride electrode. The corrosion potential thresholds for silver/silver chloride electrode (see *Table 4-1*) for low, medium and high risks is used as > −100, −100 to −250, and < −250 mV, respectively. Considering the corrosion risk levels for both the corrosion potential and concrete resistivity, the graphical representations of the corrosion risk zones is obtained as shown in *Figure 5-7*. The respective proposed ratings (with scale 1 to 6) for both cases (based on using copper/copper sulfate electrodes and silver/silver chloride electrode) are shown in *Table 5-7* and *Table 5-8*. The two reference electrodes (copper/copper sulfate and silver/silver chloride electrode) are considered in this study because they are commonly used for chloride-induced corrosion measurements.

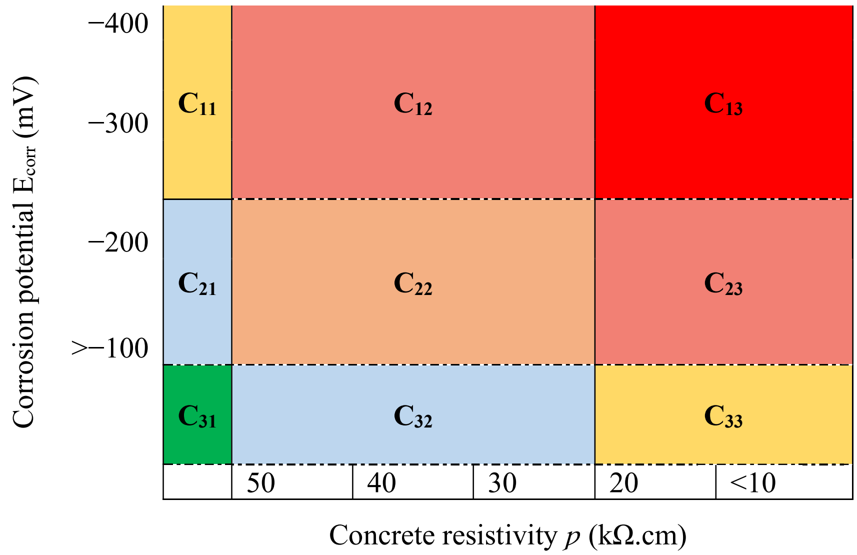


Figure 5-7: Corrosion risk zones based on integrated corrosion potential (silver/silver chloride electrode) and concrete resistivity

Table 5-7: Proposed corrosion risk rating scale for copper/copper sulfate electrode (CSE)(adapted from Akgul [9])

Corrosion potential, E_{corr} (mV) and concrete resistivity, p (kΩ.cm)	**Corrosion coefficient**	**Rating $CR_{CorrRisk}$**	**Description**
$E_{corr} < 200$, $p > 50$	C_{31}	1	Low risk
$200 < E_{corr} < 350$, $p > 50$ and $E_{corr} < 200$, $20 < p < 50$	$C_{21} = C_{32}$	2	Low to medium risk
$E_{corr} > 350$, $p > 50$ and $E_{corr} < 200$, $p < 20$	$C_{11} = C_{33}$	3	Medium to severe risk
$200 < E_{corr} < 350$, $20 < p < 50$	C_{22}	4	Severe risk
$E_{corr} > 350$, $20 < p < 50$ and $200 < E_{corr} < 350$, $p < 20$	$C_{12} = C_{23}$	5	Severe to critical risk
$E_{corr} > 350$, $p < 20$	C_{13}	6	Critically high risk

Table 5-8: Proposed corrosion risk rating scale for silver/silver chloride electrode (SCE)

Corrosion potential, E_{corr} (mV) and concrete resistivity, p (kΩ.cm)	**Corrosion coefficient**	**Rating $CR_{CorrRisk}$**	**Description**
$E_{corr} < 100$, $p > 50$	C_{31}	1	Low risk
$100 < E_{corr} < 250$, $p > 50$ and $E_{corr} < 100$, $20 < p < 50$	$C_{21} = C_{32}$	2	Low to medium risk
$E_{corr} > 250$, $p > 50$ and $E_{corr} < 100$, $p < 20$	$C_{11} = C_{33}$	3	Medium to severe risk
$100 < E_{corr} < 250$, $20 < p < 50$	C_{22}	4	Severe risk
$E_{corr} > 250$, $20 < p < 50$ and $100 < E_{corr} < 250$, $p < 20$	$C_{12} = C_{23}$	5	Severe to critical risk
$E_{corr} > 250$, $p < 20$	C_{13}	6	Critically high risk

The corrosion rate is another considered parameter that characterises corrosion propagation in existing structures. Threshold values for this parameter are quite limited in the literature. The most used criteria to define corrosion state include the corrosion current being < 0.1 µA/cm^2 when the steel is in a passive state, 0.1 to 0.5 µA/cm^2 when corrosion is low, 0.5 to 1.0 µA/cm^2 when corrosion is moderate, and > 1.0 µA/cm^2 when corrosion is high (see *Table 4-3*). These criteria in this study are defined to set as thresholds limits for corrosion state rating. *Table 5-9* provide the proposed rating for the measured corrosion rate, with a 1 to 5 rating scale.

Table 5-9: Proposed condition rating for corrosion rate (adapted from [13,21])

Corrosion current (µA/cm^2)	**Rating $CR_{I\text{-}corr}$**	**Description (reinforcement condition)**
0 – 0.1	1	Passive condition
0.1 – 0.5	2	Low to moderate corrosion
0.5 – 1.0	3	Moderate to high corrosion
1.0 – 1.5	4	High corrosion
>1.5	5	Very high corrosion

The most important parameters for assessing the risk of chloride-induced corrosion in RC are cover depth and chloride content measurements. The concrete cover depth and its quality affect the ease of chloride ions, oxygen and moisture into the concrete; hence it governs the corrosion rate of rebars embedded in concrete. Lower cover depths indicate areas prone to corrosion and chloride content at the steel level indicates the immediate corrosion risk. Chloride content measured at various depths within the concrete cover is typically presented as chloride profile versus cover depths. Analysing the chloride profile for surface concentrations and diffusion coefficients in combination with the age of the structure and cover depth allows predicting future corrosion initiation.

