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An analysis of the predominant causes of deterioration of concrete structures in South Africa

Minor dissertation submitted for the partial fulfilment of the Master of Engineering in civil infrastructure management and maintenance

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DEDICATION

To the Almighty and my family

ABSTRACT

Concrete deteriorates due to, but not limited to the ingress of deleterious substances which react with the cement matrix, reinforcing bars corrosion, mechanical effects, physical effects, structural damages, poor construction practices. All these factors individually or combined, ultimately reduce the expected service lives of the concrete structures. The trends vary with different exposure conditions and geographical locations, and a reference guide is required in South African context.

A total of twenty-four concrete structures were visually assessed by different University of Cape Town (UCT) scholars and findings were captured in project reports. The reports of these assessments were analysed in this research to identify the main causes of concrete deterioration and severity of damages in the three provinces considered in South Africa, whilst linking these to environmental exposure conditions and geographical location. It is important to elucidate that deterioration mechanisms and trends were drawn from the limited number of visual assessment reports, and the mechanisms assumed might not have been necessarily correct.

The rating of the defects was done using the DER-U rating system, a method available for bridges and retaining walls. DER-U rating system was developed for buildings, exploiting the available rating system for bridges as there is no available established rating system for buildings, and the author considered it an important tool for the preliminary evaluation taking note of all limitations. However, reinforcing bars corrosion has been found to be the most prominent deterioration mechanism on structures assessed and severity was high on the structures located in the Western and Eastern Cape provinces, and was exacerbated by the inadequate cover provided on most structures. Furthermore, it was also noted that the severity of the damage increased with age of a structure.

Although petrographic analysis as an additional investigation was required to ascertain Alkali-Silica Reaction (ASR), damage was observed in the Western Cape and Gauteng provinces. Even though the occurrence was low, it still required special attention as the effects are usually disastrous and very expensive to maintain the affected structures. Leaching was observed on all the bridge structures assessed though it was more prominent on the structures situated at the coast. Plastic and drying shrinkage cracks were observed on all structures in the Gauteng province and it has been noted from the literature that shrinkage cracks were exacerbated by

very high seasonal temperatures in these provinces. Abrasion was high on all structures on the tidal zones and the elements of structures located in the water courses.

The proposed in-situ and laboratory tests have been discussed in this report and they are recommended for full-scale condition assessments to complement the visual assessments in an endeavour to ascertain the mechanisms identified. Evidence of poor maintenance practices was observed in the Eastern Cape province where delamination and spalling were observed on freshly repainted structures. As a result, in South Africa there is undoubtedly, a constant need of developing and employing effective and efficient tools to ensure quality is not compromised. Design engineers must always take into cognisance the exposure conditions and ensure strict quality control measures during the construction phase. Maintenance engineers should take into consideration the location of the structure and deterioration mechanisms in the specific areas when determining the maintenance strategies. The clients should always employ knowledgeable design and maintenance engineers, to ensure durable structures are erected and correctly maintained.

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CHAPTER 1

1. INTRODUCTION

1.1 Background and overview

Concrete is widely used in the construction of civil infrastructure worldwide which includes; buildings, bridges, drainage systems, concrete pavements, etcetera. The excellent durability characteristics if correctly designed and placed results in a very long service life (Beushausen & Alexander, 2009). Furthermore, its low maintenance requirements have led to its acceptance in the construction industry.

Although concrete has been accepted as durable, it is not immune to deterioration as the structures are usually exposed to aggressive environmental conditions during their service lives. It is also important to note that numerous existing concrete structures in South Africa have deteriorated to a state where they require urgent intervention. Beushausen and Alexander (2009) state that concrete deterioration has a direct bearing on durability hence the development of performance-based durability testing in South Africa to help mitigate the problems associated with concrete deterioration. Problems associated with concrete deterioration range from aesthetics to the decrease of concrete structures' service lives.

Concrete deteriorates when deleterious substances react with the cement matrix affecting the expected service lives of the structures (Ballim, Alexander & Beushausen, 2009). Deterioration occurs in two forms which are; degradation of the concrete itself and anodic or cathodic breakdown of reinforcing bars. There are a variety of deterioration mechanisms, discussed in more detail in Section 2.1 of this document, and these include but not limited to the following; corrosion of reinforcing bars, ASR, chemical attack, fire, shrinkage, impact, construction defects, and abrasion.

The deterioration mechanisms probabilities vary with changing environmental exposure conditions. The environmental conditions differ from one area to the other, and thus trends can be established. Establishing the trends of concrete deterioration mechanisms is discussed in Section 3.3.

The deterioration mechanisms of concrete structures are typically ascertained after full-scale condition assessment. This research analysed the deterioration mechanisms of structures that had been visually assessed by UCT scholars in their postgraduate course assignments. According to Beushausen and Alexander (2009), the main reasons for the damage of concrete structures can be established and determined by implementing systematic visual survey, thus the acceptance of the visual assessments reports done by other scholars for establishing the trends.

The knowledge on deterioration mechanisms and trends assists Maintenance Engineers in the implementation of informed maintenance and repair strategies. Design Engineers also make informed decisions during the design stage by taking into consideration the exposure conditions of the proposed structures and ensuring strict quality control measures during the construction phase.

1.2 Research objectives

The objectives of this research are as follows:

- To review literature on deterioration mechanisms and condition assessments of concrete structures,
- To analyse the condition assessment results done by other scholars of which the visual assessment findings have been correlated to the common deterioration mechanisms, and
- To establish trends of the predominant causes of concrete deterioration with emphasis on the South African context.

1.3 Dissertation structure

Chapter 1: Introduction

This section introduces the topic and the objectives of the research.

Chapter 2: Literature review

This chapter provides a literature review on various concrete deterioration mechanisms, problems and condition assessments from available literature. It also discusses the rating system used for the defects severity.

Chapter 3: Case studies

Chapter 3 focuses on the analysis of the condition assessments reports done by other scholars on different structures. Condition assessments of twenty-four reinforced concrete structures which were conducted by other scholars in South Africa have been analysed. The assessments were done to evaluate concrete deterioration and to determine the causes of deterioration as well as the severity of damages. The structures evaluated are in only three different provinces of South Africa where the environmental exposure conditions are different. Maps showing the location of the structures are included.

The likelihood of occurrence of each deterioration mechanism in different areas has been analysed.

Chapter 4: Conclusion

It provides the findings on concrete deterioration on the structures assessed, trends of concrete deterioration in the three regions considered and the severity of the defects.

References

This section has a list of references and is included at the end of this document.

1.4 Methodology

Since the thesis consists of a critical literature review and analysis of predominant causes of deterioration of concrete structures in South Africa using assessment reports done by other scholars, a qualitative research was performed by reviewing journals that were published in the past, as well as recent published works. Books, technical reports and technical method for highways (TMH) manuals were also considered. In addition, TMH 19 and 22 manuals, designed for road structures assessments and rating of defects, were considered in an endeavour to standardize the rating of defects.

Furthermore, the DER-U rating system was exploited for buildings which are non-road structures. The different types of structures and relevant inspection items were rated and analysed separately. However, it is important to note that the rating system was developed for bridges and retaining walls, and is not necessarily directly applicable to buildings. And since no method for buildings exists, the rating system was considered an appropriate tool for a preliminary evaluation of the severity and significance of damage observed.

1.5 Limitations

- This research was confined only to the three provinces of South Africa where condition assessments of selected concrete structures were conducted and these provinces are; Eastern Cape, Western Cape and Gauteng.
- The case studies were based on available and a limited number of assessed structures, hence may not represent the trends across South Africa and generalisations not possible.
- The sample sizes for the three provinces and different types of structures analysed varies significantly, hence conclusions drawn may not necessarily be a true

representation of the structures in the provinces considered in this research, although the information has been considered useful in coming up with preliminary evaluation trends of concrete deterioration.

- There is a possibility that scholars chose the structures that had major damages in-order to write comprehensive reports, hence the data may not represent the actual distribution of damages across South Africa.
- The research was based on the available literature for critical review of deterioration mechanism and there could be some other mechanisms not identified or it could be that the literature was not readily available.
- There was time constraint related to carrying out full-scale condition assessments to sample the results to ascertain some of the information collated. Therefore, reservations were given to conclusions drawn from visual assessments done by other scholars.
- The severity of the damages was rated based on the DER-U rating system. The condition stipulated for the use of the rating system in TMH19 as developed by COTO, (2013) is that, relevancy rating can never be greater than degree rating for bridges rating. Applying the same principle for buildings where the degree rating for cracking is dependent on the crack width, relevancy of cracks on the buildings may be underestimated.
- The DER-U rating system was developed for bridges and retaining walls only, and is not necessarily directly applicable to buildings. And since no method for buildings exists, the rating system was considered an appropriate tool for a preliminary evaluation of the severity and significance of damage observed.
- Deterioration caused by structural design errors was not considered in this research.

CHAPTER 2

2. LITERATURE REVIEW

Concrete is composed of three essential components which are; water, aggregates, and cement of which cement is the binding agent. Cement paste gives concrete its alkalis properties, and deterioration is related to the change in its properties due to harsh environmental conditions. Deterioration results in a broad range of problems which include; affecting aesthetics, exorbitant repair costs and strength loss of concrete structures changing the long-term performance (Stewart, Wang & Nguyen, 2011). Concrete deterioration occurs in two forms which are; deterioration of the concrete itself and reinforcing bars corrosion which is the anodic or cathodic breakdown of reinforcing bars.

2.1 Concrete deterioration mechanisms and associated problems

There is a variety of concrete deterioration mechanisms worldwide. However, the mechanisms do not necessarily occur in isolation, and the probability of occurrences is influenced by the presence of deleterious substances which create an environment conducive for concrete deterioration to occur (Portland Cement Association, 2002). Selected mechanisms have been covered in this research as they are linked to deterioration of structures considered.

2.1.1 Reinforcing bars corrosion and associated problems

Corrosion is one of the mechanisms that lead to concrete deterioration. All the reinforced concrete structures visually assessed have suffered deterioration due to reinforcing bars corrosion. It may then be concluded that reinforcing bars corrosion is the most common deterioration mechanism as confirmed by (Otieno et al., 2015).

In sound concrete, the reinforcing bars is protected by Iron (II) hydroxide or ferrous hydroxide ($2Fe(OH)_2$) which covers the reinforcing bars, forming a passive layer that significantly reduces the chances for further oxidation of reinforcing bars to insignificant levels. Furthermore, the alkaline environment of the concrete, where the pH is higher than twelve, prevents progression of reinforcing bars corrosion. The protection of the reinforcing bars can however be destroyed by the ingress of deleterious substances which are chlorides and carbon dioxide, leaving the reinforcing bars prone to corrosion (Portland Cement Association, 2002).

The service life of the reinforced structure is affected by reinforcing bars corrosion. The entire process of deterioration due to corrosion damage can be represented by the following three phases;

i. Initiation phase, which is the time taken by deleterious substances to cause depassivation of the reinforcing bars through the dissolution of the protective passive layer and there is no evidence of corrosion damage,

ii. Propagation phase, where corrosion leads to cracking due to formation of products that occupy more volume space than reinforcing bars, and

iii. Acceleration phase, where there is a rapid rate increase of corrosion because of the easy ingress of oxygen and moisture caused by the cracks formed in the propagation phase, resulting in widening of cracks and spalling.

In a bid to predict the propagation phase, Otieno et al., (2010) reported that modeling of the phase resulted in the development of several models.

A simplified diagrammatic representation of the three-phase corrosion model reproduced is as shown in Figure 1.

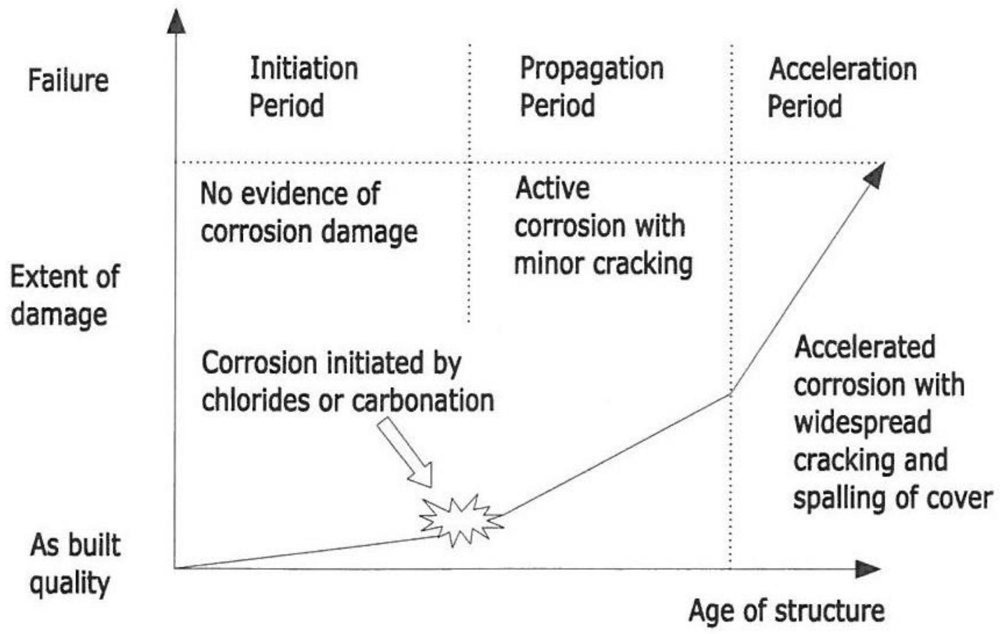

Figure 1: Three phase corrosion damage model (Beushausen & Alexander, 2009)

3.3.1.1 Mechanisms of corrosion

Corrosion is the oxidation of reinforcing bars consequently reducing the reinforcing bars cross-sectional area. This compromises the structural integrity of reinforced concrete element by reducing the carrying capacity of reinforcing bars. The by-products of corrosion occupy more volume resulting in the development of internal stresses in a bid to create more space. The pressure created at the concrete and reinforcing bars interface eventually exceeds the concrete's tensile capacity and that forces the concrete matrix to crack (Bhattacharjee &

Pradhan, 2010). The number of cracks increases as corrosion progresses (Andrade et al., 1993; Liu & Weyers, 1998). Deterioration rate is directly proportional to the increase in the number of cracks. When the number of cracks increases, delamination and spalling subsequently occur.

Oxidation is the electrochemical reaction which takes place in the presence of reinforcing bars, sufficient oxygen, adequate moisture and low pH environment. The reinforcing bar is the media of flow of electrons and concrete is a media of flow of ions. There is a loss of electrons (oxidation) by the reinforcing bars atoms at the anode and addition of electrons (reduction) at the cathode. Ultimately the ferrous ions move to the cathode through pore water in concrete and react with hydroxyl ions forming iron hydroxide (Bhattacharjee, 2013).

The half-cell reactions occur at both locations i.e. the cathode and anode. Below is an illustration;

i) Anodic reaction

$2Fe \longrightarrow 2Fe^{2+} + 4e^-$ (Oxidation process by loss of electrons)

ii) Cathodic reaction

$O_2 + 2H_2O + 4e^- \longrightarrow 4OH^-$ (reduction process by addition of electrons)

iii) The ultimate reaction is as follows

$2Fe^{2+} + 4OH^- \longrightarrow 2Fe(OH)_2$ (Iron (II) hydroxide or ferrous hydroxide ($2Fe(OH)_2$))

The sketch in Figure 2 shows the diagrammatic representation of the reactions that take place during corrosion.

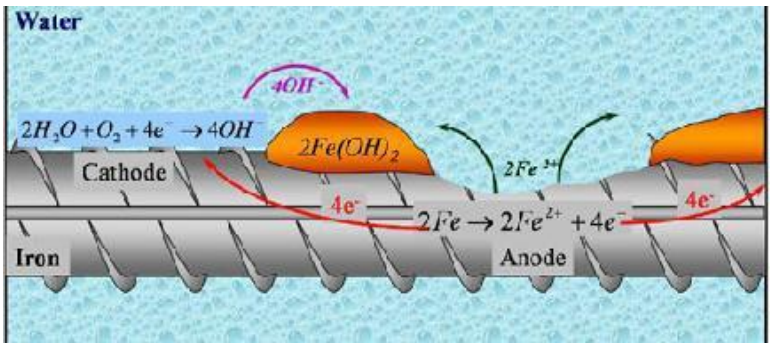

Figure 2: Reinforcing bars corrosion diagrammatic representation (Arito, 2014)

3.3.1.2 Factors influencing corrosion

Reinforcing bars corrosion is influenced by the ingress of deleterious substances. The deleterious substances include; chlorides that break down the passive layer around the reinforcing bars. The breaking down of the passive layer is called depassivation which consequently exposes the reinforcing bars. The exposed reinforcing bar is then prone to corrosion. Acidic carbonaceous gasses e.g carbon dioxide reacts with cement paste which is alkaline, resulting in alkalinity of concrete being neutralised.

It is important to note that penetrability of concrete matrix is linked to the ease with which fluids and ions move into the concrete microstructure, influencing movement of deleterious species. Deterioration mechanisms can be linked to penetrability of concrete as the ions and molecules freely move into the microstructure in the form of liquids and gases (Ballim et al., 2009).

Chloride-induced corrosion

MacDonald et al., (1991) found out that reinforcing bars corrosion as a result of ingress of chlorides is the most common deterioration mechanism. Similarly, Otieno et al., (2015) confirmed that the rate of deterioration is profound in coastal areas where there is an abundance of chlorides from the sea water.

Chlorides do not directly corrode the reinforcing bars, but, they act as a catalyst that can break down the passive layer with the protective film of iron oxide. The chlorides penetrate the concrete microstructure through the diffusion process and attack the passive layer. The diffusion of chlorides does not require pH reduction. Furthermore, the presence of sufficient chlorides around the reinforcing bars attack the passivating layer i.e. iron hydroxide to soluble metal chloride. Mackechnie and Alexander (2001) in their research found out that the chloride content threshold value is about 0.4% by mass of cement. They also explain that moisture and oxygen must also be available reinforcing bars corrosion to take place for.

Table 1 as researched by Mackechnie and Alexander (2001) indicates the chloride content values and the probability of corrosion.

Table 1: Qualitative risk of corrosion based on chloride levels (Mackechnie and Alexander, 2001)

Chloride content % by mass of cement	Probability of corrosion
< 0.4	Low
0.4 – 1.0	Moderate
> 1.0	High

Chlorides that influence corrosion are available from admixtures used during casting of concrete, ground water, sea water, and etcetera. Capillary absorption, permeation and diffusion are the main mechanisms that determine how chloride ions can penetrate concrete.

Diffusion is considered the principal method of transporting the chlorides to the reinforcing bars.

According to Kuosa et al., (2014), a typical profile of chloride concentration is exponential with the highest concentration near the surface and reduces towards the reinforcing bars.

Carbonation induced corrosion

The process of carbonation involves the reaction of calcium hydroxide which is present in the cement paste; with acidic carbonaceous gasses i.e., carbon dioxide from the atmosphere (Talakokula et al., 2016). The following illustrates the reaction equation;

$$Ca(OH)_2 + CO_2 \longrightarrow CaCO_3 + H_2O$$

Carbonation can occur in concrete whilst it is plastic or after it has hardened.