Both cover depth and chloride content have discrete thresholds. A minimum cover depth of 65 mm is recommended by SANS 10100-2 for concrete structures in contact with seawater. This should be considered in the proposed approach when interpreting covermeter measurements. However, the cover depth should not exceed 75 mm, because cover depths above this value has been found to result in cracking, which is detrimental to corrosion [16]. On the other hand, chloride thresholds showed much variation in a considerable amount of research, with the most widely used chloride threshold values for practical applications being 0.4 % total chloride by mass of binder. This should be considered for the interpretation of measured chloride concentrations.

Furthermore, the moisture content is also a critical parameter that governs the acceleration of corrosion rate. This parameter is indirectly related to concrete resistivity, as it indicates moisture in the concrete. Hence, the proposed rating for corrosion risk shown in *Table 5-7* should be used for the interpretation of this parameter's measurements. This is because concrete resistivity cannot be considered on its own; it just helps with interpreting HCP and corrosion rate measurements. However, it should be noted that the proposed rating is only applicable if the same concrete is used throughout the structure, as concrete resistivity is affected by many factors, including exposure condition, temperature, binder type, use of blended cement and others

Pulse velocity is another parameter that characterises the concrete quality. This method provides a qualitative indication of the risk of corrosion and can be useful when additional investigations of certain defects are required. As discussed in Section 4.2.2.2, concrete quality (with respect to concrete strength) based on ultrasonic pulse velocities has general guidelines for its prediction. The pulse velocity rating shown in *Table 5-10* is based on the mentioned guidelines. The pulse velocity of a bridge sub-element is calculated as the mean value of all pulse velocity measurements on that sub-element. This mean value will provide the condition of a particular bridge sub-element, while the most critical mean value on any bridge sub-element is selected to obtain the overall bridge element condition. This is a similar procedure currently used by Struman BMS, of which the worst defect on any bridge element represents the condition of the bridge. The scale used in Struman BMS is applicable, with a range of values for this parameter being a 1 to 4 rating scale. The range of pulse velocity values for a given rating scale is based on the threshold limits defined in the IS 13311-1 [81].

Table 5-10: Proposed condition rating for pulse velocity

Pulse velocity (km/s)	**Rating CR_{puls}**	**Concrete quality**
>4.5	1	Excellent
3.5 – 4.5	2	Good
3.0 – 3.5	3	Medium
<3.0	4	Doubtful

Finally, cracks and delamination are some of the corrosion-related parameters looked at in the Struman BMS condition assessment; however, the assessment is limited to surface or visible cracks and delamination. The condition rating is only provided for surface cracking, depending on the type of cracks detected. In this new approach, different corrosion monitoring methods can detect internal cracks and delamination (shown in *Table 7-4* of Appendix B*)* before they become severe. The proposed condition rating should be based on the total defective areas identified on the bridge elements. The defective area combines all internal delaminations, cracks, voids in the subsurface of the concrete. There is no literature regarding the thresholds of defective areas. However, the Pontis BMS assess delamination on concrete members by considering the defective areas of the assessed element area and using *Table 5-11* to provide condition ratings to assessed elements. The ratings provided in *Table 5-11* could be used to assess defective areas in the new approach, though further research and development is needed for the specified percentages.

Table 5-11: Example of condition rating for defective areas based on the Pontis BMS [93]

Rating $CR_{defective}$	**Concrete condition**	**Description**
1	Sound	No internal cracks, delamination, or voids within the subsurface
2	Fair	Combined areas of defects is 2% or less
3	Poor	Combined areas of defects is more than 2% or less than 10%
4	Severe	Combined areas of defects is more than 10% or less than 25%
5	Very severe	Combined areas of defects is more than 25%

All the condition ratings discussed above represent the degree (D) rating, which can be assigned to various bridge sub-elements during the assessment process. The data for assigning condition ratings could be collected from periodic or permanent monitoring. The extent (E) and relevancy (R) rating should be assigned using the same Struman rating on a scale of 1 to 4 (see *Table 5-6*). This will result in having various DER ratings on the sub-elements of the 21 predefined bridge elements. In order to obtain the overall rating of each bridge element, the worst DER rating of any sub-element is used considering a particular parameter, e.g., corrosion rate. Hence, the proposed Integrated Condition Rating (ICR) for the bridge element is the sum of the worst rating for all defects (visual inspection rating) and corrosion-related parameters multiplied by their weight factors, as illustrated in Equation 5.1. An illustration of the formula including proposed corrosion parameters is shown by Equation 5.2.

$$ICR_{bridge\ element} = \sum (w_i * CR_i) \qquad \text{Equation 5.1}$$

$$\begin{aligned} ICR_{bridge\ element} &= w_{VisInsp} * CR_{VisInsp} + w_{Corrorisk} * CR_{Corrorisk} \\ &+ w_{CorroRate} * CR_{CorroRate} + w_{PulseV} * CR_{PulseV} \\ &+ w_{DefectiveA} * CR_{DefectiveA} + w_{Chloride\ C} * CR_{Chloride\ C} \\ &+ w_{MoistureC} * CR_{MoistureC} \end{aligned} \qquad \text{Equation 5.2}$$

Where w_i is the weight factor of the monitored parameter i, and CR_i is the condition rating for that specific corrosion parameter. The monitored parameters include visual inspection, corrosion risk (from corrosion potential and electrical resistance), corrosion rate, pulse velocity, defective areas (from the presence of cracks, delamination and voids), chloride concentration, and moisture content.

The weight factors in Equation 5.1 signify the importance of each parameter on the evaluation of the overall bridge element condition rating. The information on the weight factors will depend on the corrosion monitoring methods chosen for a specific project or research. The author recommends that an approach by Akgul [9] be used to determine the weight factors. The method includes combining results of two techniques 1) using statistical data available in the literature on the frequency of use of monitoring methods for bridge inspection, and 2) using the analytical hierarchy process (AHP) approach to determine the expected weights. In addition, a pairwise comparison of the parameters measured by the chosen monitoring methods needs to be done based on their relative importance to the overall bridge element condition.

5.4.5 Bridge condition analysis

As discussed in Section 3.3.3.3, the condition analysis in the Struman BMS is done under the condition module. The DER rating allocated to each bridge element is used to place bridges in order of priority. In order to evaluate the condition of bridges, two Condition Indices (CI) need to be evaluated; the Structural Average Condition Index (SACI) and Structural Priority Condition

Index (SPCI). The procedures for calculating the two indices are still the same and have been outlined under Section 3.3.3.3. The only distinct difference is the consideration of DER ratings for corrosion parameters and not visual inspection only. The equations for the two indices are illustrated by Equation 3.2 through Equation 3.7, and the bridge is rated using one of the five condition categories presented in *Table 3-10*, with both SACI and SPCI ranging from 0 (worst condition) to 100 (best condition). The SACI rating indicates the bridge condition, while the SPCI rating is used to prioritise bridges with critical damage that needs urgent treatment.

5.4.6 Allocation of remedial repairs

Remedial repairs are allocated during inspection (whether by visual inspection or monitoring methods) based on the integrated condition rating of the bridge elements. The standardised remedial activities are chosen for all bridge elements that need repair or maintenance from the list provided in the TMH 19 manual. In addition, the urgency (U) rating of defects and corrosion parameters using the 1 to 4 rating scale (see *Table 5-6*) determines the need to repair a particular bridge element. Usually, inspectors are required to estimate the quantity of repair work used to determine the repair needed and cost. In this new approach, deterioration maps and quantities of defective areas should be used to provide the quantity of repair.

5.5 Summary

This chapter presented a consideration for the possible integration of corrosion monitoring methods into the Struman BMS. An overview of the previous studies that used integrated methods for condition assessments and some used in the BMS was presented. This provided information on how different monitoring methods were used for bridge inspections by various researchers, where they were used and what they were used for. In relation to previous studies, a methodology was proposed for integrating monitoring methods into the Struman BMS, which is currently the South African Bridge Management System. The emphasis was placed on the technical aspects of the monitoring methods, with a touch on their practical aspects.

Previous studies have noted that using multiple technologies for condition assessments reduces error and provides high accuracy than standalone tests. The integration of results from different corrosion monitoring methods increases the confidence in detecting corrosion-related parameters and enhance the reliability of bridge condition rating process as the monitoring methods proved to be more efficient and timeous. It has also been observed that research on the use of multiple NDTs on existing structures for defect detection and condition assessment has evolved in the past decades; however, researches are very limited for remote monitoring methods.

Reasearches attempting to integrate monitoring methods into the BMS has been explicitly done in the USA, for the Pontis BMS. Researchers have developed ways of utilising quantitative and qualitative data from NDT methods for condition assessments in the BMS, as they believe NDT inform the BMS on the initiation process, extent and degree of deterioration of bridges. NDT also provide a better understanding of damage and deterioration that is not yet visible. It should be noted that no literature was found on the possible integration of remote monitoring into the BMS. In addition, no effort has been made yet on the subject of monitoring methods integration in the South African BMS.

A new integration process of corrosion monitoring methods into the Struman BMS was proposed, following the same approach that Struman BMS uses. Instead of using visual inspection data alone, data from corrosion monitoring methods are incorporated for condition rating and analysis. The process begins by identifying bridges that need to be assessed (new or existing bridges), choosing monitoring methods specific for the identified bridge and intended purpose of inspection, and then collecting data. The identification of bridges is based on criteria including the type of structure and material, exposure environment, the age of the structure and existing condition. Selecting monitoring methods for new bridges is based on the needs and the type of data that need to be collected from bridges, and for existing bridges, it depends on the past assessment data and visual inspection. Monitoring is proposed to incorporate periodic and permanent monitoring. The former includes NDT measurements using any selected methods on existing structures, and the latter includes installing sensor systems in new and existing bridges. Condition ratings were defined for monitored parameters, including corrosion risk (from corrosion potentials and electrical resistivity), corrosion rate, pulse velocity, defective areas (from the presence of cracks and delamination), moisture content and visual inspection. Other corrosion-related parameters considered include chloride content, cover depth measurement and monitoring time to corrosion onset. The obtained data from visual inspection and corrosion monitoring methods are then holistically analysed and interpreted to obtain integrated condition ratings, which are used for bridge condition analysis and allocation of remedial repairs.

Chapter 6

6 Conclusions and recommendations

6.1 Conclusions

The main aim of the present study was to identify monitoring methods for chloride-induced reinforcement corrosion-affected bridges to be included in the overall assessment, rating, and prioritisation of bridges for repair and maintenance in the Struman Bridge Management System (BMS). Based on the study of literature and the current approach of Struman BMS, the following conclusions can be drawn:

- Rebar corrosion is the most critical durability issue for RC structures, affecting both the reinforcing steel and the concrete. This durability problem reduces the service life of RC structures due to early deterioration, which results in defects such as cracking, delamination, spalling and rust staining. The early rebar corrosion damage often remains undetected in visual assessments, as they only become visible on the concrete surface after significant deterioration. This leads to high repair costs when the defects are detected too late. The rebar corrosion-related damage needs to be detected early to allow effective management of structures. Thus, monitoring technologies that detect corrosion-related damage at all stages within the service life of an RC bridge are needed.

- The main purpose of doing this research is to prevent this deviation when there is a reactive approach to only start doing major repairs once the damage has occurred; the risk of damage should be quantified early enough to prevent severe damage. Then proactive maintenance would be set in place.

- The review of various BMSs used in the USA, Canada, UK and Southern Africa shows that all BMSs differ in architecture, principles and assessment methodology. However, they all follow a similar underlying strategy in prioritising bridges for repair within limited budgets.

- In the USA, the Pontis BMS is commonly used. Their bridge inspections generally involve visual inspections, with the use of few non-destructive testing (NDT) to determine the condition of bridges. The USA Bridge Inspectors Reference Manual (BIRM) specified various NDT methods and destructive testing methods, however, only conventional methods including visual inspection, chain drag, covermeter, rebound hammer, and half-cell potential are used. These conventional methods are used only during damage and special inspections to determine the deterioration level, but their data are not used in the Pontis BMS (only visual inspection results are used for condition ratings).