Although carbonation products i.e. calcium carbonate fill up concrete pores lowering permeability, carbonation in the hardened state reduces the pH value of concrete pore structure, increasing probability of reinforcing bars corrosion that may lead to significant loss of structural serviceability (Talakokula et al., 2016).

The sketch illustrating the movement of the carbonation front in concrete is as shown in Figure 3.

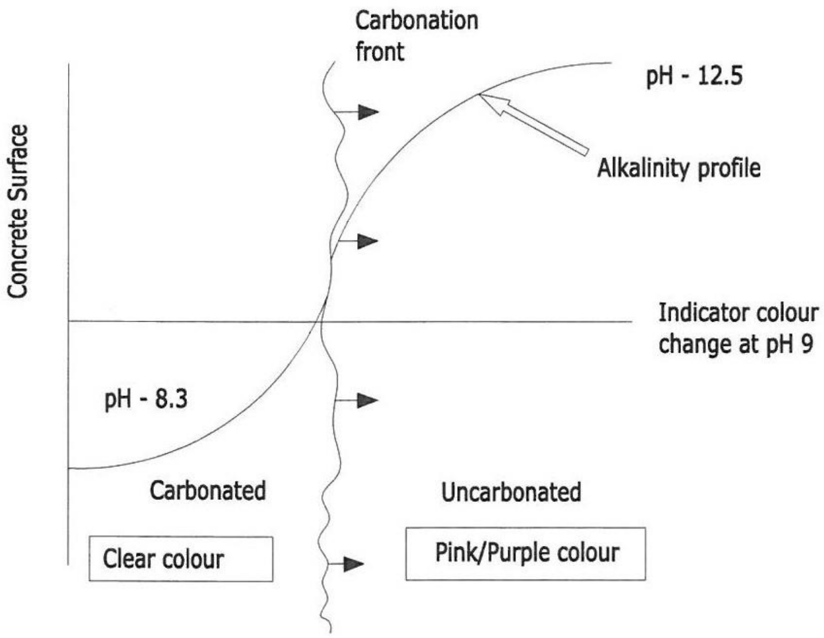

Figure 3: Carbonation Front movement (Beushausen, 2014)

The carbonation process can penetrate the pores of concrete matrix. Carbonation starts at the surface of the reinforced concrete structure and moves towards the reinforcing bars embedded in the structure. Carbonation moves as a distinct front from exposed ends of the concrete surface towards the interior. The clear colours depicted in Figure 3 refers to the results of the widely used method of carbonation testing where phenolphthalein indicator solution is sprayed onto a core taken from the concrete structure. The carbonated section doesn't show any change in colour and it remains clear. The uncarbonated section changes its colour to purple or pink. The use of phenolphthalein indicator solution is discussed in more detail in Section 2.2.2.2.

To predict carbonation depth, several researchers developed various mathematical models. Ashraf (2016), Yoon et al., (2007), DuraCrete (1998) and Zhang (2016) concur to the principal that the depth of the front is proportional to the square root of the exposure time. The principle was originally developed by Meyer et al., (1967) and the equation is as illustrated below;

"X_c (t) = A $t^{1/2}$ where

X_c = carbonation depth after time t

t = carbonation exposure duration

A = empirical constant", Meyer et al. (1967).

The rate of carbonation depends on the presence of pore water, grade of concrete, the permeability of concrete, coated or uncoated concrete, cover depth and time.

It is critical to note that the percentage of CO_2 present in the air varies from one area to the other. The concentration of CO_2 may be about 0.03% by volume in the countryside, and could be in the region of 0.3% to 1.0% in industrial areas hence very high probability in the industrial zones (Zhang, 2016).

Carbonation is a slow process of which the carbonation front can proceed at an annual rate of up to one millimetre. Furthermore, the highest rate of carbonation has been found to take place when relative humidity is in the range of between 50% and 70% (Beushausen et al., 2015). When the relative humidity is higher, the pores in the concrete are usually filled with water, consequently restricting the ease when carbon dioxide penetrates the concrete, hence reduction in carbonation rate. According to Stewart et al., (2011), quality, relative humidity, cover and ambient carbon dioxide concentration are the main factors that influence carbonation.

Examples of the defects induced by carbonation are as depicted in Figure 4 and 5.

Figure 4: Carbonation Induced Corrosion (Portland Cement Association, 2002)

Figure 5: Spalling due to Carbonation induced corrosion (Draft TMH 19, 2013)

2.1.2 ASR and associated problems

As researched by Islam and Ghafoori (2013), ASR was discovered in 1940 and has been identified as one of the deterioration mechanism that has caused a major concern. The reaction between the reactive silica present in some aggregates and alkalis i.e. Na_2O and K_2O in the cement paste is known as ASR which subsequently results in the formation of an alkali-silicate gel. Sims and Nixon (2003) confirmed the silica gel has a high affinity for water molecules. Osmosis is the transport mechanism that results in water molecules being absorbed from the environment. Furthermore, the water absorbed by the gel comes from the cement paste and the ultimate result is the volumetric increase. The volumetric increase results in pressure build up leading to internal stresses and the ultimate result is the cracking of the concrete if unrestrained, due to hydraulic pressure (Sims & Nixon, 2003). Cracking may be aggravated by the constant supply of moisture.

ASR takes a long time to cause damage to the reinforced concrete structure as it is a very slow process. However, the time it takes to visibly see large cracks depends on whether the aggregates used during construction are fast or latent reactive. Cracks due to ASR follow the path of least resistance i.e. parallel to stress flow on the structural members under significant stress (Karthik et al., 2016). The cracks may cause serviceability issues (Karthik et al., 2016). Karthik et al., (2016) observed from field and laboratory experiments that the first cracks further provide a path for moisture, which results in the acceleration of formation of cracks by ASR. The cracks also expose the reinforcing bars to corrosion which has been discussed in Section 2.1.1. Some researchers have suggested that the ASR gel may have some protective effects on the reinforcing bars (Ueda et al., 2013).

Map cracking is typical of deterioration due to ASR and examples of structures under ASR attack are shown in Figure 6 and 7.

Figure 6: Typical ASR visible defects, draft TMH 19 (COTO, 2013)

Figure 7: Typical ASR damage, draft TMH 19 (COTO, 2013)

Figures 6 and 7 clearly show that aesthetics has been negatively affected and the structural integrity could have been severely compromised.

It is important to mention that the mitigation measures of defects due to ASR are very costly, as no promising repair solution has been established to date, although lithium compounds have been found to suppress the reaction (Ueda et al., 2012).

According to Oberholster (2009) the following have been found to significantly reduce the effects of ASR;

- Use of non-reactive aggregates,
- Reduce the cement content consequently limiting the alkali content and,
- Use of cement extenders.

2.1.3 Shrinkage and associated problems

Restrained contraction in concrete causes cracks to occur. The presence of cracks accelerates ingress of deleterious substances resulting in durability problems, which have negative effects to the service lives of the concrete structures (Fu et al., 2016).

Shrinkage occurs due to the hydration process and the loss of water due to evaporation (Mora-Ruacho et al., 2008). The hydration products have volume which is less than that of the un-hydrated cement combined with water. Shrinkage cracks occur before concrete hardens or after hardening and it has been researched that shrinkage cannot be fully reversible due to the formation of additional bonds (Mora-Ruacho et al., 2008). The source of shrinkage is the cement paste and there are different types of shrinkage cracks. Below are different types considered in this research which are relevant to the structures visually assessed:

- Plastic shrinkage,
- Drying shrinkage,
- Autogenous shrinkage, and
- Carbonation shrinkage.

2.1.3.1 Plastic shrinkage

Examples of typical plastic shrinkage cracks are as depicted in Figure 8.

Figure 8: Typical Plastic shrinkage cracks (Portland Cement Association, 2002)

Plastic shrinkage cracks occur before hardening i.e. in the plastic stage when there is rapid surface moisture loss due to evaporation and commonly takes place in the first four hours after casting. Evaporation of the moisture results in the formation of water menisci and the subsequent contraction forces in the concrete microstructure (Mora-Ruacho et al., 2008). The contraction is often accompanied by random surface cracking and the effects are high when the weather conditions are hot, low relative humidity and windy (Mora-Ruacho et al., 2008). It is important that there is no definite pattern for plastic shrinkage cracks.

2.1.3.2 Drying shrinkage

The loss of capillary water from the hardened concrete results in drying shrinkage i.e. contracting of hardened concrete. Water not consumed by hydration process is the source of capillary water. Addis (2008) adds that, loss in moisture once adequate curing stops promotes drying shrinkage.

Capillary tension results from the increase in the curvature of the menisci as water is drawn out, resulting in shrinkage which is the reduction in the volume of C-S-H (Calcium Silicate Hydrate). Restrained shrinkage then causes an increase in tensile stress, which could lead to cracking. Zhang et al., (2013) confirmed that the loss of free water and absorbed water forces concrete to shrink due to tensile stresses created. The ultimate result is cracking that can have a direct effect on the structural performance. Durability and serviceability must be investigated and considered in the design stage. It is worth mentioning that the degree of shrinkage cracking is a result of many factors including; aggregate type, cement type, etcetera which are discussed elsewhere.

Drying shrinkage is more prominent than other shrinkages in conventional concrete and is irreversible due to the formation of additional bond in the cement gel when adsorbed water has been removed.

When the structure is restrained, concrete cracks as shown in Figure 9.

Figure 9: Typical restrained Drying Shrinkage Cracks (Portland Cement Association, 2002)

The drying shrinkage crack width normally ranges from 0.3 millimetres to 0.5 millimetres per meter length of the concrete element (Addis, 2008).

2.1.3.3 Autogenous shrinkage

Autogenous shrinkage occurs under constant temperature whereby microscopic reduction of the length of concrete occurs. The reduction in length occurs when there is insignificant loss or absorption of moisture into the matrix. Shrinkage occurs in sealed specimen due to hydration and self-desiccation.

According to Li and Li (2014) it has been found that two main reasons that influence autogenous shrinkage are;

1) Low water to binder (W/B) ratio under 0.42 - all the water is consumed by the hydration process which may result in surface tension within the capillaries. The fine capillaries are formed due to the demand of additional water for the hydration process.

2) A significant number of active mineral admixtures as they augment pores refinement. The result is an increase in capillary tension that may lead to increased shrinkage.

Qin et al., (2017) also reveal that temperature changes influence autogenous shrinkage and the likely reason is that the microstructure evolution during hydration process and the apparent activation energy has been found to be influenced by temperature.

2.1.3.4 Carbonation shrinkage

The by-products of carbonation which includes calcium carbonate ($CaCO_3$), occupy less volume than the reacting products and that may cause cracks.

2.1.4 Abrasion and associated problems

Abrasion is rampant in windy areas, river flows, on concrete floors and pavements. The damage due to abrasion occurs when the concrete surface cannot resist the frictional forces resulting in loss of outer cement paste. The aggregates will then be exposed and consequently loss of aggregates as the process continues. The result is the cover reduction or exposure of reinforcing bars in a reinforced structure, increasing the probability of reinforcing bars corrosion and reduced structural capacity in extreme cases. Figure 10 shows an example of abrasion on a road slab.

Figure 10: Typical Abrasion damage, draft TMH 19 (COTO, 2013)

2.1.5 Chemical attack and associated problems

Concrete deteriorates when exposed to aggressive chemicals in the presence of moisture and thus durability is affected. The primary chemical attack mechanisms are sulphate, sea water, and acid attack. Below is the discussion on the principles of sulphate attack.

2.1.5.1 Sulphate attack

Sulphates are absorbed into the concrete pore structure and consequently react with hydration products which include; calcium hydroxide ($Ca(OH)_2$) and tricalcium aluminate (C_3A). The reaction results in the formation of gypsum, and then ettringite which is an expansive product that occupies more volume than the reactants. Furthermore, sulphate attack results in the alteration of C-S-H consequently destroying the microstructure of concrete.

Sulphates can be present in the groundwater, sea water, waste-water effluent, and etcetera. The cautions usually associated with the sulphates are Na^{2+} (Sodium), K^{2+} (Potassium) and Mg^{2+} (Magnesium).

It is generally accepted that the chemical reaction is initiated when sulphates react with $Ca(OH)_2$ forming $CaSO_4$ and hydroxides i.e. Magnesium hydroxide ($Mg(OH)_2$), Sodium hydroxide ($Na(OH)_2$) or Potassium hydroxide ($K(OH)_2$). According to the research conducted by Ballim, et al., (2011) C-S-H is not stable in $Mg(OH)_2$, hence the effects of Magnesium sulphates are more severe. The C-S-H decomposes forming Mg-S-H which has no binding characteristics, a process known as decalcification, resulting in the disintegration of the cement paste. $Mg(OH)_2$ is also known as brucite.

The chemical reactions can be represented as shown below (Ballim, et al., 2011);

$$Ca\,(OH)_2 + SO_4^{2-} \longrightarrow \underset{\text{(Gypsum)}}{CaSO_4.2H_2O + 20H(aq)}$$

$$3CaSO_4 + 3CaO.Al_2O_3.6H_2O + 25H_2O \longrightarrow \underset{\text{(Ettringite)}}{3CaO.AlO_3.3CaSO_4.31H_2O}$$

It is imperative to note that gypsum and ettringite are relatively insoluble in water, but, they are more soluble in chlorides ions solutions and this implies that deterioration of concrete in such environments is not because of expansion forces.

C_3AH13+3SC- ----------------- C_3A.3CS.31H +CH

Evidence of sulphate attack

- White crystals of gypsum, cracking and spalling,
- White powder of gypsum together with powder formation on scratching is symptoms of sulphate attack.

Sulphate attack can be reduced using low w/c ratio to reduce penetration of sulphate into the concrete. The use of low C_3A cement and blended cement also improves sulphate resistance.

2.2 Concrete condition assessment

Concrete deteriorates from the day it is cast, through to the end of its service life. As concrete deteriorates, there are visible external defects that can be linked to the deterioration mechanism(s), and there are defects that cannot be identified by visual assessment. Concrete condition assessment is done to identify the defects in the structure so that proactive or reactive maintenance strategy may be implemented. The background visual inspection on the structure involves identifying the visible external concrete distresses as well as corroding reinforcement. It also involves identifying the prevailing environmental conditions that surround the concrete structure and influence the deterioration mechanisms.

Some visible defects however, need additional tests to ascertain the deterioration mechanisms where the specialised equipment can be employed. The recommended full-scale condition assessment is established during the visual assessment whereby the degree of deterioration motivates the need for further investigations. There are destructive and non-destructive testing methods available. The testing methods can be implemented to increase the acceptance of the assessment outcomes with a higher-level degree of confidence. A selected number of useful tests which include; visual assessments, destructive testing and non-destructive testing used to ascertain the deterioration mechanisms for the observed defects are discussed below.

2.2.1 Visual assessments

Surface defects i.e. but not limited to the following; impact (mechanical damage), cracking, crazing, rust staining, delamination, abrasion, and leaching are observed during visual assessments. The assumptions on the deterioration mechanism are derived from the visible defects and the actual causes can only be concluded when the full-scale assessment is done. Most of the deterioration mechanisms result in cracking of concrete, however, careful analysis of cracks pattern is envisaged to determine the causes. Conclusions on the causes of cracking may however not be based on the visual assessment in isolation.

The defects that can be visually observed and linked to each respective mechanism are discussed below under the relevant deterioration mechanism.

2.2.1.1 Reinforcing bars corrosion

Based on the principle that reinforcing bars corrosion causes cracking, delamination and spalling as researched by Matthew and Banville (2008), one can link the defects to reinforcing bars corrosion depending on the pattern of the cracks.

The sketches shown in Figure 11 are a typical simplified representation of the common failure mechanisms that can be visible depending on the stage the reinforcing bars corrosion is at (Matthew & Banville, 2008).

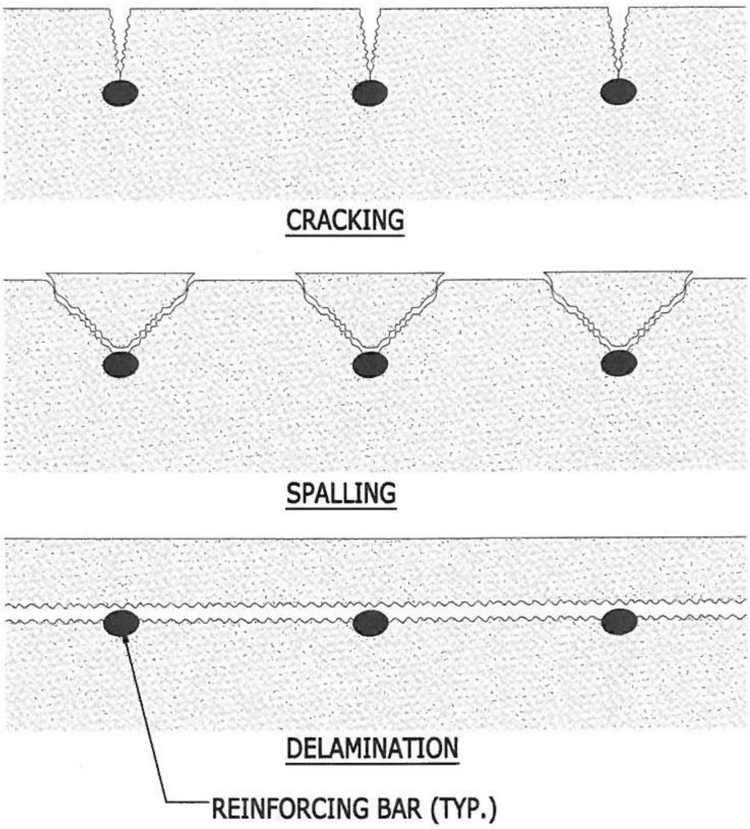

Figure 11: Common reinforcing bars corrosion failure mechanisms (Matthew & Banville, 2008)

Although most of the deterioration mechanisms discussed cause cracking, the pattern of the visible cracks allows the assessor to come up with conclusions on the probable deterioration mechanisms. For example, generally, surface cracks due to reinforcing bars corrosion are usually parallel to steel bars (El-Reedy, 2007). Rust staining also provides proof that reinforcing bars corrosion has taken place.

2.2.1.2 ASR

With the aid of the discussion in Section 2.1.2, if the cracks pattern as depicted in Figure 6 and Figure 12 is observed and the width of cracks are very wide, the possibility that ASR is the deterioaration mechanism is very high. However, conclusions based on crack pattern only may be misleading and it is highly recommended that petrographic analysis has to be conducted to ascertain ASR damage. Crack width monitoring is also required to determine whether ASR is active or not. The cracks due to ASR are expected to continuously widen if the harsh environment conditions prevail. ASR may lead to the loss of structural integrity which consequently may lead to total collapse of the concrete structure.