- The Ontario BMS (OBMS) used in Canada is based on the Pontis BMS and follows the Ontario Structure Inspection Manual (OSIM) for bridge inspections. The OSIM standard defined the use of visual inspections, measurements (of crack openings or deflections) and condition surveys (deck condition assessment using GPR and thermography and load-carrying capacity assessment). Though the OSIM standard specifies these monitoring methods, agencies still focus on using traditional destructive methods.

- In the UK, the typical BMS, Highways Structures Management Information System (HiSMIS) use visual inspections to record visible deterioration during routine inspections. However, during special inspections often, material sampling and NDTs are used. The material sampling involves coring of concrete samples for strength determination and NDTs involve measuring concrete cover depths and half-cell potential.

- In the South African BMS (Struman BMS), condition ratings of RC bridges are based on visible defects assessed by visual inspection, which is still the dominant method used for bridge inspection. The same technique is used to monitor the deterioration of defects specified during the principal inspections. The assessment manual TMH19 of the Struman BMS did not define any other method. However, visual inspection does not adequately address the assessment of reinforcement corrosion. This provides an opportunity to refine the condition assessment process by incorporating corrosion monitoring methods in the BMS to allow earlier detection of damage in concrete.

- The present study critically reviewed the principles, applications and technical aspects of available corrosion monitoring technologies, including NDT and remote monitoring methods. Based on the review, it is evident that monitoring technologies are evolving, and the progress achieved to date is promising. However, further development is needed when using these methods in a BMS for condition assessments, particularly in continuous monitoring. The use of testing and monitoring technologies will allow earlier detection of durability problems, and hence promote evidence-based maintenance and damage prevention. This is expected to simultaneously mitigate the costs needed for maintenance and repair. In addition, remote monitoring methods can improve the speed and extend the scope of condition assessment, provide reliable and relevant data, and reduce traffic interruption during the assessment.

- NDTs help detect surface and subsurface defects associated with reinforcement corrosion in RC structures. The NDTs reviewed in this study include electrochemical, elastic wave, electromagnetic and thermal methods. Some methods can detect corrosion at an early stage during the corrosion initiation (e.g., Half Cell Potential (HCP), Chloride content) and propagation (e.g., HCP, Electrical Resistivity (ER), Ground Penetrating Radar (GPR), Linear Polarisation Resistance (LPR), Ultrasonic Pulse Velocity (UPV), Impact Echo (IE), Acoustic Emission (AE) and Infrared Thermography (IRT)) and others can detect corrosion damage indicators during the acceleration stage (e.g., visual inspection). Among these methods, the HCP measurements indicate the likelihood of reinforcement corrosion in concrete and LPR indicates the corrosion rate. The ER measurements supplement HCP or LPR measurements and help interpret their results. UPV, IE, AE and IRT detect and locate flaws such as internal cracks, voids, honeycombing and delamination resulting from reinforcement corrosion. Covermeter and GPR are used to locate reinforcement and cover thickness. The NDT methods are thus found to be applicable for both laboratory and field assessments in new and existing structures.

- Six remote monitoring systems are reviewed: Anode Ladder System (ALS), Expansion Ring System (ERS), Multi-ring Electrode (MRE), CorroWatch (CW), CorroRisk (CR) sensors and potentiometric sensors. The ALS, ERS, CW and CR measures the critical depth of chloride content, thereby determining the time to corrosion initiation. MRE measures moisture content and potentiometric sensors measure chloride concentrations. It was found that these monitoring systems are applicable for use in new structures (e.g., ALS, MRE, and CW) and existing structures (ERS, MRE, CR, and ISE sensors). In new structures, the systems are installed prior to concrete placement during the construction stage and in existing structures, the systems need to be installed or drilled into the concrete.

- In the South African context, corrosion monitoring methods applicability is very limited to visual inspection. Seemingly because these methods required more time to be accepted as reliable for use by different agencies, and visual inspection has been the traditional and norm method that is simple to be carried out. More work is needed in demonstrating the feasibility of corrosion monitoring methods to bridge managers and decision-makers.

- Previous studies that used integrated methods for condition assessment indicated that no particular NDT or remote monitoring method is complete. A single technique is generally not sufficient for detection and quantification of all relevant defects; each method has its own advantages and limitations. Combining multiple methods for evaluating reinforcement corrosion damage reduces errors, compensates for each method's limitations, and provides higher accuracy compared to a standalone test. As a consequence, multiple monitoring methods should be used to evaluate RC structures, and to provide a complete condition assessment.

- Reasearches attempting to integrate monitoring methods into the BMS has been carried out specifically in the USA, for the Pontis BMS. In these studies, quantitative and qualitative data from NDT methods were used for condition assessments in the BMS. It should be noted that no literature was found on the possible integration of remote monitoring into the BMS. In addition, no effort has been made yet on the subject of monitoring methods integration in the South African BMS.

- Typical monitoring methods that can be incorporated in the Struman BMS include HCP measurement for determining the corrosion risk, LPR for corrosion rate, cover depth and chloride measurements, sensors for determining the corrosion onset in new structures, and other methods for detecting the internal damage such as internal cracking, delaminations and voids. ER measurements are also useful for interpreting HCP and LPR results and the indication of moisture in the concrete. From the practical point of view, HCP is an accepted and improved method for condition assessment in South Africa. The measurements are quite easy to do and do not need equipment. Corrosion rate measurements are a bit specialised and difficult to obtain on-site. Hence, based on the current industry experience, HCP is the better method for once-off assessment.

- A new integration process of corrosion monitoring methods into the Struman BMS is proposed. The proposed methodology follows the same approach that Struman BMS uses, except that instead of using visual inspection data alone, data from corrosion monitoring methods are incorporated for condition rating and analysis. The approach includes the following summarised steps:
    i. Identifying bridges that need to be assessed, using the following criteria: the structure and material, environmental condition, age of the structure, and existing condition.
    ii. Choosing monitoring methods specific for the selected bridges and intended purpose of inspection, which is vital to save time and resources. A guideline process for selecting a method is outlined.
    iii. Collecting inspection data using the DERU rating system for visual inspection and selected monitoring methods for bridge condition assessment of invisible defects. Both periodic and permanent monitoring

are considered for data collection; the former is based on NDT measurements using any of the selected NDT methods, the latter includes installing sensor systems in new bridges and existing bridges.

iv. Condition ratings are defined for monitored parameters, including corrosion risk (from corrosion potentials and electrical resistivity), corrosion rate, pulse velocity, defective areas (from the presence of cracks and delamination), moisture content and visual inspection. Other corrosion-related parameters considered include chloride content, cover depth measurement and monitoring time to corrosion onset.

v. The obtained condition ratings from visual inspections and corrosion monitoring methods are integrated to form the Integrated Condition Rating (ICR) for bridge elements. The condition ratings for corrosion monitoring methods provide the severity of the measured corrosion parameters. The ICR is then used for bridge condition analysis (bridge condition ratings and ranking for prioritisation) and allocation of remedial repairs.

- The proposed approach presented in the present study just provided the underlying philosophy and general structure for the possible integration of corrosion monitoring methods into the Struman BMS. This looks into principles as to which methods to apply; the practicality of the reviewed methods still have to be developed, which include the testing frequencies and locations of testing. More work is thus required to evaluate the practicality and applicability of existing corrosion monitoring methods to ensure their acceptance in bridge inspections. This will also help to practically integrate these methods into the South African BMS.

- Monitoring methods can thus supplement visual inspection to improve condition assessment in BMSs. Integrating these methods with the existing approach is expected to compensate for their limitations and enhance their capability.

6.2 Key challenges and recommendations

Based on all findings during the course of this research, several challenges were identified, which are discussed in this section. Potential recommendations for addressing the identified challenges are suggested for future research.

- Most available corrosion monitoring technologies are evaluated under laboratory settings, which is different from in-service bridge conditions. The laboratory setting allows controlling of critical parameters such as temperature, RH, concrete moisture content, concrete materials and their properties, and uses smaller specimens compared to a real-life scenario. This does not in all cases provide a good indication of all the relevant factors influencing the implementation of these methods on site. A comprehensive study of the implementation of these corrosion monitoring methods and the factors influencing them on in-service bridges need to be conducted.

- It is observed that the use of methods such as IE, UPV, AE, GPR and IRT for reinforcement corrosion monitoring has emerged in the past decade and it is still developing. These methods still need to be understood more through research, hence more investigation is needed in this regard.

- From the practical point of view, HCP measurements are an accepted and improved method for condition assessment in South Africa. The measurements are quite easy to do and do not need equipment. Corrosion rate measurements are a bit specialised and challenging to obtain on-site. However, based on the review, there seem to be quite new technologies available, which have been applied elsewhere, such as the Giatec iCOR available in USA and Canada, so for application in South Africa and the Struman BMS, these methods should be investigated in detail. In addition, only one of these two methods (HCP or corrosion rate) can be recommended; they cannot be used both.

- The present study has presented an approach of integrating data of the selected monitoring methods that can be implemented on bridge elements. Results are collected from standalone technologies and integrated manually as one output. Future research needs to focus on the automation of this integration process as to be able to obtain one output from multiple technologies automatically.

- The author of this research is not experienced in bridge inspection, therefore the findings of this research need to be taken by an experienced bridge inspector to help develop practical guidelines for applying corrosion monitoring methods in the Struman BMS. Hence, the corrosion monitoring methods will be able to be included in the detailed planning of new monitoring of bridges.

- The introduction of the Internet of Things and Artificial intelligence as new technologies for data management by bridge owners can allow complete automation and allow the development of effective decision-making procedures. Big data and machine learning tools will also enable improving the use of data, especially to establish deterioration models for bridges future conditions. These processes will allow real-time data capture and potentially shorten the time required for the inspection process.

- Working closely with the Transport officials and bridge inspectors will help further develop and implement monitoring systems into the Struman BMS. The use of these technologies are evolving with increasing benefits, and transport agencies should move towards adapting them for condition assessment.

- Evaluation should be carried out through surveys for reactions of BMSs users towards the proposed approach.

7 Appendix

7.1 Appendix A: Bridge inspection practice in various BMSs

Table 7-1:Inspection type and intervals in various countries [39,42,44,48,49]

Country	Inspection type	Interval	Description
South Africa	Monitoring inspection	As needed	Involve a quick look for new defects and the status of known defects. It is also conducted after accidents or extreme events such as floods or cyclones.
	Principal inspection	Every 5 years	Comprehensive visual inspections of the whole bridge that record all defects
	Verification inspection	~60 bridges per year	Done as part of quality assurance to verify the accuracy of inspection data
	Acceptance inspections	After project completion	Carried out after completion of a new structure or maintenance and rehabilitation projects
	Project-level inspection	Before repair project	Inspections done to collect information for contract documents
USA	Initial inspection	As needed	First inspection carried out on new structures to capture inventory and appraisal data
	Routine inspection	Every 2 years	Regular inspection based on observations and measurements to determine the condition of bridges and identify any changes from previously recorded conditions
	In-depth inspection	Every 5 years	A close-up inspection using NDT* and destructive test e.g. core sampling, hammer and chain dragging, covermeter, GPR**, and petrographic examinations
	Fracture-critical member inspection	Every 2 years	A hands-on inspection of a fracture-critical member that may include visual and other NDTs
	Special inspection	As needed	Scheduled inspections used to monitor particular known or suspected deficiencies
	Damage inspection	As needed	An unscheduled inspection to assess structural damage resulting from environmental factors or human actions
	Underwater inspection	Every 5 years	Inspection of the underwater portion of a bridge substructure and the surrounding channel that cannot be inspected visually at low water by wading or probing, generally requiring diving or other appropriate techniques.
Canada	Routine inspection	Frequent	Involve general visual inspections on bridges with spans over 6 m.
	Hands-on inspection	As needed	Used in response to extreme events such as floods, accidents, or critical findings.