Figure 12: Typical example of ASR damage, draft TMH 19 (COTO, 2013)

2.2.1.3 Leaching

Leaching can be linked to dissolution of the hydroxide ions as researched by Rozière et al., (2009). Rozière et al., (2009) also found out that various mechanisms cause leaching and can be linked to dissolution of calcium out of the concrete matrix. The visible defect is the efflorescence of the reaction products. Efflorescence can be linked to the chemical attack i.e. ingress of chloride, sulphate, magnesium, etcetera. When acid water or poorly mineralised water is absorbed with concrete, the white powder on the concrete surface if observed may be linked to leaching (Rozière et al., 2009). It is important to note that leaching is a diffusion-reaction phenomenon. Figure 13 shows a typical example of leaching.

It is imperative to highlight that lime leaching occurs but is usually harmless. However, leaching due to dissolution of reaction products results in increased porosity of concrete. Furthermore, leaching of corrosion products i.e. rust indicate severe deterioration as it confirms that corrosion is at a propagation period. Refer to Section 2.1.1 for more clarity on corrosion.

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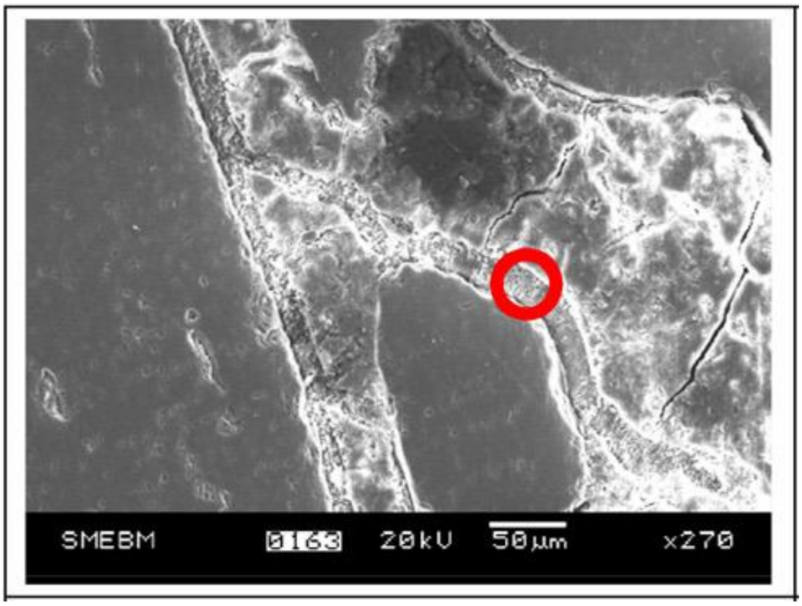


Figure 13: Lime leaching (Rozière et. Al.,2009

2.2.1.4 Abrasion

As alluded to in Section 2.1.6, the loss of aggregates due to frictional forces on the surface is visible when visual assessments are done. Figure 14 is a typical example which indicates that abrasion has taken place. Another example picture extracted from the visual assessments reports done is as shown in Figure 14.

Figure 14: Abrasion damage (Takaindisa, 2015)

2.2.1.5 Impact

Mechanical damage is when the concrete element is exposed to mechanical impact by an external force which results in some spalling of concrete. Typical deterioration due to mechanical impact is shown in Figure 15.

Figure 15: Impact Damage, draft TMH 19 (COTO, 2013)

2.2.2 Destructive testing methods

Inorder to complement the visual assessment outcomes, deterioration mechanisms can be assertained when further investigations are conducted. There are several destructive testing methods that can be conducted on an existing structure to determine the deterioration mechanisms. Careful selection of the appropriate tests by experienced engineers is always recommended. There are several destructive testing methods available and are generally divided into two categories i.e. in-situ testing and laboratory testing methods. The descriptions and the intended outcomes of the destructive testing methods are explained below

2.2.2.1 In-situ testing

Half-Cell Potential Test (HCP) is an in-situ testing method that can be employed on an existing structure. As shown in Section 2.1.1.1, corrosion is an electrochemical process. According to Rendell et al., (2002), cathodic and anodic half-cell reactions occur on the embedded reinforcing bars. Hydroxyl ions are formed from the cathodic half-cell reaction and iron cations from the anodic half-cell reaction. Monetemor et al., (2003) confirmed that corrosion current is generated when the cathodic and anodic reactions are not balanced, which enables the measurements of the reinforcing bars potential relative to the reference half-cell. The reference half-cells i.e. copper/copper sulphate or silver/silver chloride are generally used by placing them on the concrete surface when measuring the embedded reinforcing bars potential. The potential readings depend on the type of reference half-cell used. Concrete cover may be removed to connect to the reinforcing bars as confirmed by (Bungey et al., 2006).

Table 2 gives an indication of the risk of reinforcing bars corrosion in the reinforced concrete structure for different electrode solutions. It is important to note that the readings are not quantitative.

Table 2: HCP readings interpretation as specified in ASTM C876-91

Reinforcing bars potential (mV)		**Qualitative risk of corrosion/likely corrosion condition**
$Cu/CuSO_4$	**Ag/AgCl**	
> -200	> - 106	Low (10% risk of corrosion)
-200 to -350	- 106 to - 256	Intermediate corrosion risk (uncertain)
< - 350	< - 256	High (> 90% risk of corrosion)
< - 500	< - 406	< - 500 < - 406 Severe corrosion

In order to carry out the testing, the following need to be established:

- Locate the connection point to the reinforcing bars and remove the cover at the proposed location,
- There should be the continuity of electrical conductivity of the reinforcing bars

As a rule, a more negative reading of potential results in higher probability of corrosion (Ping & Beaudoin, 1998).

Advantages and limitations of Half-Cell Potential are tabulated in Table 3.

Table 3: Advantages and limitations of Half-Cell Potential (HCP)

Test	Advantages	Limitations
Half-Cell Potential Test (HCP)	– Method is simple – "Iso-potential contour map" can be generated (Bungey et al., 2006). – The risk of local corrosion can be identified (Bungey et al., 2006).	– Only the probability is established and not actual corrosion rate. – It is destructive in an endeavour to ensure electrical contact with embedded reinforcing bars. – Thorough surface preparation may be required (Bin Ibrahim et al., 2002). – Moisture conditions influence the readings, which entails that results are only valid for the time of testing. Tests done at the same point, but at varying time intervals may differ drastically.

2.2.2.2 Laboratory testing methods

It is necessary to assess the possibility of corrosion in regions with no visible signs of deterioration. Chloride levels and carbonation depth measurements can be determined and extrapolated from the chloride profiling graphs and carbonation profiles respectively to estimate the future levels. The estimated future levels of the deleterious substances can be used to determine the estimated remaining service life using diffusion theories. In the case of reinforcing bars corrosion, the test results can indicate whether it is chloride induced or carbonation induced.

Sampling can be conducted by taking cores from the concrete structure and various tests conducted in the laboratory. Below is a brief discussion on selected laboratory test methods.

Carbonation depth measurement

Sampling can be conducted by taking cores from the existing reinforced concrete structure and ensuring the risk of measuring the carbonated sample that occurred after sampling is minimised.

Phenolphthalein indicator solution is used and carbonation depth is measured by spraying cores with the solution. Phenolphthalein turns pink or purple where the concrete is highly alkaline and does not change colour, but remains clear where concrete is carbonated. The distance from the surface to the reinforcing bars can easily be measured and this will give an indication of the corrosion probability. It is important to note that phenolphthalein indicator solution is composed of 1% phenolphthalein by mass in ethanol or water solution as researched by (Mackechnie & Alexander, 2001).

Although the use of phenolphthalein indicator solution has a limitation e.g. corrosion is underestimated as depassivation occurring at pH+/-10.5 (Mackechnie & Alexander, 2001) yet phenolphthalein only changes colour at pH9, it provides useful results for reinforcing bars corrosion rating.

The example of the cores taken, and phenolphthalein indicator sprayed is as shown in Figure 16. The typical photographs depict that carbonation has taken place from both ends of the cores. The middle sections where phenolphthalein changed colour to purple are still not yet carbonated.

Figure 16: Carbonation depth measurement (Arito.,2014)

Advantages and limitations of using Phenolphthalein indicator solution for carbonation depth testing are tabulated in Table 4.

Table 4: Advantages and limitations of using Phenolphthalein indicator solution

Test	Advantages	Limitations
Phenolphthalein indicator solution	– The distance from the surface and to the reinforcing bars can easily be measured and this will give an indication of the corrosion probability, hence the useful information provides useful results for reinforcing bars corrosion rating	– Corrosion is underestimated as depassivation occurring at pH+/- 10.5 (Mackechnie & Alexander, 2001) yet phenolphthalein only changes colour at pH9.

Chloride content

Chlorides testing samples are extracted from the reinforced concrete structures in the form of cores or drilled powder samples (using diamond drill bits) at precise depth increments from the surface of the concrete cover. The samples are dipped and thoroughly mixed with concentrated nitric acid to release chlorides. Potentiometric titration is then used to analyze the concentration of chloride ion. The chloride content is expressed as a percentage by mass of cement.

When the actual test and profiling is done, the chlorides concentration at any depth may then be interpolated from the chloride profiling graph. The probability of corrosion can be categorised into the ratings indicated in Table 1 under Section 2.1.1. It is important to note that the research on the threshold values is on-going.

2.2.3 Non-destructive test methods

Several non-destructive test methods can be done in the field to complement the visual assessments in establishing the deterioration mechanisms. Below is a discussion on selected test methods.

Corrosion rate measurement

The most reliable method for measuring actual corrosion activity is the corrosion rate measurement method. Galvanostatic linear polarisation resistance is normally used in the field, but is a time-consuming process as it requires adequate planning and mapping out of test points before testing. The test points need to be systematically recorded.

Table 5 reproduced from RILEM TC-154-EMC, (2004) gives an indication of the corrosion rate values likely to be obtained in the field and their interpretation. An example is the Gecor, a widely-used tool.

Table 5: Interpretation of corrosion rate readings (RILEM TC 154 - EMC, 2004)

Corrosion rate (μA/cm2)	Qualitative assessment of corrosion rate
> 1.00	High
0.5 – 1.0	Moderate
0.1 – 0.5	Low
< 0.1	Passive

The test is very sensitive to the relative humidity and the air temperature, hence the recommendation to do the resistivity test as a complementary test to ascertain the corrosion rate test results.

Cover depth measurement

Insufficient cover has been observed to have a direct influence in the deterioration of reinforced concrete structures in South Africa (Ballim et al., 2009). The covermeters however can detect the depth of reinforcing bars in the reinforced concrete structure which translates to the cover depth of the existing structures (Ballim et al., 2009).

The Ground Penetrating Radar (GPR) is the non-destructive test used to locate the reinforcing bars embedded in the concrete, hence mapping of reinforcement. It is also used to estimate the concrete cover to reinforcement depth. Also, the equipment can determine the thickness of concrete slabs. Another important application of GPR is to detect voids in the concrete (Maierhofer, 2003). Because of all mentioned applications of the GPR it has been realised that the instrument is suitable for gathering essential information during preliminary study.

Electromagnetic phenomena are principles used in covermeters. The phenomena enable the determination of the reinforcing bar diameter and its location. According to Bungey et al., (2006), the concealed reinforcing bars interacts with the magnetic field generated from the electric current in the excitation coil. The interaction is caused by magnetic induction effect for low-frequency covermeters and eddy current effect for high-frequency covermeters. The degree of interaction is directly proportional to the bar size and cover depth. It increases with increasing bar size and decreases with increasing cover depth. Rendell et al., (2002) stated in his research that the signal strength can be linked to cover depth and absolute figure can be calculated if the covermeter is correctly calibrated.

The researched advantages and limitations of GPR are as outlined in Table 6.

Table 6: Advantages and limitations of using GPR

Testing	Advantages	Limitations
GPR	– It provides data in real-time, so it's fast to get the as-built data. – The GPR is portable.	– It is expensive (Bungey, 2004), – Requires an expert to interpret the results of which the resources are not always available.

Concrete strength

The Schmidt rebound hammer is the common tool used to measure the concrete surface hardness. The empirical correlations have been established to enable the tool to be used for the determination of concrete strength (Bungey et al., 2006; Bin Ibrahim et al., 2002). However, the correlations should be done within limits. The measurements of the surface hardness are determined from the rebound distance which is linked to mechanical energy. A hard surface absorbs less mechanical energy and ultimately the rebound distance is expected to be excessive.

The Bungey et al., (2006) listed the four applications which are as summarised below:

- Variations in the quality of concrete matrix can be checked,
- A specific requirement can be compared with the concrete sample,

- Empirical correlations of surface hardness to strength can be used to estimate concrete strength,
- Abrasion resistance is proportional to surface hardness; hence abrasion resistance can be classified.

Table 7: Advantages and limitations of rebound hammer test

Test	Advantages	Limitations
Schmidt rebound hammer	– Instant results as they are recorded in-situ – is simple and inexpensive to conduct	– The rebound hammer is affected by the texture of concrete section tested, for example, if applied directly where a large aggregate is underneath; the results are extremely high which might not be an accurate reflection of the strength of the concrete structure. Also, if the test is done at the edge of the member, the reading of the strength is lower than what it should be.

Resistivity measurements

Resistivity is directly linked to concrete quality. The greater the quality, the higher the resistivity and the lower the corrosion rate (Rendell et al., 2002). The electrolytic resistivity is used to determine the ease with which corrosion current flows through the matrix. It is also a measure of pore water. The concrete matrix is the electrolyte when the resistivity tests are conducted (Bungey et al., 2006). It is important to note that concrete quality and porosity are inversely proportional hence resistivity increases as w/c ratio decreases.

Wenner four-probe is used to measure resistivity. The equipment has four equally spaced electrodes in a straight line. When the Wenner probe is placed on the surface, Bin Ibrahim et al., (2002) found out that “an alternating low-frequency current is passed between the outer two electrodes while the voltage drop between the inner electrodes is measured”.

Wenner four-probe is easy to use however, according to Bungey et al., (2006), there is need to ensure practical considerations are accounted for before interpreting the recorded results. A list of the practical aspects to be considered is shown below:

- Surface carbonation results in the hard carbonation skin. The hard skin in turn results in significant overestimation of the resistivity,
- Measurements taken near the edge of the concrete element have been found to overestimate resistivity. Similarly, measurements taken on small members have the same effect,

- There is a high possibility of underestimating the resistivity in cases where reinforcing bar is near a measurement,

- The measurements vary with changing weather conditions.

The size of the Wenner probe is reasonable and portable. Furthermore, Wenner probe is straight forward to operate. However, reinforcing bar conducts electricity thereby affecting the readings when present in the test vicinity (Bungey et al., 2006). Furthermore, it is recommended that to increase the acceptance of the results with more confidence, readings should be complemented by other measurements obtained from other techniques (Rendell et al., 2002).

Table 8 outlines the probable corrosion rates that are derived from the expected resistivity rate results measurements.

Table 8: Probable corrosion rate based on resistivity (Mackechnie & Alexander, 2001)

Resistivity rate (kΩ-cm)	Probable corrosion rate
<12 (low resistivity)	High
12-20	Moderate
>20 (high resistivity)	Low

From Table 8, it can be noted that concrete with low resistivity of less than 12kΩ-cm is of poor quality and sound quality has high resistivity; greater than 20kΩ-cm.

Petrographic testing

Cores are taken from the section identified as ASR being a possible cause of cracking and if ASR is the cause, there will be visual signs on the cores taken. Cracks around the aggregates are easily identified. The petrographic analysis may also be done to determine the reactivity of the aggregates. Petrographic testing involves the use of microscopes to examine material samples. The mineralogical characteristics of the rock can be determined as well as the chemical characteristics of concrete. The active standard used to carry out petrographic testing is ASTM C295 / C295M.

2.3 Exposure classes

EN206-1:2000 is a European Standard which was prepared by the Technical Committee CEN/TC 104 in an endeavour to come up with relevant exposure classes representing the environmental exposure conditions of concrete structures.

There are six exposure symbols and abbreviations used in the EN206-1:2000 but, only the following three were considered relevant to the structures assessed and these are shown below;

XC : risk of corrosion induced by carbonation

XS : risk of corrosion induced by chlorides from sea water

XA : chemical attack

XC, XS and XA are referred to as exposure classes. Table 9 is an extract of the exposure classes relevant to the sphere of study for this dissertation, extracted from EN206-1 (2000) pages 15 and16.

Table 9: Exposure related to environmental actions (EN206-1, 2000)

Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity. Concrete permanently submerged in water.
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact, for example, many foundations.
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity. External concrete sheltered from rain.
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2.
Corrosion induced by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast.
XS2	Permanently submerged	Parts of marine structures.
XS3	Tidal, splash and spray zones	Parts of marine structures.

2.4 DER-U Defect rating system

The DER-U rating system has been specifically developed for bridge structures for the rating of defects observed during the visual assessments.

The defect rating system has been adopted by COTO, (2013) in an endeavour to standardise the rating of defects during the visual assessment, of which is detailed in the visual assessment guide i.e. the Draft TMH 19 series, Manual for the Visual Assessment of Road Structures (2013). The Draft TMH 19, (2013) provides a benchmark for defects rating of which the DER-U rating system has been adopted. It is important to note that the rating enables the road authorities to come up with condition indices which in turn assist in compiling maintenance priority lists of the road structures, hence mandatory for inspectors to apply the same principles when doing visual assessments.

Furthermore, COTO developed the formulae to calculate the several indices and that includes the inspection priority indices (Ip) which has been adopted in this research to determine the severity of defects observed. The relevant TMH series is the Draft TMH 22, Road Asset Management Manual.

DER-U is a defect rating system whereby the degree, extent, relevancy and urgency ratings of the defect are rated during the visual assessments. The meaning of degree, extent, relevancy and urgency are as follows;

2.4.1 Degree rating

Degree rating (D) is the visual rating that indicates how severe the defect is. The degree ranges from zero to four, and the recommended rating is as shown in Table 10.

Table 10: D rating, draft TMH 19 (COTO, 2013)

		D – DEGREE				
X	U	No visible defects	Minor	Moderate	Warning	Severe
Not applicable	Unable to inspect	0	1	2	3	4

TMH 19 outlines different guidelines used to rate the defect and an example is the use of crack widths which is detailed below;

Crack width less than 0.3 mm with no signs of water leakage or corrosion of reinforcement is considered minor. Crack width greater than 0.3 mm but smaller or equal to 0.6 mm with no signs of water leakage or corrosion of reinforcement is considered fair. Furthermore, crack of

0.6 mm with signs of water passing through crack and evidence of corrosion of reinforcing bar is rated as poor and finally, crack greater than 0.6 mm is considered severe.

Typical defects correlated to the recommended ratings for degree rating are shown in Figure 17, which were extracted from the Draft TMH 19.