	Detailed inspection	Every 2 years	Involve detailed observations and review of previous reports (inspection or maintenance work). Sketches, pictures, and simple measurements are taken.
	Condition survey	Every 5 years	In-depth inspections that require access to the assessed area and measurements of all areas of defects. It incorporates bridge deck condition assessment using NDTs, delamination survey, and structural evaluation.
	Damage inspection	As needed	An unscheduled inspection to assess structural damage resulting from environmental factors or human actions
	Underwater inspection	Range from 5 to 10 years	Inspections in water depths of greater than 1 m under the bridge elements
UK	Superficial inspections	Frequent	Frequent visits to bridges and urgent inspection done after impact damage from flood, natural disasters or unexpected damage on the bridge.
	General inspection	Every 2 years	Involve visual inspections of all elements without access to the structure to provide information on the visible condition.
	Principal inspection	Every 6 years	Provide detailed observations within 1 m proximity to the structure, reporting all conditions and noting all defects.
	Special inspection	As necessary	Are used to determine the cause and extent of deterioration of identified defects. Usually involve extensive NDT and material sampling.
	Acceptance inspections	N/A	Inspections done after completion of projects or when contracts for maintenance changes

*NDT – Non-Destructive Testing also known as Non-Destructive Evaluation
**GPR – Ground Penetrating Radar

Table 7-2: NDT and destructive methods specified by the USA Bridge Inspectors Reference Manual [54] for concrete structures

Testing methods for concrete structures	**Description**
Ultrasonic velocity measurements	Delineate areas of internal cracking (including delamination), estimate strength and elastic modulus.
Electrical methods	Half-cell potentials used to evaluate the corrosion activity of reinforcing steel embedded in concrete.
Delamination detection machinery	Determines delaminated areas
Ground-penetrating radar	Measures the thickness of the concrete and acquire subsurface information such as concrete flaws and delaminations
Impact echo testing	Determine the location and extent of defects such as cracks, delaminations, voids, honeycombing and debonding in RC
Infrared thermography	Infer subsurface delamination based on surface temperatures
Laser ultrasonic testing	Using laser to locate flaws and position of rebars in concrete
Magnetic field disturbance	Evaluate fatigue damage to rebars in concrete members.
Neutron probe	Detect chlorides in construction materials.
Nuclear methods	Measure the moisture content in concrete
Pachometer	Determining the position of reinforcement
Rebound and penetration methods	Measure the hardness of concrete and can be used to predict the strength of concrete.
Smart concrete	Carbon fibre-reinforced cement can be used as a strain-sensing coating on conventional concrete.
Radiography	Evaluate concrete for signs of hidden flaws.
Carbonation depth	Indicates the probability of reinforcement corrosion due to carbonation of concrete
Concrete permeability	Measure air and water permeability through the concrete
Concrete strength	Determine the actual concrete strength and quality
Moisture content	Serve as an indicator of corrosion activity
Petrographic examination	Determine the characteristics of hardened concrete
Reinforcing steel strength	Determine the actual properties of the reinforcing steel
Chloride test	Used to assess the resistance of concrete to chloride ions penetration

7.2 Appendix B: Summary of corrosion monitoring methods

Table 7-3: Summary of corrosion monitoring methods used in RC structures

Methods	Principles	Parameter measured	Advantages	Limitations	Reference
Non-destructive corrosion monitoring methods					
Electrochemical methods					
Half-cell potential	Measure electrical potential difference between steel reinforcement of RC and reference electrode (indicates corrosion potential of the embedded steel in concrete)	corrosion potential level (mV or V)	• The equipment used are lightweight and portable. • Indicate the probability of corrosion • Not expensive	• It only evaluates corrosion risk and do not assess corrosion rate. • Its semi-intrusive as it requires connection to the rebar • It is difficult to establish a connection to steel when the concrete cover is large. • It cannot give reliable results with coated concrete surfaces or in the presence of overlays e.g. on bridge decks • Not applicable to epoxy-coated bars • Require lane closure	[56,57,63, 74]
Concrete resistivity (Wenner configuration)	Measure resistivity (ρ) of RC, which allows ease flow of current between anode and cathode areas of the concrete.	Resistivity (ohm.cm)	• Non-destructive, easy, portable and fast technique • Indicates areas of high corrosion risk • Provide insight on concrete durability	• Used in conjunction with HCP, hence cannot be used alone for assessment • Influenced by the concrete exposure conditions and its composition. • The presence of reinforcement in the test region can provide short-circuiting, hence affect results	[56,57,63, 74]

Linear Polarization resistance	The change in potential during reactions (polarization) is recorded using an electrode probe on the concrete surface.	corrosion current density (Icorr- A/cm2)	• The equipment used are lightweight and portable. • Indicate the instantaneous corrosion rate at the time of measurement	• Require electrical connection to the rebar • Not applicable to epoxy-coated bars. • Cover depth has to be less than 100 mm. • Testing and interpretation has to be performed by experienced personnel. • Determining the polarized area is difficult and time-consuming.	[56,57, 74]
iCOR	Same as linear polarization resistance	Rebar corrosion rate, Concrete resistivity Corrosion potentials	• Requires no connection to the rebar • Its wireless, fast and easy to use • It has a user-friendly app that can be upgraded	• Not applicable to epoxy-coated bars.	[72,73]
Elastic wave methods					
Ultrasonic Pulse velocity	Mechanical energy transmits through the concrete as stress waves and is converted into electrical energy by a second transducer.	Pulse velocity (V)	• The equipment used are lightweight, portable and simple to use. • Provide an indication of the uniformity and quality of concrete • It can survey a large coverage and thick members • Estimate the size and nature of defects • Results are relatively easy to interpret	• Require expert analysis and careful data collection • Less effective for irregular shapes or very thin members. • Require access to both sides of the structure. • Results are influenced by moisture and the presence of reinforcement.	[56,74], [94, 140]
Impact echo	Stress waves are propagated within the concrete through vibrations and impact load.	Wave velocity (Vp)	• It is quick, accurate and reliable • Only need access to one side of the structure • Detect cracks, voids, delaminations and debonding • Determine thickness	• Reliability decreases with an increase in thickness, and accuracy depends on the impact duration • Less reliable on elements with overlays • Difficult interpretation of results	[56,74], [94, 140]