Wing/Retaining Wall Defect Photos		
D=2 Left DER-U 211-2 Right DER-U 211-3		
	Diagonal crack in wing wall	Spalling in wing wall
D=3 Left DER-U 312-3 Right DER-U 312-3		
	Diagonal crack in wing wall	Spalling at wing wall joint
D=4 Left DER-U 414-4 Right DER-U 442-3		
	Restraint crack in wing wall	AAR cracking in wing wall

Figure 17: Illustration of D rating, draft TMH 19 (COTO, 2013)

2.4.2 Extent rating

Extent (E) is the rating on how widespread the defect is on the item being inspected. The illustration of the rating is shown in Table 11.

Table 11: E rating, draft TMH 19 (COTO, 2013)

E - EXTENT			
Local	More than local	Less than general	General
1	2	3	4

Typical illustrations on the principle of extent rating employed by the inspector during visual assessment are shown in Figure 18, extracted from the Draft TMH 19, (2013).

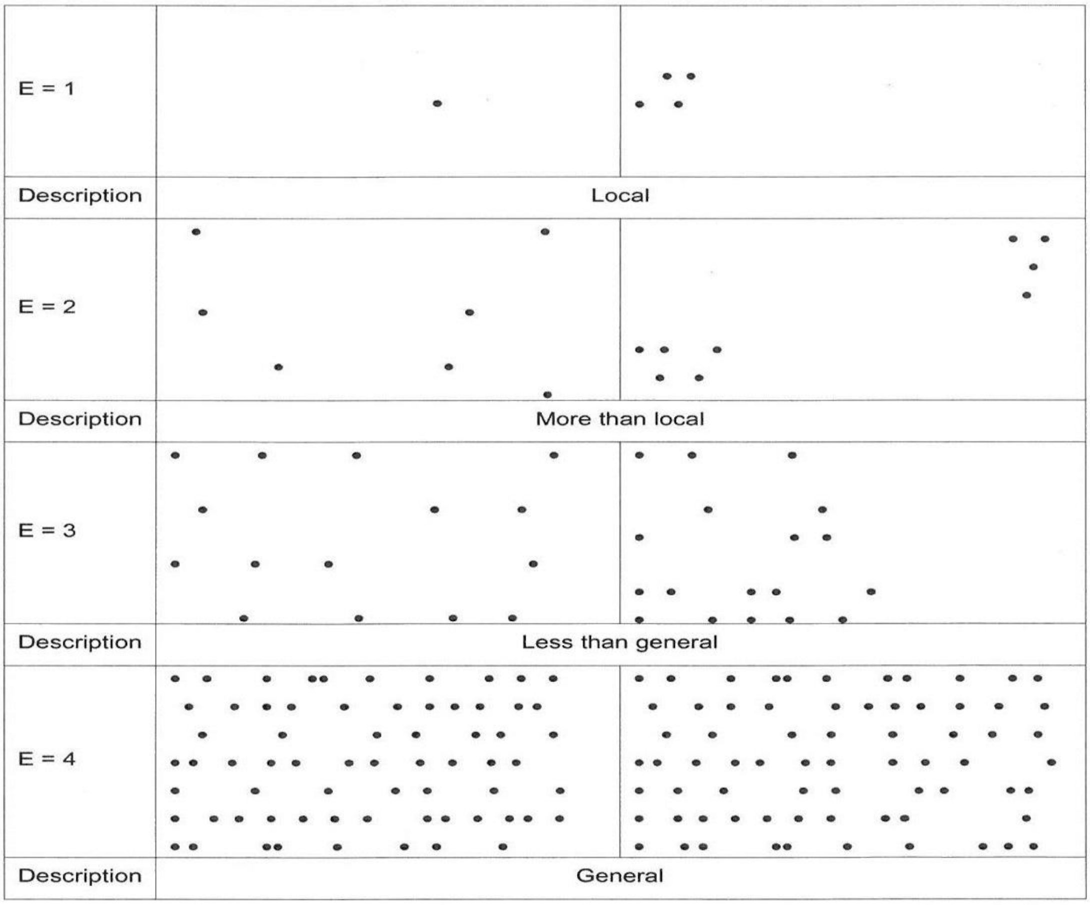


Figure 18: illustration of Extent rating

Another example outlined in the TMH 19 is the specific reference to spalling which is described as follows;

When spalling is shallow, and reinforcement is not visible, the defect is considered minor. When spalling is shallow, reinforcement partly exposed and there are minor signs of corrosion, the defect is considered fair. However, when the reinforcement is partially or fully exposed and corrosion is a problem, the defect is rated as poor. The severe degree rating is applicable to a defect when reinforcement is exposed and significantly corroded, prestress duct is exposed and when there is section loss.

2.4.3 Relevancy rating

Relevancy (R) is a rating of the consequence of the defect with regards to the structural or functional integrity.

Table 12: R rating

R - RELEVANCY			
Min	Moderate	Major	Critical
1	2	3	4

2.4.4 Urgency rating

Urgency (U) rating gives the direct time limits to do the repairs considering the present and future environmental conditions as well as events that may adversely affect the observed defects.

The values used for U rating are given in the Table 13.

Table 13 : U rating

U - URGENCY					
Record purposes only	Monitor only	Routine	<10yrs	<5yrs	ASAP
R	0	1	2	3	4

2.4.5 Inspection Item priority index (Ip)

According to Draft TMH 22, COTO (2013), the rating of structures is very complex. The development of Structure Priority Condition Indices (SPCI) assists in identifying structures with critical defects. SPCI is calculated using the inspection of sub-item priority index (Ip) values detailed in TMH 22. However, in this research, the focus is only on the inspection item priority index which is the average of the sub-items priority indices of each structure assessed.

Inspection sub-item priority index Ip_{ij} is calculated from the following empirical equations extracted from draft TMH 22, COTO (2013).

$$Ip_{ij} = 100 - \frac{100(k_d \, x \, D + k_e \, x \, E)R^a}{b_p}$$

……………………Equation 1

Where: Ip_{ij} = the priority index of inspection sub-item j of inspection item i,

D = degree rating for inspection sub-item j of item i,

E = extent rating for inspection sub-item j of item i,

R = relevancy rating for inspection sub-item j of item i,

k_d = degree coefficient (tentative default value: 1.0),

k_e = extent coefficient (tentative default value: 0.25);

a = relevancy exponent (tentative default value: 1.5), and

$$b_p = (k_d x\, D_{max} + k_e x E_{max}) R^a \qquad \text{……………………Equation 2}$$

Ip_{ij} ranges from 0 for D = 4, E = 4 and R = 4, which is a critical condition to 100, which reflects that there are no defects.

The inspection item priority index (Ip_i) for an item i, is then calculated from the following equation.

$$Ip_i = \frac{\sum_{j=1}^{j=n} Ip_{ij}}{\mathrm{n}} \qquad \text{…………………...Equation 3}$$

It should however be noted that D, E and R ratings were initially determined for each inspection item assessed in this research for a specific deterioration mechanism and determining the condition of the inspection item considering the worst defect.

2.4.6 Procedure for Rating of Defects

All defects on the inspection item are identified and recorded. The worst defect is considered for final rating of the inspection item in terms of D, E and R. The worst defect is the one usually with the highest relevancy rating.

2.4.7 Inspection Items

According to Draft TMH 22, COTO (2013), there are different numbers of inspection items for the different types of road structures. For example, there are twenty-one inspection items for bridge (general), eleven for bridge (cellular) and six for retaining walls.

The inspection items conventional numbering for bridges was used for the bridges assessment; for retaining walls, the conventional numbering was used for and is shown in Table 14 and 15 respectively.

Table 14: Bridge general inspection items

Inspection item number	description		Inspection item number	description		Inspection item number	description
1	Approach embankment		8	Surfacing		15	Bearings
2	Guardrail		9	Super-structure drainage		16	Support drainage
3	Waterway		10	Kerbs / sidewalks		17	Expansion joints
4	Approach embankment. Protection works		11	Parapet / handrail		18	Longitudinal member
5	Abutment foundations		12	Pier protection works		19	Transversal members
6	Abutments		13	Pier foundations		20	Deck slab
7	Wing / retaining walls		14	Piers & columns		21	Miscellaneous items

Conventional Inspection items for retaining walls are shown in the Table 15.

Table 15: Retaining wall defects

Inspection item number	description		Inspection item number	description		Inspection item number	description
1	External drainage defects		3	Wall defects		5	Internal drainage defects
2	Slope protection defects		4	Joint defects		6	Foundation defects

It is important to note that the bridges and the retaining walls were rated based on the requirements of TMH19, 2013 and the inspection item numbering conforms to the conventional numbering in this document. The DER-U rating system was specifically designed for road structures. However, for buildings, the author assumes that the rating system is applicable since no method for buildings exists and the numbering system employed for the purposes of this report is detailed in Section 3.3.3.

The advantages and limitations of the DER-U ratings system are as discussed in Table 16.

Table 16: Advantages and limitations of DER-U rating system

Rating System	Advantages	Limitations
DER-U	– It standardises the inspection and rating approach that is useful for the rating and prioritisation of the damages and subsequently informed maintenance strategies can be developed	– The condition stipulated for the use of the rating system in TMH19 as developed by COTO, (2013) is that, relevancy rating can never be greater than degree rating for bridges rating. Applying the same principle for some other structures, where the degree rating for cracking is dependent on the crack width, relevancy of cracks on the such structures may be underestimated

Despite the limitations highlighted in Table 16, DER-U rating is an important tool for the preliminary evaluation of non-road until a specific rating system has been developed for such structures.

CHAPTER 3

3. TRENDS OF CONCRETE DETERIORATION – CASE STUDIES

Twenty-four concrete structures were visually assessed by different UCT scholars. Structures assessed were in only three different provinces of South Africa namely; Gauteng, Western Cape and Eastern Cape. The findings from the visual assessments were captured in project reports. The reports of these assessments were analysed in this research to identify the main causes of concrete deterioration in South Africa and link these to environmental exposure conditions and geographical location. The specific structures were undergoing minor to significant deterioration of various elements. Concrete structures deteriorate at different rates depending on the location of the structures which have varying environmental conditions.

The structures assessed were also further classified based on the three main locations linked to the environmental exposure conditions i.e.

i. Marine - the structures which are either submerged in sea water or partly submerged and those that are in the tidal and splash zones of the sea,
ii. Coastal Areas – structures located at the coast, and
iii. Inland - structures located in any other areas other than Eastern Cape and Western Cape.

Five locality plans have been included. The coordinates of the assessed structures were used to plot the approximate positions. Figure 19 shows all the structures assessed and Figures 20, 21, 22 and 23 show zoomed in locations of structures in their respective provinces. Two maps for Western Cape province were included as there were many structures assessed in that province.

Table 16 presents a summary of all the assessed structures. It is important to note that the ages of some of the assessed structures were not specified in the assessment reports hence indicated as unknown. The structure reference number in Table 17 refers to the assessment report numbers and is consistent in the entire document for easy of reference. The age of the structure assessed is relative to the assessment date of the concrete structure.

The structure reference numbers indicated in Table 17 are consistent in the entire dissertation for easy of reference. Furthermore, in this dissertation context, the deference between marine and coastal structures has been discussed in Section 3.2.1 and 3.2.2.

Table 17: Summary details of assessed structures

Structure reference number	Type of structure	Age of Structure	City	Exposure Class	Marine/Coastal / Inland	Assessor	Assessment Date
1	Building	85	East London	XS1	Marine	Evance F Nyambalo	Aug-15
2	Bridge	Unknown	Port Elizabeth	XS1	Inland	Tulen Lawrence Zahemen	Aug-15
3	Building	20	East London	XS3	Marine	Jimmy Takaindisa	Aug-15
4	Bridge	30	Coffee Bay	XS1	Coastal	Darison Mashanda	Jul-14
5	Building	Unknown	Tshwane	XC4	Inland	Grandeur Tofara Hove	Jul-14
6	Bridge	Unknown	Johannesburg	XC4	Inland	Malaudzi Mukhethwa	Aug-15
7	Building	Unknown	Pretoria	XC1	Inland	Myezo Poyo	Jul-14
8	Building	Unknown	Johannesburg	XC4	Inland	Kamlin Moodley	Jul-14
9	Building	70	Johannesburg	XC4	Inland	Keamogetswe Mmekwa	Aug-15
10	Liquid Berth Bridge Structures	Unknown	Cape Town	XS3	Marine	Luqmaan Jappie	Aug-15
11	Retaining Wall	50	Cape Town	XS1	Coastal	Olukayode O. Alao	Jul-14
12	Retaining Wall	15	Cape Town	XS3	Marine	Jarryd Buratovich	Jun-15
13	Building	Unknown	Cape Town	XS1	Coastal	Bester	Jul-13
14	Bridge	20	Cape Town	XS1	Coastal	John B. Kamara	Jun-14
15	Building	Unknown	Cape Town	XS1	Coastal	William Smith	Aug-15

Table 17 continued…

Structure reference number	Type of structure	Age of Structure	City	Exposure Class	Marine/Coastal / Inland	Assessor	Assessment Date
16	Building	Unknown	Cape Town	XS3	Coastal	Gerard De Swardt	Aug-15
17	Bridge	Unknown	Cape Town	XS1	Coastal	Gesant Abed	Aug-15
18	Building	Unknown	Cape Town	XS1	Coastal	Primesh Jassa	Jul-15
19	Building	40	Cape Town	XS1	Coastal	Golden G.C	Jul-14
20	Bridge	55	Cape Town	XS1	Coastal	Emmanuel Jenkeri Okwori	Jul-14
21	Bridge	Unknown	Cape Town	XS1	Coastal	Yusuf Salie	Jul-14
22	Bridge	Unknown	Cape Town	XS1	Coastal	Ezekiel Arito	Jun-14
23	Bridge	45	Cape Town	XS1	Coastal	Owen Davies	Aug-15
24	Building	40	Cape Town	XS1	Coastal	Anton Marais	Aug-15

3.1 Locality maps of the assessed structures

The approximate geographical locations of the structures visually assessed were plotted on the map of South Africa and are shown in Figure 19. Zoomed in maps for the specific provinces are also shown in Figure 20, 21, 22 and 23.

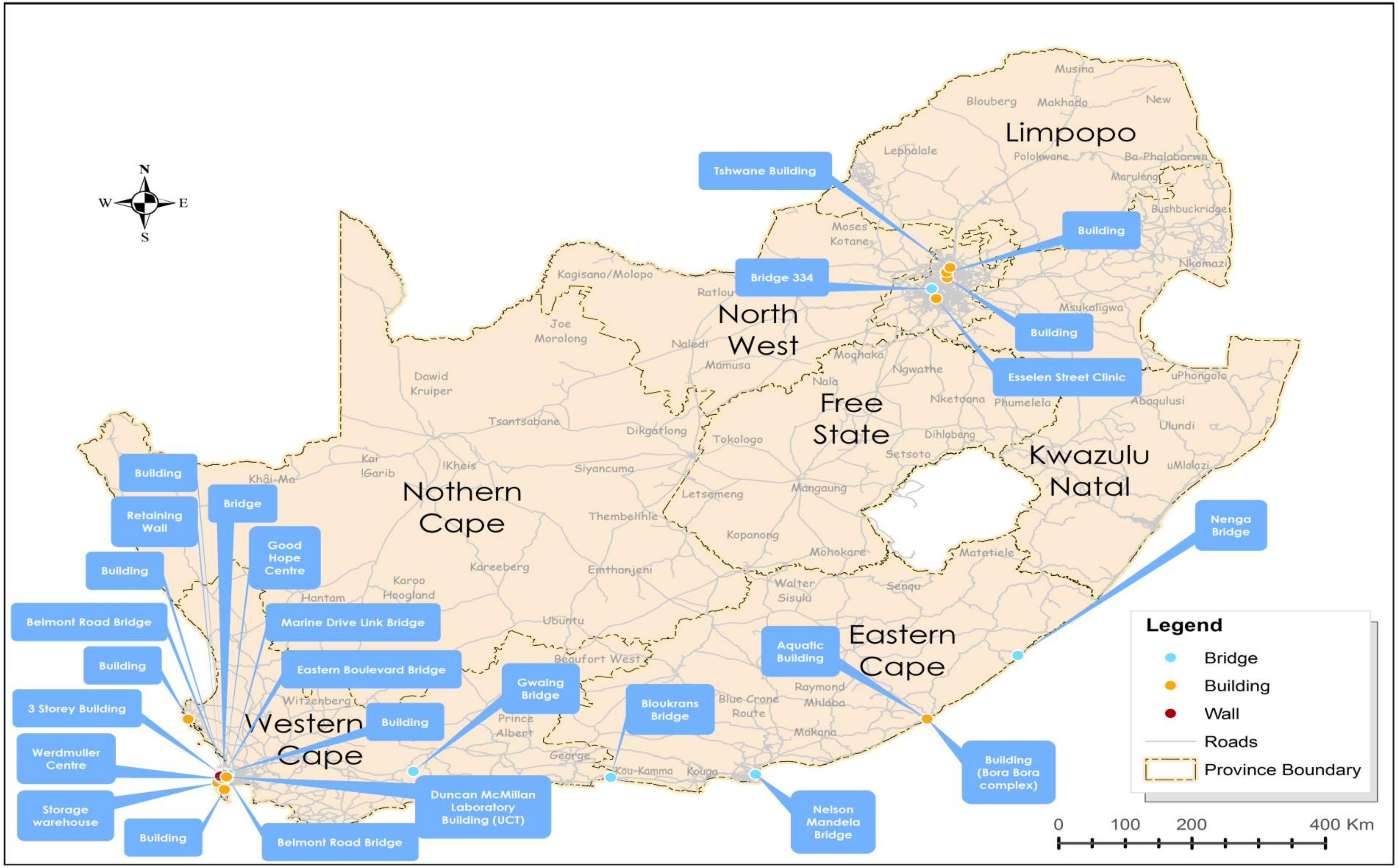


Figure 19: Location of structures visually assessed in South Africa

A map showing the locations of the assessed structures in the Western Cape Province is shown in Figure 20 and Figure 21.

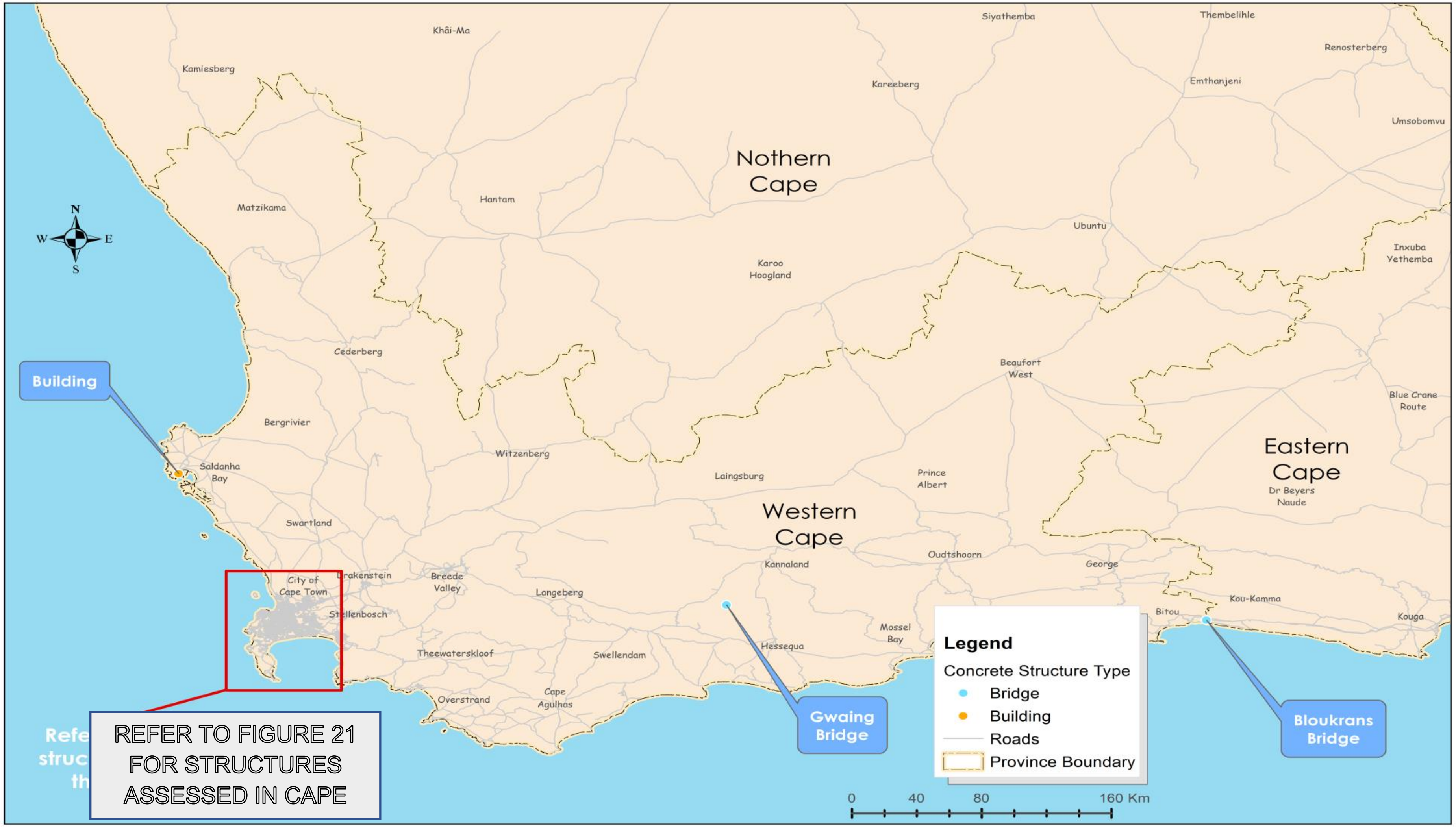


Figure 20: Location of Structures in the Greater Western Cape

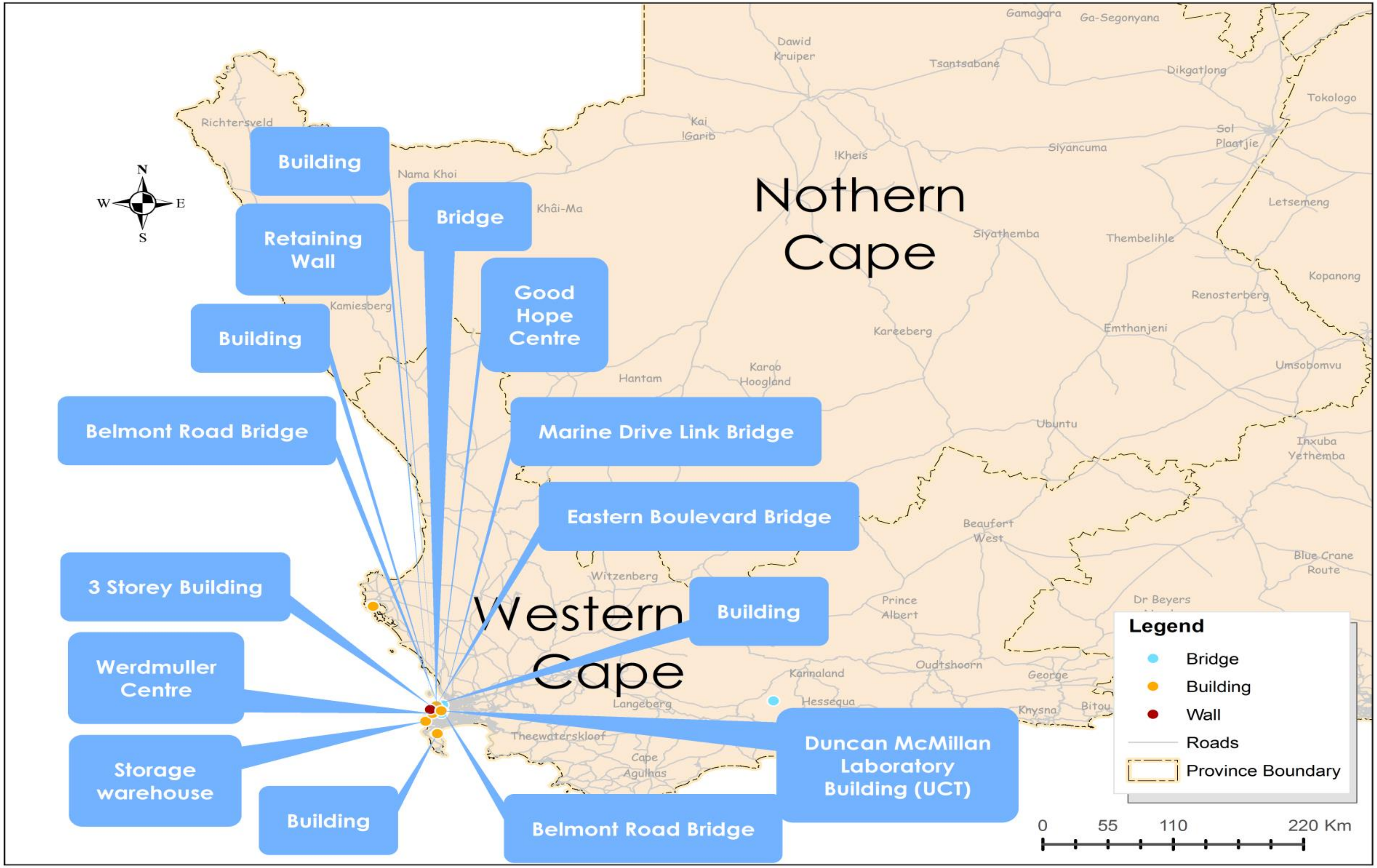


Figure 21: Location of structures evaluated in the Western Cape Metro

A map showing the locations of the assessed structures in the Eastern Cape Province is shown in Figure 22.

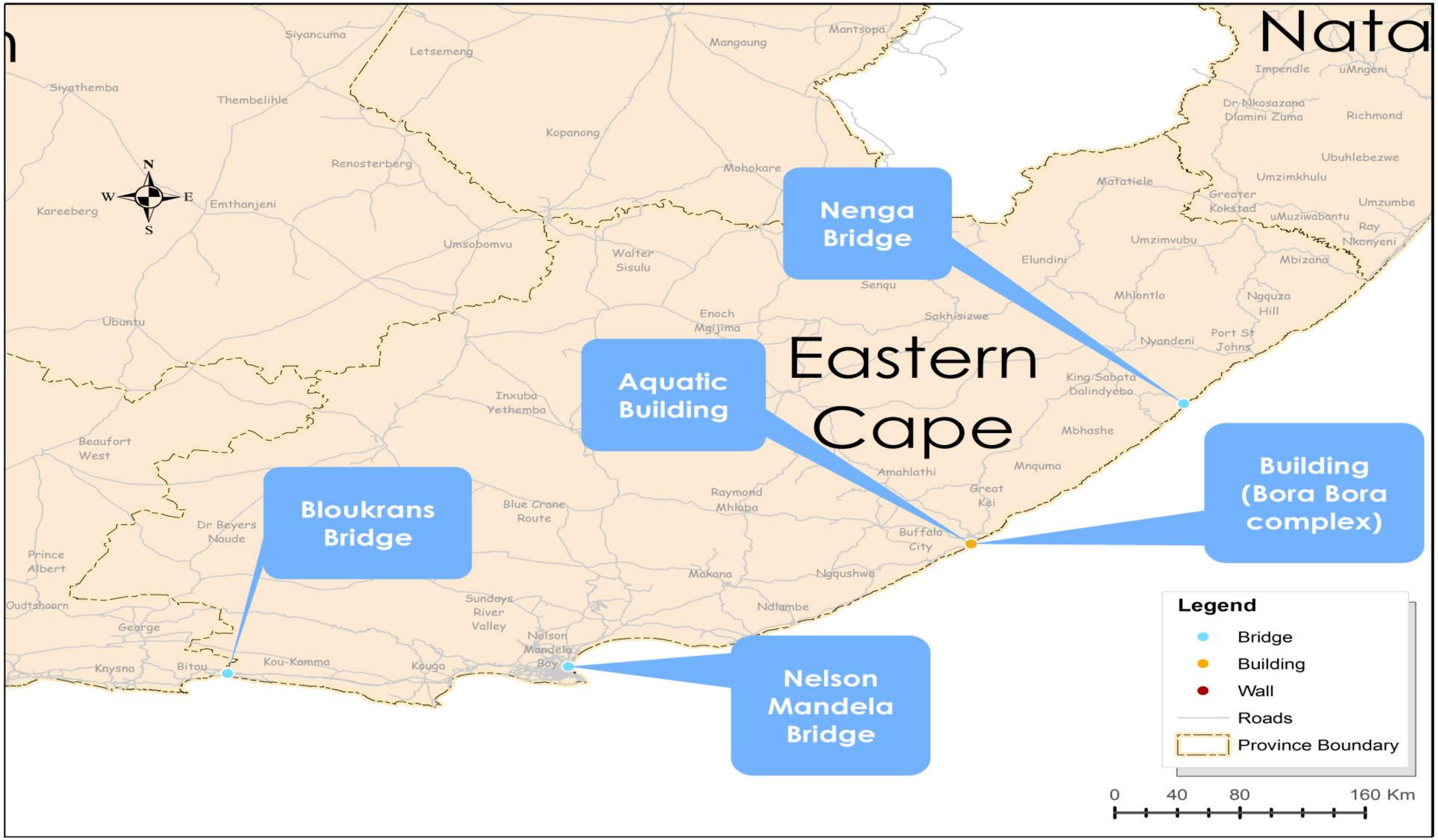


Figure 22: Location of structures assessed in the Eastern Cape Province

A map showing the locations of the assessed structures in the Gauteng is shown in Figure 23.

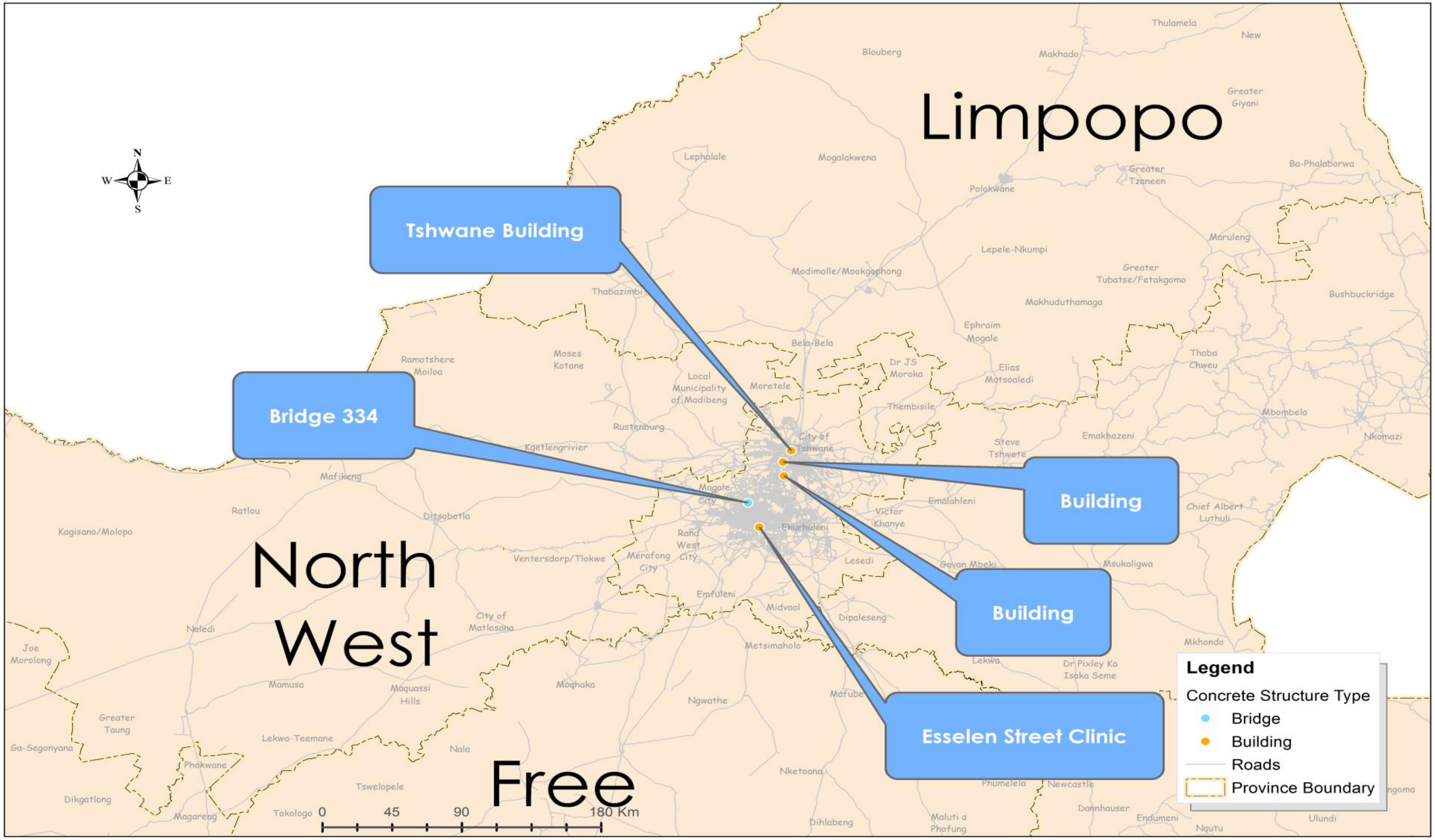


Figure 23: Location of structures assessed in Gauteng province

3.2 Photographs of the observed defects

As alluded in Section 3 of this report, twenty-four concrete structures were visually assessed. The assessment reports incorporated photographs of the specific observed defects and a brief description of the likely deterioration mechanism. The photographs were extracted and are grouped in different locations.

3.2.1 Marine structures

Marine structures are those structures which are partly submerged in the sea. Typical photographs of observed defects in the Marine Environment are shown below.

3.2.1.1 Reinforcing bar corrosion

All the marine structures assessed and with the exposure class XS3 have suffered corrosion damage. The defects observed were cracking and rust stains on the unsubmerged sections. The sample photos from Figure 24 to Figure 28 bear the same characteristics as discussed in Section 2.1.1, hence conclusions that the observed defects were because of reinforcing bar corrosion deterioration mechanism.

Cracking and rust stains on the edge beam of the Liquid berth structure as a result of reinforcing bar corrosion.

Figure 24: Reinforcing bar corrosion damage: Liquid Berth Bridge structures at Port of Cape Town (Jaapie, 2015)

There was no information provided on the age of the visually assessed Liquid Berth bridge structure.

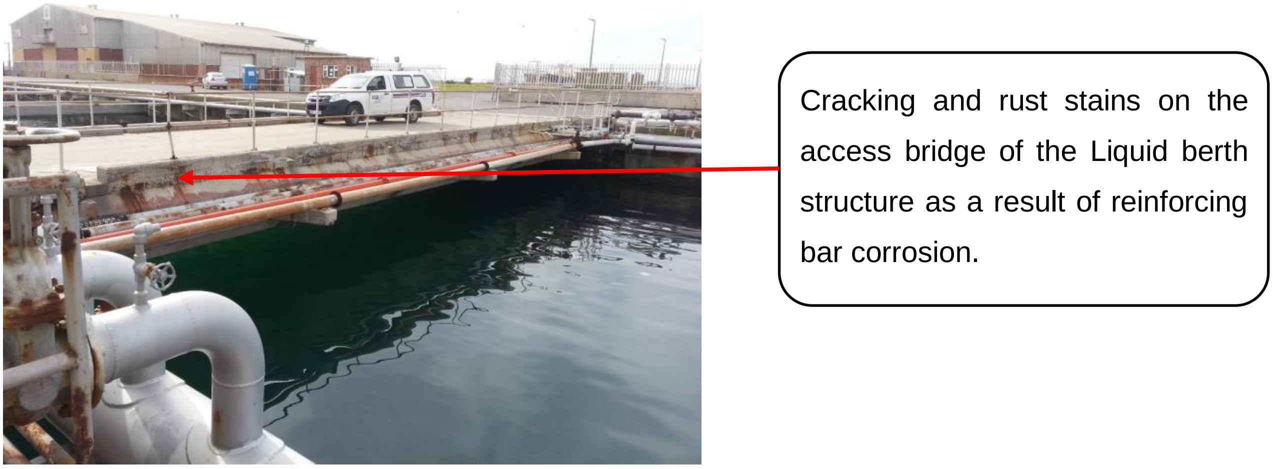


Figure 25: Reinforcing bar corrosion damage: Liquid Berth Bridge structures at Port of Cape Town (Jaapie, 2015)

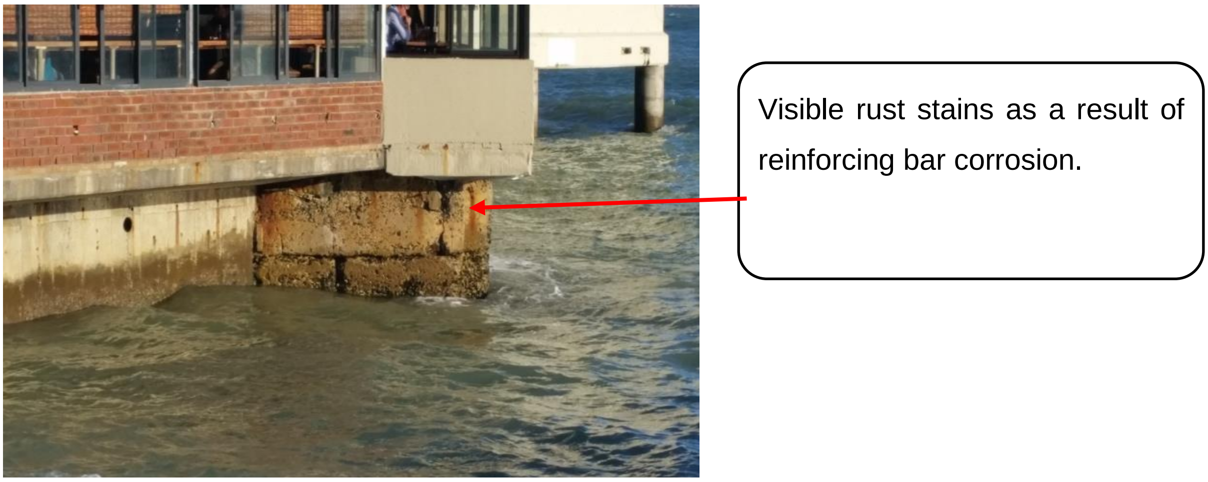


Figure 26: Reinforcing bar corrosion damage: Building in the tidal zone, Saldanha Bay, Cape Town (Swardt, 2015)