Acoustic emission	Elastic waves are generated from a localized source within the RC member and detected by a sensor on the concrete surface.	Cracks, delaminations, corrosion risk	• A cost-effective and sensitive method that can detect and locate active defects such as cracks • Require no lane closure	• Passive defects cannot be effectively detected. • Provide qualitative results	[56]
Electromagnetic methods					
GPR	An electromagnetic impulse is transmitted into the concrete, which is reflected to the antenna and produce an output signal.	Delamination Rebar depth, location and size Concrete thickness Bridge deck condition	• It is fast at testing • Can assess bridge decks with or without overlays • Inspection results are reproducible. • Provide good quantity estimates and general locations of defects. • Minimal traffic control • Commercial equipment and application softwares are well-developed.	• It does not give information on corrosion rate. • It is sensitive to moisture, metal objects and electrical conductivity • Less effective as concrete thickness increases • Require experts or trained operators for inspection and interpretation of results.	[56,74]
Covermeter	Use an alternating magnetic field to locate the steel and other magnetic materials.	Depth of concrete cover Rebar coverage	• It is fast, simple and portable	• Cannot directly detect reinforcement corrosion	[56]
Infrared thermography method					
Infrared thermography	Detect the temperature gradient within the concrete under heat exposure.	Thermal differentials, delaminations, cracks, voids	• Does not require access to the inspected element. • Can assess a large coverage in a short time period • Portable, simple with easy interpretations • Minimal traffic interference	• It is sensitive to contaminants • The equipment are expensive • It is unable to detect defects at greater depths • Its ineffective on decks with overlays • Need adequate solar radiation to obtain the required temperature differentials • Need experts for interpretation of results.	[56]
Remote monitoring methods					

Anode-ladder system	Determine the onset of corrosion of each of the anodes (sensors) placed at different depths in the concrete cover at any time against the cathode, provided the cover to reinforcement is known.	Critical depth of chloride content	• Warn bridge owners to take proactive measures prior to damage depiction • Applicable in inaccessible areas • They have a long design life	• Not suitable for use in submerged elements • Do not provide absolute chloride content • Cannot be installed where poker vibrator are used for compaction • Need to be installed near the concrete surface	[107] [106] [105]
Expansion ring system	Determine the onset of corrosion of each of the anodes (sensors) placed at different depths in the concrete cover at any time against the cathode, provided the cover to reinforcement is known.	Critical depth of chloride content	• Warn bridge owners to take proactive measures prior to damage depiction • Easy and fast to install • Applicable in inaccessible areas	• Not suitable for use in submerged elements • Do not provide absolute chloride content	[109] [103,104]
CorroWatch and CorroRisk	Determine the onset of corrosion of each of the anodes (sensors) placed at different depths in the concrete cover at any time against the cathode, provided the cover to reinforcement is known.	Critical depth of chloride content	• Applicable in inaccessible areas	• All anodes need to be connected to the monitoring box	[75]
Multi-ring electrode	Measures the electrical resistance between any adjacent rings and provide a resistance profile across the sensor depth.	Electrical resistance	• Its used in both new and existing structures • Easy and fast to install	• The concrete-specific calibrated curves need to be calibrated always • Considers the properties of the anchoring mortar used in installation, in the interpretation of the results	[115] [110,112]
Potentiometric sensors	The potential difference between the reference electrode and the sensor is measured and associated with the dissolved chloride ions.	Chloride concentration	• Easy and fast to install • They are very sensitivity, stable and robust • Requires low cost for preparation	• -	[116–118] [119,120]

Table 7-4: Corrosion related parameters measured and assessed by different monitoring methods

Monitoring methods	**Corrosion potential**	**Corrosion current**	**Corrosion rate**	**Electrical resistance**	**Chloride concentr-ation**	**Moisture content**	**Pulse velocity**	**Internal cracking**	**Delami-nation**
Half-cell potential	X	-	-	-	-	-	-	-	-
Electrical resistance	-	-	-	X	-	-	-	-	-
Linear polarization resistance	-	-	X	-	-	-	-	-	-
Ultrasonic pulse velocity	-	-	-	-	-	-	X	-	-
Impact echo	-	-	-	-	-	-	-	-	X
Acoustic emission	-	-	-	-	-	-	-	X	-
Ground-penetrating radar	-	-	-	-	-	-	-	X	X
Covermeter	-	-	-	-	-	-	-	-	-
Infrared thermography	-	-	-	-	-	-	-	X	X
Anode ladder system	X	X	-	X	-	-	-	-	-
Expansion ring system	X	X	-	X	-	-	-	-	-
Multiring electrode	-	-	-	X	-	X	-	-	-
CorroWatch	X	X	-	-	-	-	-	-	-
CorroRisk	X	X	-	-	-	-	-	-	-
ISE sensors	X	-	-	-	X	-	-	-	-

7.3 Appendix C: Evaluation of corrosion monitoring methods with general criteria

Table 7-5: Evaluation of corrosion monitoring methods with general criteria

Criteria	VI*	HCP*	ER*	LPR*	IE*
Nature of method	Surface	Steel level	Covercrete	Steel level	Subsurface
Types of defects detected	Surface cracks, surface delaminations, spalling, rust staining	Rebar corrosion	Rebar corrosion	Corrosion rate	Internal cracks, delaminations, voids
Detection depth	None	Varies	Varies	Varies	Good results for defects > 51 mm
Installation/setting up speed	Not required	Fast	Not needed	Fast	Fast
Testing speed	Fast	Fast	Fast	Fast	Slow
Results interpretation	Depends on the inspector's judgement	Needs processing	Instantaneous	Real-time	Needs processing
Requirement of lane closure	Not required	Require minimal lane closure	Require minimal lane closure	Require minimal lane closure	Requires closure
Maintenance requirement	Not required	Required	Required	Required	Required
Surface preparation during testing	Not required	Surface cleaning and dampening needed	Surface cleaning and dampening needed	Surface cleaning required	Surface cleaning needed and chipping if rough
Accuracy	80 to 85% effective	± 1 mV	From ± 0.2 to ± 2 kΩcm	± 11 mV	Depth calculation precision > 95%
Equipment cost	None	Low	Low	Moderate	Low
Access required to the assessed element	No	Yes	Yes	Yes	Yes

*VI – Visual Inspection
HCP – Half-Cell Potential
ER – Electrical Resistance of concrete
LPR – Linear Polarization Resistance
IE – Impact Echo