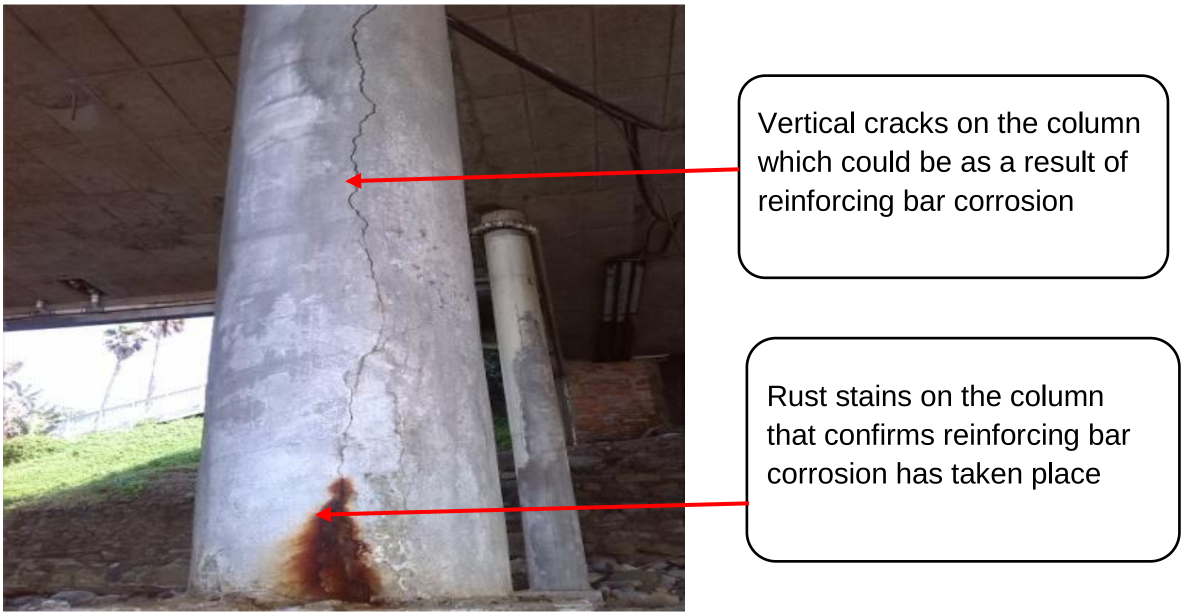


Figure 27: Reinforcing bar corrosion: column support of a building in the tidal and splash zone, East London (Takaindisa, 2015)

Poor construction practice influences reinforcing bars corrosion as depicted in Figure 28.

Horizontal crack with notable rust staining. The horizontal crack could be as a result of the construction defect from improper construction joint. The joint allowed the ingress of water and chlorides resulting in chloride induced corrosion

Figure 28: Cold joint: Retaining Wall at Glen Beach, Camps Bay Cape Town (Buratovich, 2015)

3.2.1.2 Leaching

Leaching has been observed on all the marine structures assessed with the exposure class XS3. The defects observed were the white patches on the surface of the concrete structure.

The sample photos from Figure 29 to Figure 33 bear the same characteristics as discussed in the literature review Section 2.2.1 and Figure 15, hence conclusions that the observed defects were as a result of leaching deterioration mechanism.

Dry dock stair showing signs of leaching at cracks which can be linked to sulphate attack. Age of structure is unknown

Figure 29: Leaching: Dry dock stair at Port of Cape Town (Jaapie, 2015)

Figure 30: Column support of a building in the tidal and splash zone, East London (Takaindisa, 2015)

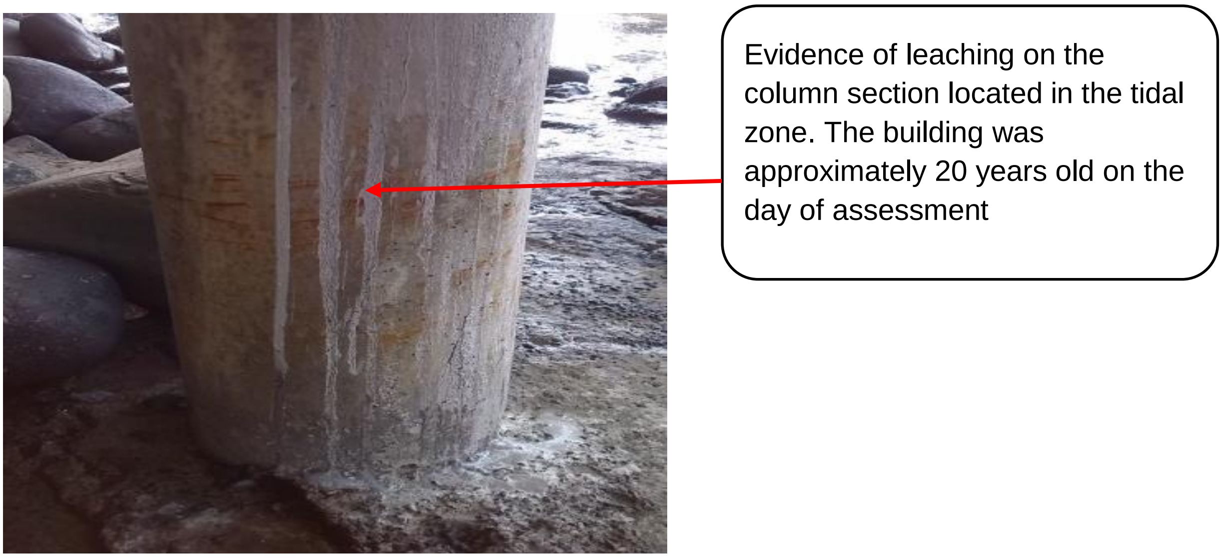


Figure 31: Leaching: Column support of a building in the tidal and splash zone, East London (Takaindisa, 2015)

Figure 32 is a typical example of leaching of chemical reaction products as a result of sulphate attack as discussed in Section 2.1.5.

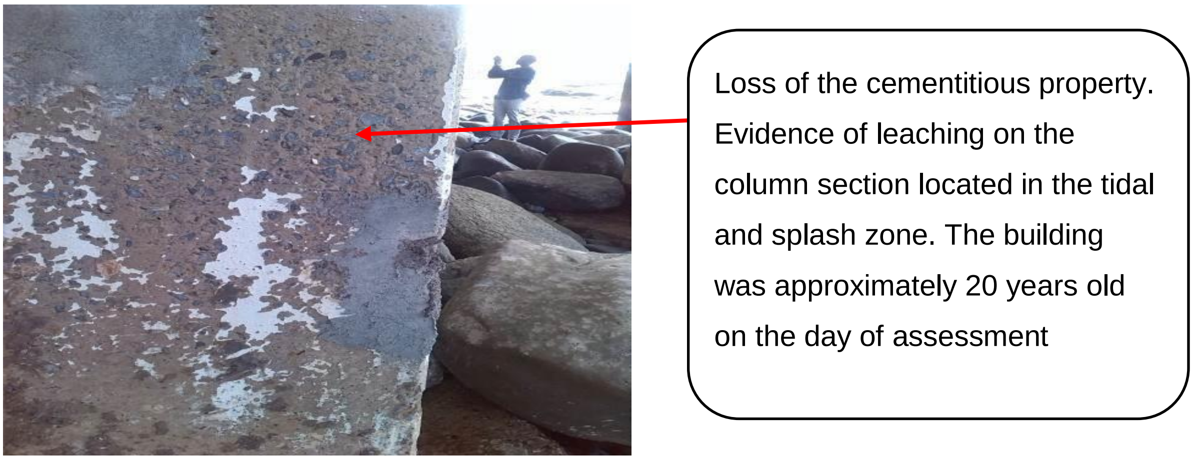


Figure 32: Leaching: Column support of a building in the tidal and splash zone, East London (Takaindisa, 2015)

Retaining Wall at Glen Beach, Camps Bay showing with white patches on the surface which is a sign of leaching

Figure 33: Leaching: Retaining Wall at Glen Beach, Camps Bay, Cape Town (Buratovich, 2015)

3.2.1.3 Drying shrinkage

Drying shrinkage deterioration mechanism was observed on the retaining wall at Glen Beach, Camps Bay Cape Town; a marine environment with the exposure class of XS3. The defects observed confirm that the mechanisms were large cracks at regular intervals on the entire wall. The retaining wall was approximately fifteen years old on the day of assessment and no expansion joints were provided.

Typical example photos are shown and zoomed in photographs from Figure 33. The vertical cracks observed bears the same characteristics as discussed in Section 2.1.3 and a typical shrinkage as depicted on Figure 9 and 11, hence the conclusion that the cracks shown on Figure 34 were because of drying shrinkage.

Retaining Wall at Glen Beach, Camps Bay showing white patches on the surface and cracks which is a sign of leaching and drying shrinkage cracks respectively

Figure 34: Drying shrinkage cracks: Retaining Wall at Glen Beach, Camps Bay, Cape Town (Buratovich, 2015)

3.2.1.4 Abrasion

Abrasions have been observed on all the marine structures assessed with the exposure class XS3 and typically exposed to tidal and splash zones. Typical example photos are shown in Figure 35 and Figure 36.

Evidence of abrasion on the foundation located in the tidal and splash zone. The retaining wall was 15 years old on the day of assessment

Figure 35: Abrasion on retaining wall foundation in the tidal and splash zone (Buratovich, 2015)

Evidence of abrasion on the column section located in the tidal and splash zone. The building was 20 years old on the day of assessment

Figure 36: Abrasion on column in the tidal and splash zone (Takaindisa, 2015)

3.2.2 Coastal areas

The structures located at the coastal areas were observed and showed extensive deterioration. The coastal areas in this context are close to the sea to a maximum of 20km from the sea. Various deterioration mechanisms were observed, and selected photographs of the defects observed during the visual condition assessments for the buildings located at the coastal areas are shown below. The defects observed and the possible deterioration mechanisms are discussed below.

3.2.2.1 Reinforcing bars corrosion

The sample photos from Figure 37 to Figure 48 have the same characteristics as discussed in the literature review in Section 2.1.1, hence conclusions that the observed defects were because of reinforcing bars corrosion deterioration mechanism.

The retaining wall is severely damaged due to chloride-induced corrosion. Furthermore, the cover provided was not adequate for the exposure conditions. The wall was 50 years old at the date of assessment

Figure 37 : Reinforcing bars corrosion: Retaining wall 50m from the sea in Cape Town (Alao, 2015)

The Aquarium Building has severely deteriorated; spalling due to reinforcing bars corrosion is rampant. The building was 85years old at the time of assessment

Figure 38: Reinforcing bars corrosion: Spalling on deck slab, East London Aquarium Building (Nyambalo, 2015)

The deck has deteriorated severely. It can be noted from the photograph, Figure 38 that the building was recently repainted without applying proper patch repair procedures and without application of informed maintenance strategies.

Severe spalling on the columns and vertical cracks due to reinforcing bars corrosion. The building was 85 years old at the time of assessment

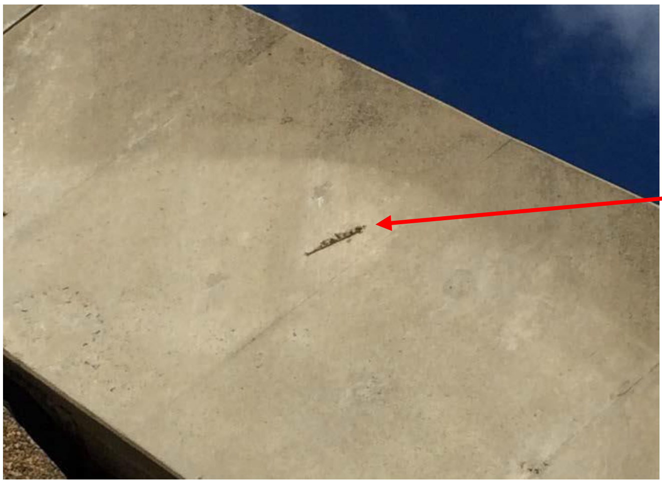
Figure 39: Reinforcing bars corrosion: Spalling and rust on column, Aquarium building, East London (Nyambalo, 2015)

Spalling on the beam support due to reinforcing bar corrosion. The reinforcing bars have rusted, and it is suspected that the structural integrity is compromised. The building was 85 years old at the time of assessment

Figure 40: Reinforcing bars corrosion: Spalling and rust on beam, East London Aquarium Building (Nyambalo, 2015).

Concrete spalling below a window opening due to reinforcing bars corrosion. The building was 85 years old at the time of assessment

Figure 41: Reinforcing bars corrosion: Spalling on column and rust, East London Aquarium Building (Nyambalo, 2015).



Spalling due to localised reinforcing bars corrosion on the crown of the west gable arch. The building was 39 years old at the time of

Figure 42: Reinforcing bars corrosion: Good Hope Centre, Cape Town, (Bester, 2013)

Figure 43: Reinforcing bars corrosion: Storage Warehouse, Hout Bay, Cape Town, (Smith, 2015).

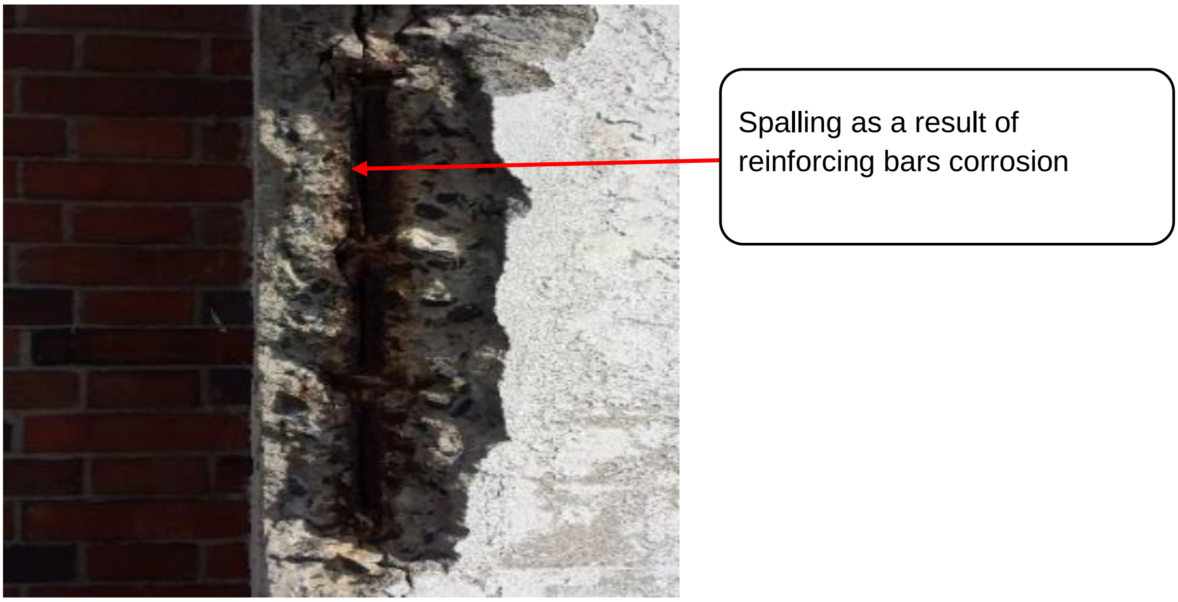


Figure 44: Reinforcing bars corrosion: Storage Warehouse, Hout Bay, Cape Town, (Smith, 2015).

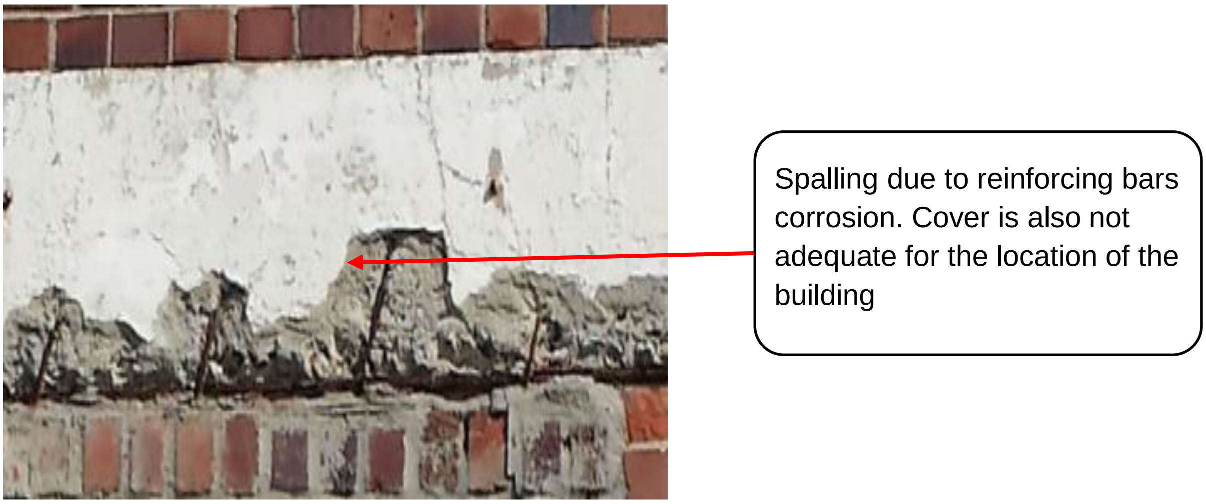


Figure 45: Reinforcing bars corrosion: Storage Warehouse, Hout Bay, Cape Town, (Smith, 2015).

Spalling as a result of reinforcing bars corrosion and exacerbated by inadequate cover, +/-1Km from the sea.

Figure 46: Reinforcing bars corrosion: Nelson Mandela Bridge, Port Elizabeth, (Zahemen, 2015).

Spalling as a result of reinforcing bars corrosion, the bridge was +/-1Km from the sea.

Figure 47: Reinforcing bars corrosion: Nelson Mandela Bridge, Port Elizabeth, (Zahemen, 2015).

Rust stains on the foundation plinths as a result of chloride induced corrosion. The building was 39 years old at the time of assessment.

Figure 48: Reinforcing bars corrosion: Good Hope Centre, Cape Town, (Bester, 2013)

Alkali-silica reaction (ASR)

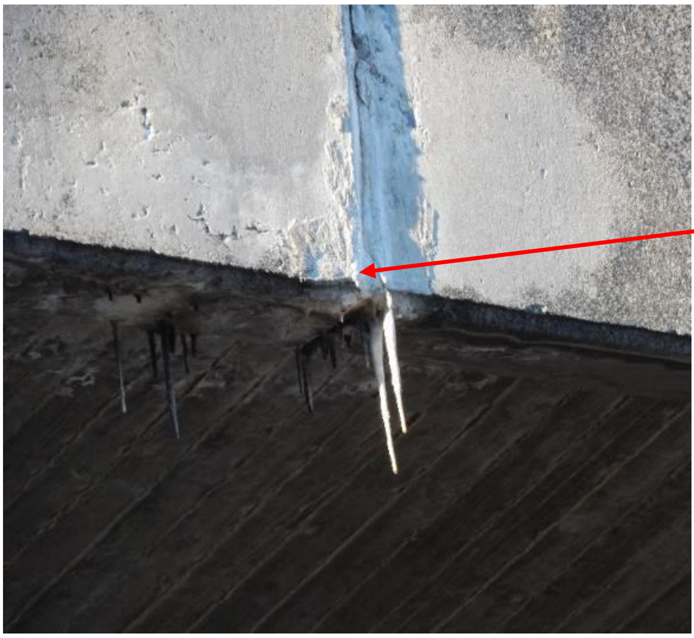
The sample photo in Figure 53 bear the same characteristics as discussed in the literature review in Section 2.1.2 and as shown on the typical photographs i.e. Figure 7 and Figure 8, hence conclusions that the observed defects were due to Alkali-silica reaction to deterioration mechanism.

Severe cracks on the foundation plinth could be as a result of ASR. The cracks allowed the ingress of moisture and chlorides that led to the rebar corrosion. The building was 39 years old at the time of assessment

Figure 49: ASR: Good Hope Centre, Cape Town, (Bester, 2013)

3.2.2.2 Leaching

The sample photo in Figure 50 bears the same characteristics as discussed in the literature review in Section 2.1.1, hence conclusion that the observed defects were due to leaching.



Leaching on bridge deck beams structure. The bridge was +/-1Km from the sea.

Figure 50: Leaching: Nelson Mandela Bridge, Port Elizabeth, (Zahemen, 2015).

3.2.2.3 Mechanical damage

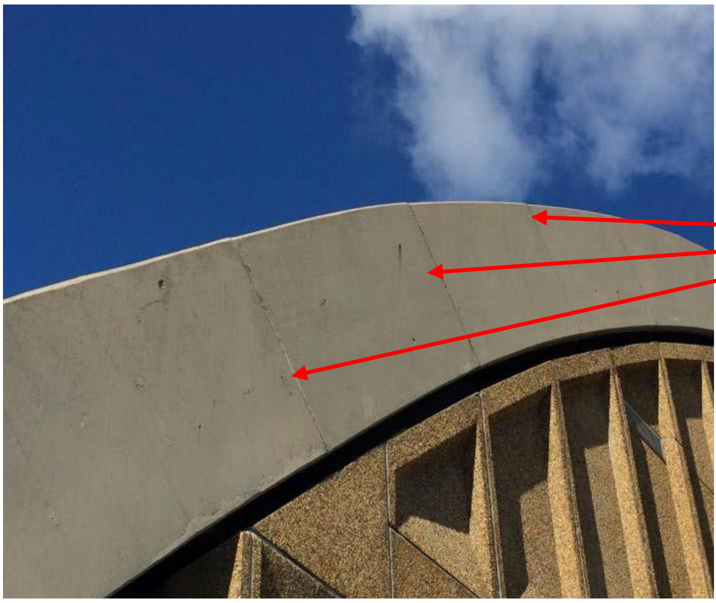
The sample photo in Figure 51 bears the same characteristics as discussed in the literature review in Section 2.2.1 and Figure 17, hence conclusions that the observed defects were because of impact deterioration mechanism.

Mechanical damage was observed and it was reported that a mobile crane caused the damage due to the operator not lowering the crane down enough, before traveling under the bridge.

Figure 51: Mechanical damage: Road over Road Bridge, Saldanha Bay, (Swardt, 2015).

3.2.2.4 Drying shrinkage

The vertical cracks observed bears the same characteristics as discussed in Section 2.1.3 and a typical shrinkage as depicted on Figure 11, hence the conclusion that the cracks shown in Figure 52 could be a result of drying shrinkage although it can also be added that the cracks widths were exacerbated by reinforcing bars corrosion.



Drying shrinkage cracks - vertical cracks which are located at an approximately equal distance apart on the gable arch. The building was 39 years old at the time of assessment

Figure 52: Drying shrinkage cracks: Good Hope Centre, Cape Town, (Bester, 2013)

3.2.3 Inland areas

The inland areas in this context are those that are located more than 20km away from the sea. The structures located in the inland areas have been observed to have deteriorated but not as severe as the structures at the coastal areas. Various deterioration mechanisms were observed, and selected photographs of the defects observed during the visual condition assessments for the buildings located in the inland areas are shown below. The defects observed and the possible deterioration mechanisms are discussed.

3.2.3.1 Reinforcing bars corrosion

The sample photos from Figure 53 to Figure 56 bear the same characteristics as discussed in the literature review in Section 2.1.1, hence the conclusion that observed defects were a result of reinforcing bars corrosion deterioration mechanism.

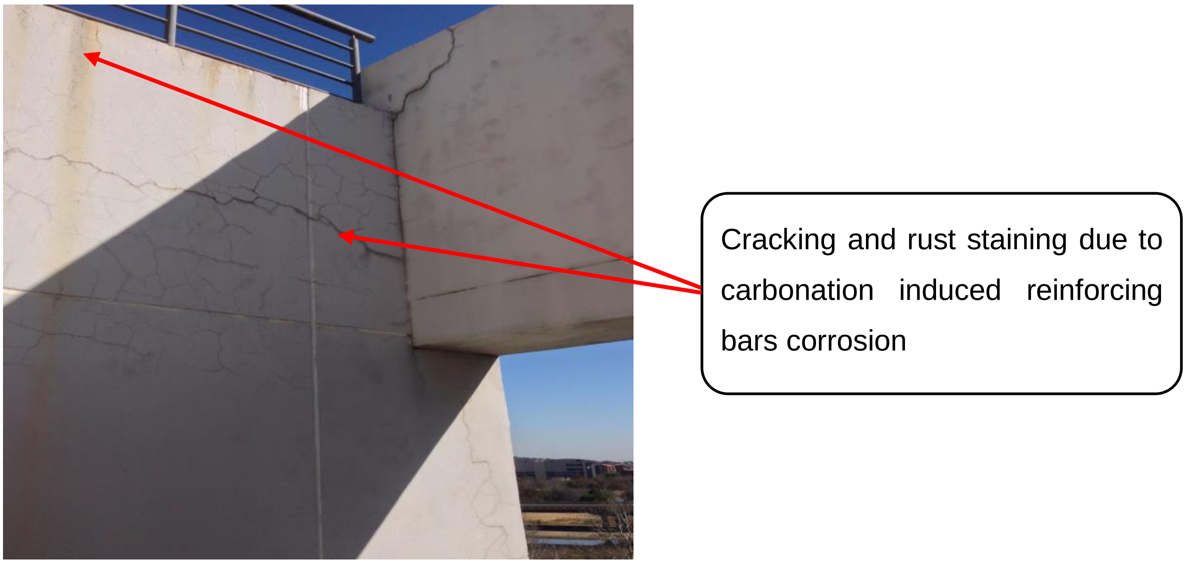


Figure 53: Reinforcing bars corrosion. Lakeside 2 Building, Centurion, (Moodley, 2014)

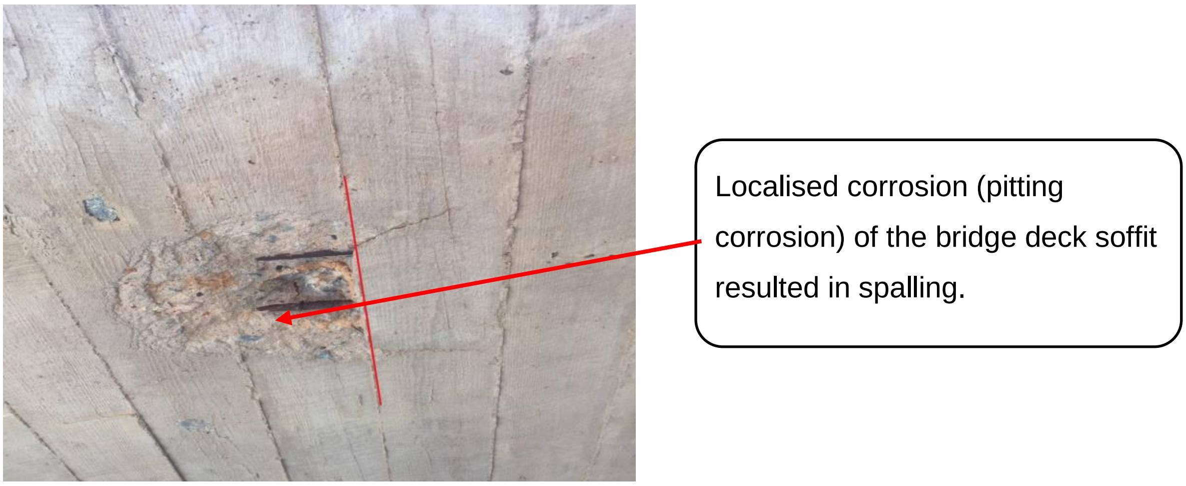


Figure 54: Reinforcing bars corrosion (Pitting). Bridge 334, Witkoppen Road, Johannesburg, (Mukhethwa, 2015)

Figure 55: Reinforcing bars corrosion. Tshwane building, Pretoria, (Hove, 2015)

Figure 56 shows a typical reinforcing bars corrosion damage taken on the soffit of the bridge deck. Corrosion was exacerbated by honeycombing which is a typical example of poor construction process. Honeycombing resulted in the steel reinforcing bars to be exposed to harsh environmental conditions and prone to corrosion.

Figure 56: Reinforcing bars corrosion. Bridge 334, Witkoppen Road, Johannesburg, (Mukhethwa, 2015)

3.2.3.2 Leaching

The sample photos in Figure 57 and Figure 58 bear the same characteristics as discussed in the literature review in Section 2.1.1, hence the conclusion that observed defects were as a result of leaching.

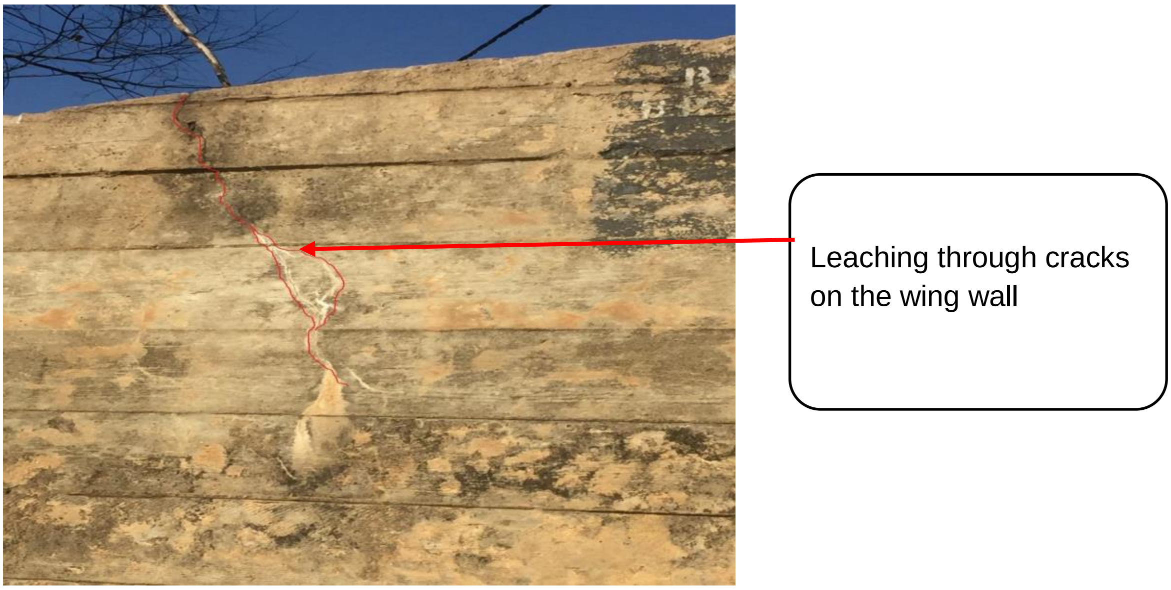


Figure 57: Leaching. Bridge 334, Witkoppen Road, Johannesburg, (Mukhethwa, 2015)

Figure 58 shows deterioration damage due to leaching which has been initiated by the leaking sewer pipe. The damage can be linked to poor construction practices.

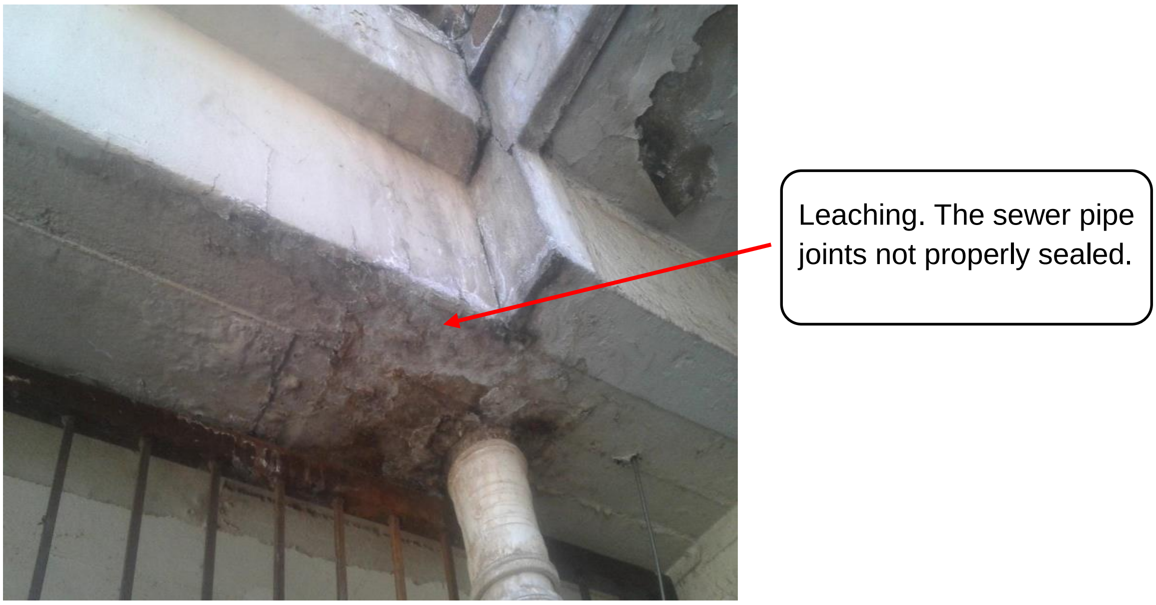


Figure 58: Leaching. Tshwane building, Pretoria, (Hove, 2015)

3.2.3.3 Alkali-silica reaction

The sample photo in Figure 65 bears the same characteristics as discussed in the literature review, Section 2.1.2. Also, as shown on the typical photographs i.e. Figure 7 and Figure 8; hence conclusions that the observed defects were due to reinforcing bars corrosion deterioration mechanism.

Crocodile cracks on the column which is by the roof access staircases. Crocodile cracks were assumed to be due to ASR.

Figure 59: Alkali-Silica reaction. Lakeside 2 Building, Centurion, (Moodley, 2014).

3.2.3.4 Drying shrinkage

The vertical cracks observed bears the same characteristics as discussed in Section 2.1.3 and a typical shrinkage as depicted on Figure 9 and 11, hence the conclusion that the cracks shown on Figure 60 were because of drying shrinkage. Furthermore, no expansion joints were provided on the retaining wall.

Drying shrinkage cracks, due to non-existence of the expansion joints on the wing wall.

Figure 60: Drying shrinkage. Bridge 334, Witkoppen Road, Johannesburg, (Mukhethwa, 2015)

The fine cracks observed bears the same characteristics as discussed in Section 2.1.3 and typical shrinkages as depicted on Figure 11, hence the conclusion that the cracks shown on Figure 61 were because of drying shrinkage.

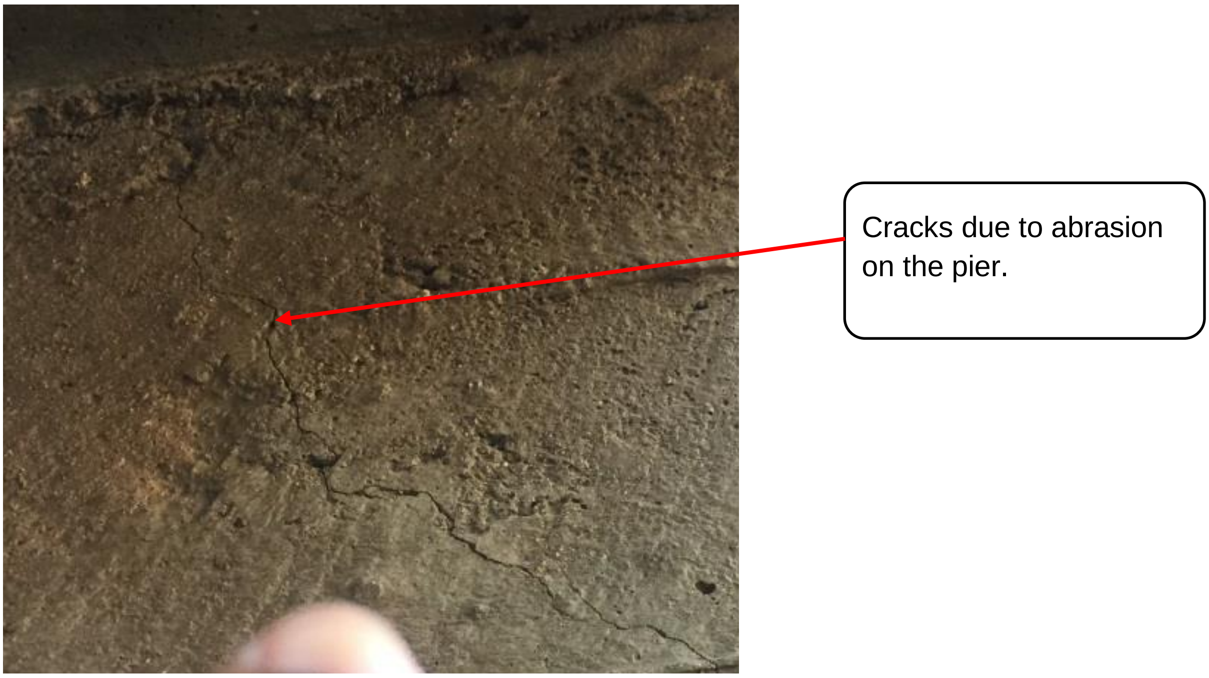


Figure 61: Abrasion. Bridge 334, Witkoppen Road, Johannesburg, (Mukhethwa, 2015)

3.2.3.5 Abrasion

The loss of aggregates observed bears the same characteristics as discussed in Section 2.1.4 and typical abrasion damage as depicted on Figure 12, hence the conclusion that the defects shown on Figure 68 were because of abrasion.