Criteria	UPV*	AE*	Covermeter	GPR*	IRT*
Nature of method	Subsurface	Subsurface	Covercrete	Subsurface	Surface
Types of defects detected	Internal cracks, delaminations, voids	Internal cracks, delaminations, voids	Locate depth, size and location of reinforcing steel	Determine corrosive environment, location of steel and cover thickness	Surface and internal cracks, delaminations, voids
Detection depth	Depth of penetration 500 to 2000 mm	Varies	Good results at a depth of approximately 80 mm	Good results for defects deeper than 25 mm	Good results up to 51 mm
Installation/setting up speed	Slow	Slow	Fast	Fast	Fast
Testing speed	Relatively slow	Fast	Fast	Fast	Fast
Results interpretation	Needs processing	Real time	Instantaneous	Needs processing	Real time
Requirement of lane closure	Require lane closure	Only require lane closure during installation	Require minimal lane closure	Require closure if ground couple antennas are used	Require minimal lane closure
Maintenance requirement	Required	Required	Required	Required	Required
Surface preparation during testing	Sometimes require coupling gel	Not needed	Cleaning	Cleaning and surveying work	Cleaning
Accuracy	Accuracy ranging from 52 to 99 %.	Ranging from 40 to 60 dB	± 1 mm to ± 4 mm	Depth calculation precision > 95%	±2ºC or 2% of the reading
Equipment cost	High	High	Low	High	Moderate
Access required to the assessed element	Yes	Only during installation or maintenance	Yes	Yes	Yes

*UPV – Ultrasonic Pulse Velocity
AE – Acoustic Emission
GPR – Ground Penetrating Radar
IRT – Infrared Thermography

Criteria	ALS*	ERS*	MRE*	CW*	CR*	Ag/AgCl ISE*
Nature of method	Covercrete	Covercrete	Covercrete	Covercrete	Covercrete	Covercrete
Types of defects detected	Monitor** onset corrosion	Monitor*** onset corrosion	Monitor moisture content	Monitor** onset corrosion	Monitor*** onset corrosion	Monitor*** chloride concentration
Detection depth	Until the steel level	Until the steel level	Within the range of 5 – 45 mm	Within 60 mm from the surface	Until the steel level	Not limited
Installation/setting up speed	Fast	Fast	Fast	Fast	Fast	Fast
Testing speed	Fast	Fast	Fast	Fast	Fast	Fast
Results interpretation	Real-time	Real-time	Real-time	Real-time	Real-time	Real-time
Requirement of lane closure	Not required	Only required during installation	Only required during installation	Not required	Only required during installation	Only required during installation
Maintenance requirement	Required	Required	Required	Required	Required	Required
Surface preparation during testing	Not required	Cleaning	Cleaning	Not required	Cleaning	Cleaning
Accuracy	Voltage – 0.2 mV Current – 1 µA LPR* – ± 0.009 % ER* – 1 % Temp – ± 0.1 °C	Voltage – 0.2 mV Current – 1 µA LPR – ± 0.009 % ER– 1 % Temp – ± 0.1 °C	Accuracy of ± 0.35 %	-	-	Accuracy of ± 4 %
Equipment cost	Low	Low	-	-	-	low
Access required to the assessed element	Only during maintenance	Only during installation and maintenance	Only during installation and maintenance	Only during maintenance	Only during installation and maintenance	Only during installation and maintenance

*ALS – Anode Ladder System
ERS – Expansion Ring System
MRE – Multi-ring Electrode
CW – CorroWatch
CR – CorroRisk
Ag/AgCl ISE – Silver/Silver Chloride Ion-Selective Electrode

LPR – Linear Polarization Resistance
ER – Electrical Resistance

**Applicable in new structures
***Applicable in existing structures

7.4 Appendix D: Ethics approval

Application for Approval of Ethics in Research (EiR) Projects
Faculty of Engineering and the Built Environment, University of Cape Town

ETHICS APPLICATION FORM

Please Note:
Any person planning to undertake research in the Faculty of Engineering and the Built Environment (EBE) at the University of Cape Town is required to complete this form **before** collecting or analysing data. The objective of submitting this application *prior* to embarking on research is to ensure that the highest ethical standards in research, conducted under the auspices of the EBE Faculty, are met. Please ensure that you have read, and understood the **EBE Ethics in Research Handbook** (available from the UCT EBE, Research Ethics website) prior to completing this application form: http://www.ebe.uct.ac.za/ebe/research/ethics1

APPLICANT'S DETAILS		
Name of principal researcher, student or external applicant		Jaziitha Simon
Department		Civil Engineering
Preferred email address of applicant:		smnjaz001@myuct.ac.za
If Student	Your Degree: e.g., MSc, PhD, etc.	MSc in Engineering specialising in CIMM
	Credit Value of Research: e.g., 60/120/180/360 etc.	120
	Name of Supervisor (if supervised):	Prof. Hans Beushausen
If this is a researchcontract, indicate the source of funding/sponsorship		No its not a research contract.
Project Title		Monitoring of reinforced concrete bridges subjected to chloride-induced reinforcement corrosion

I hereby undertake to carry out my research in such a way that:

- there is no apparent legal objection to the nature or the method of research; and
- the research will not compromise staff or students or the other responsibilities of the University;
- the stated objective will be achieved, and the findings will have a high degree of validity;
- limitations and alternative interpretations will be considered;
- the findings could be subject to peer review and publicly available; and
- I will comply with the conventions of copyright and avoid any practice that would constitute plagiarism.

APPLICATION BY	Full name	Signature	Date
Principal Researcher/ Student/External applicant	Jaziitha Simon		19/02/2021
SUPPORTED BY	Full name	Signature	Date
Supervisor (where applicable)	Hans Beushausen		21/02/21

APPROVED BY	Full name	Signature	Date
HOD (or delegated nominee) Final authority for all applicants who have answered NO to all questions in Section 1; and for all Undergraduate research (Including Honours).	Prof. Alphose Zingoni		31/05/2021
Chair: Faculty EIR Committee For applicants other than undergraduate students who have answered YES to any of the questions in Section 1.			

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Figure 7-1: Ethics approval

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