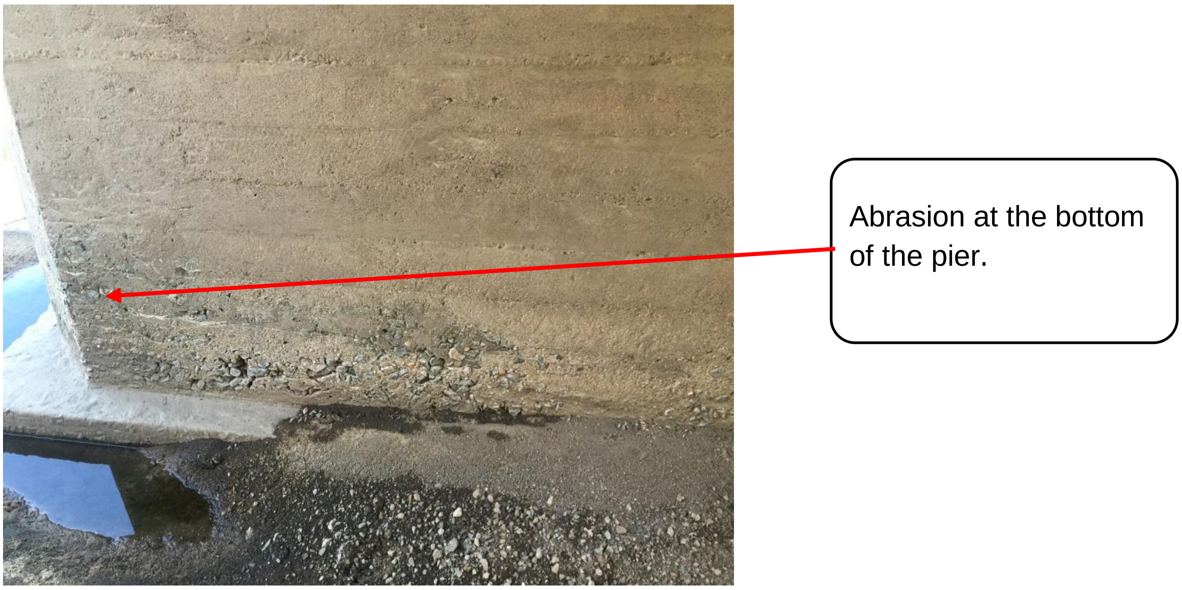


Figure 62: Abrasion. Bridge 334, Witkoppen Road, Johannesburg, (Mukhethwa, 2015)

3.2.3.6 Construction defect

Figures 64, 65 and 66 are typical examples of the observed defects due to poor construction practices. Honeycombing and cold joints allow moisture and deleterious substance to the center the structure thus, accelerating other deterioration mechanisms.

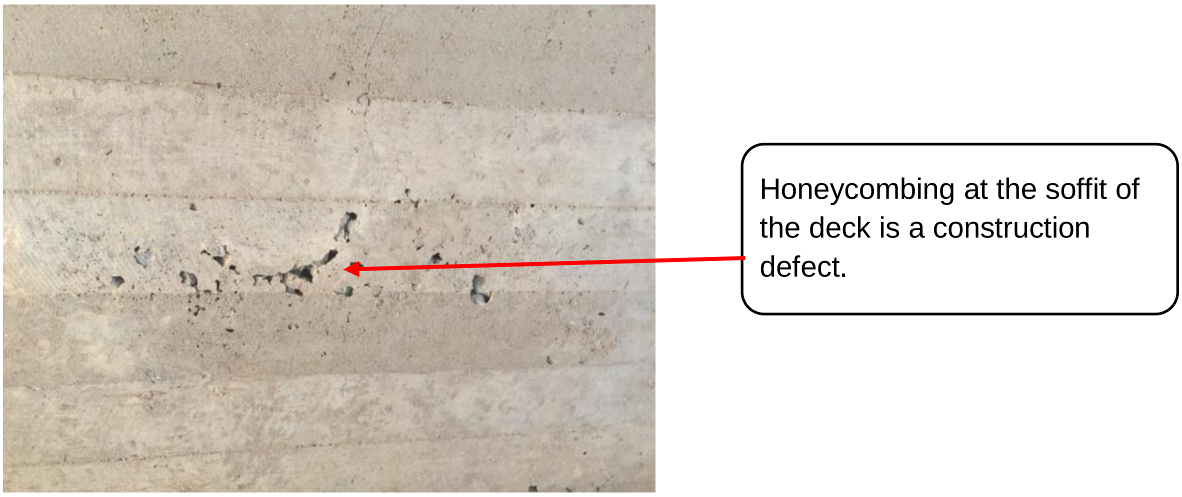


Figure 63: Construction defect, Honeycombing. Bridge 334, Witkoppen Road, Johannesburg, (Mukhethwa, 2015)

The cold joint in Figure 65 provides an easy access of deleterious substances that may result in the initiation of reinforcing bars corrosion.

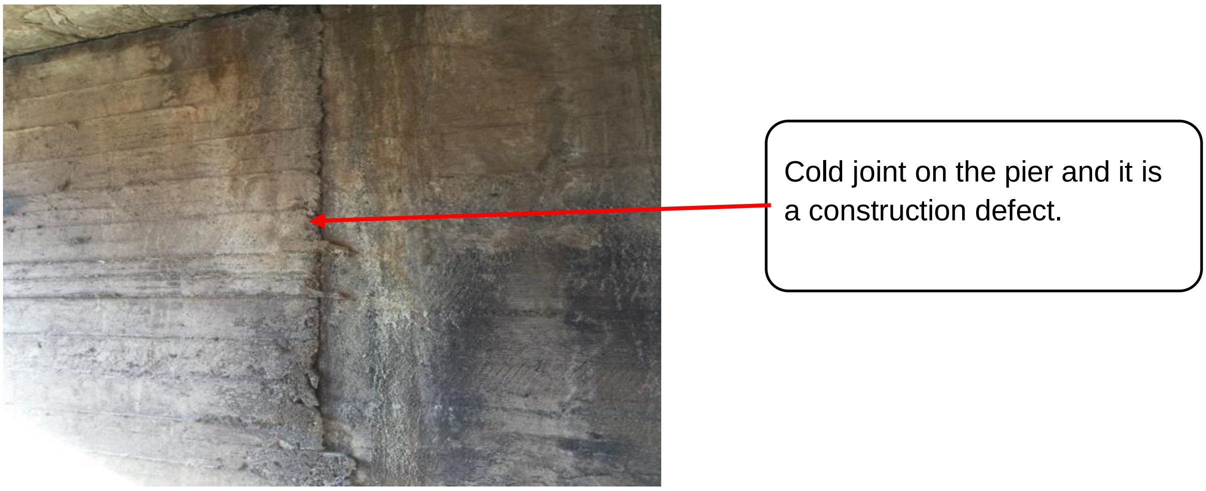


Figure 64: Cold joint. Bridge 334, Witkoppen Road, Johannesburg, (Mukhethwa, 2015)

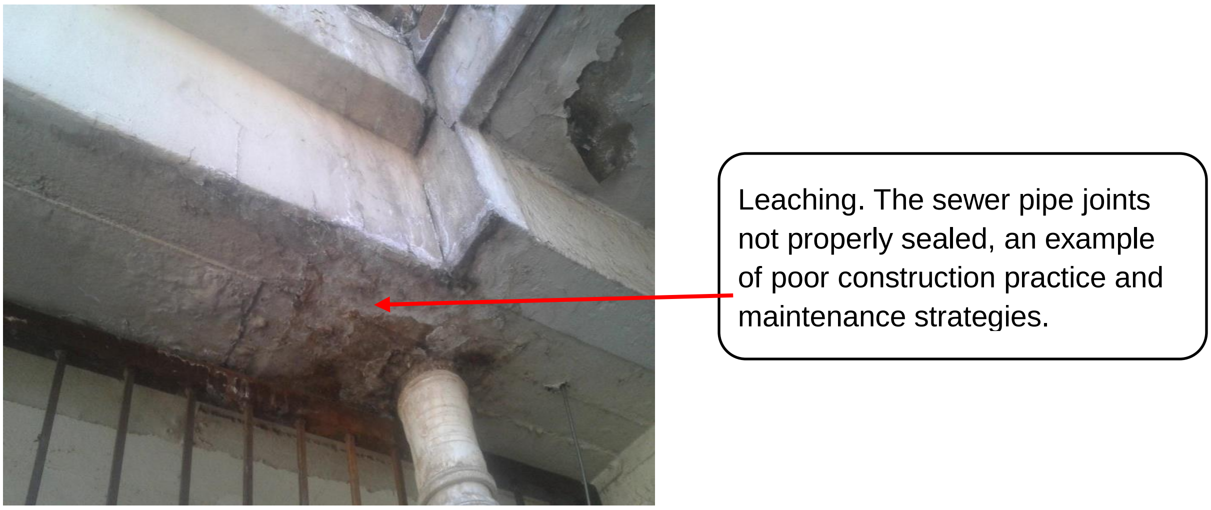


Figure 65: Poor construction practices and maintenance strategies. Tshwane building, Pretoria, (Hove, 2015)

3.3 Data Analysis

Twenty-four concrete structures were assessed in South Africa, of which fifteen were in the Western Cape province, four in the Eastern Cape province and five in the Gauteng province. The concrete structures assessed include; twelve general use buildings, ten bridges, and two retaining walls. It can be noted that many structures were assessed in the Western Cape than any other province. It is important to note that the outcome of the analysis may not be necessarily accurate but has been considered acceptable for preliminary evaluation. Large sample sizes and uniform on all provinces is preferred for narrow error margin.

The coordinates of all the assessed structures were provided for in the assessment reports and were used to plot the positions of the structures. The coordinates assisted in the determination of the geographical locations of all structures visually assessed. The maps showing the locations are included.

Furthermore, all the structures' respective exposure classes were determined based on EN 206-1:2000 classification. Literature on the exposure classes was provided and discussed in the literature review in Section 2.3.

Table 18 shows the abbreviations of the defects considered in this research document and are used in Table 19, 20 and 21.

Table 18: Abbreviations of deterioration mechanisms

Abr = Abrasion	ASR = Alkali-Silica Reaction	Rbc = Reinforcing bars corrosion
Lch = Leaching	Ds = Drying shrinkage	Tc = Thermal cracking
Md = Mechanical damage	Ps = Abrasion	Cd = Construction defect
Stf = Structural failure	Pr = Province	Exc = Exposure class

Table 19 summarises the structures assessed, exposure class of each structure and deterioration mechanisms observed on each structure assessed. Table 18 must be read in conjunction with Table 17.

Table 19: Summary of structures assessed and observed deterioration mechanisms

Structure reference number	DETERIORATION MECHANISM										Exc
	Abr	ASR	Rbc	Lch	Ds	Tc	Md	Ps	Cd	Stf	
1	-	-	√	-	-	-	-	-	-	√	XS1
2	-	-	√	√	√	-	-	√	√	-	XS1
3	√	-	√	√	-	-	-	-	-	-	XS3
4	-	√	√	√	√	-	√	-	√	√	XS1
5	-	√	√	√	-	-	-	√	-	√	XC4
6	√	√	√	√	√	√	-	-	√	√	XC4
7	-	-	√	-	√	√	-	√	-	-	XC1
8	-	√	√	√	√	-	-	-	√	√	XC4
9	-	-	√	-	-	-	-	-	-	-	XC4
10	√	√	√	√	√	-	√	-	√	√	XS3
11	-	-	√	-	-	-	-	-	-	-	XS1
12	√	-	√	-	√	-	-	-	√	-	XS3
13	-	√	√	√	√	-	√	-	-	-	XS1
14	√	-	√	√	-	-	-	-	-	√	XS1
15	√	-	√	-	-	-	-	-	-	√	XS1
16	√	-	√	-	-	√	√	-	-	√	XS3
17	-	-	√	√	-	√	√	-	-	-	XS1
18	-	√	√	√	√	-	-	√	-	√	XS1
19	-	-	√	√	-	-	-	-	-	√	XS1
20	√	-	√	√	-	-	-	-	-	√	XS1
21	-	-	√	√	-	-	-	√	-	√	XS1
22	-	-	√	-	√	-	-	-	-	-	XS1
23	-	√	√	√	-	-	-	-	-	-	XS1
24	-	-	√	√	√	-	-	-	-	-	XS1

Figure 66 is a graphical representation of the number of structures assessed and the deterioration mechanisms that have affected the specific structures. The information used to plot the graph was extracted from Table 19.

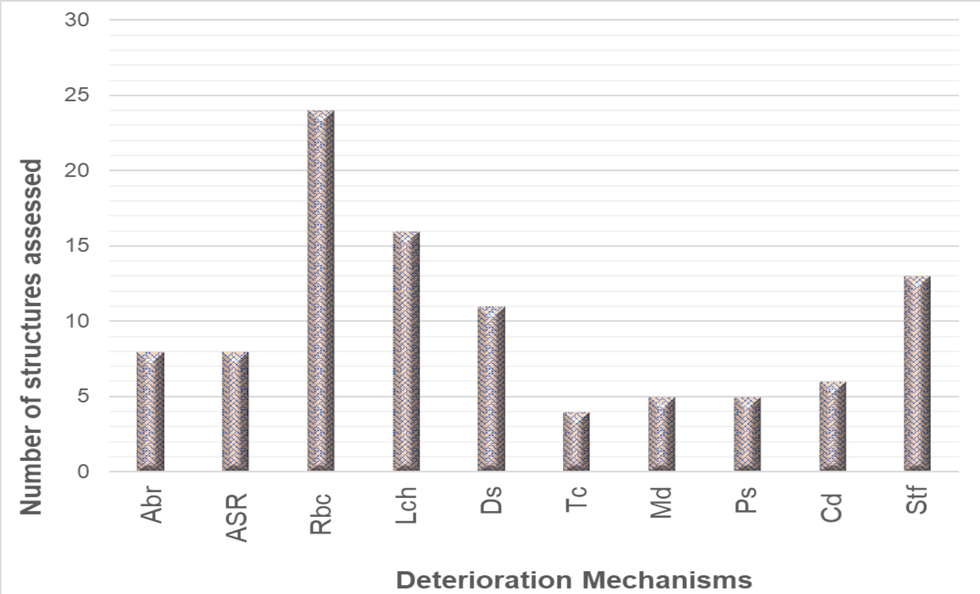


Figure 66: Number of structures showing specific deterioration mechanisms

Figure 66 shows that reinforcing bars corrosion has been observed on all the twenty-four structures assessed and mechanical damage is the least recorded deterioration mechanism.

3.3.1 Location of structures and the exposure class graph.

Figure 67 shows the number of structures assessed per province and the applicable exposure classes. Western Cape has the largest number of structures assessed. Eastern Cape and Western Cape provinces are located close to the sea which implies that all structures located in these two provinces are all exposed to very harsh environmental conditions as compared to the other provinces. Because of the harsh environmental conditions in the coastal areas and marine environment, it is expected that deterioration of structures is severe in such environments.

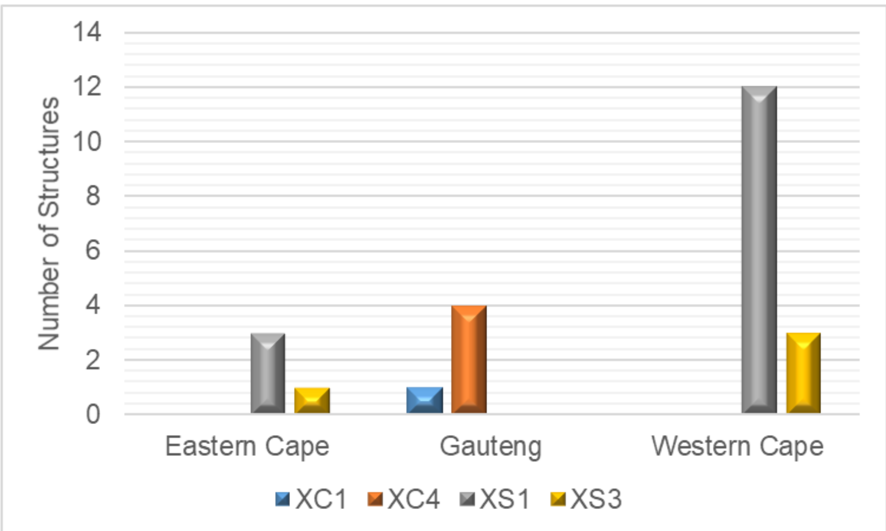


Figure 67: Number of structures per province

3.3.2 DER-U rating for structures assessed

Figure 67 has graphically shown the location of assessed structures and the exposure classes based on EN 206-1:2000. The observed defects have then been rated based on the DER-U rating system discussed in Section 2.4.

The structure types were further grouped such that buildings, bridges and retaining walls were analysed separately.

3.3.2.1 Bridges

The conventional inspection item numbering for the bridges as per COTO, (2013) and as indicated in Table 13 has been used for numbering the inspection items for bridges only. All defects observed were rated and are summarised in Table 19.

It is important to note that assessments and rating of bridge structures were done using the information available. Furthermore, the number of bridges assessed differs for all provinces considered. The defects on the bridges inspection items were rated and the inspection item priority indices (Ip) based on the draft TMH 22, the Manual for Road Asset Management prepared by COTO, (2013) were calculated.

As discussed in Section 2.4, the DER-U defect rating system has been adopted by the COTO for the visual assessment of road structures and is discussed in detail in the Draft TMH 19, (2016) series. The same rating system for the Degree, Extent, Relevance and Urgency has been adopted in this research for the bridges and the same principle was extended to buildings. The DER-U rating system is being implemented by several road authorities and has been adopted in this research as an attempt to ensure that the analysis is acceptable with a high level of confidence.

After rating the defects using the principles and the ratings as discussed in Section 2.4, Table 20 was developed to indicate the ratings scored for the observed defects with regards to D, E, R and U. Furthermore, the values for a parameter known as the Inspection Item priority index (Ip) calculated using equation 1, which incorporates the Degree, Extent and Relevancy ratings have also been included in Table 20. Ip is used to indicate the damage and severity of the defects observed.

Table 20: Bridges D.E.R-U ratings and the inspection item priority Indices (Ip)

			DETERIORATION MECHANISMS										
STR REF No.	**INSP ITEM**	**D.E.R-U / Ip**	**Abr**	**ASR**	**Rbc**	**Lch**	**Ds**	**Tc**	**Md**	**Ps**	**Cd**	**Stf**	**PR**
2	5	D.E.R-U	0	0	0	0	0	0	0	0	0	0	EC
		Ip	100	100	100	100	100	100	100	100	100	100	
	6	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	7	D.E.R-U	0	0	0	0	0	0	0	3.1.2-3	0	0	
		Ip	100	100	100	100	100	100	100	93	100	100	
	11	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	13	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	14	D.E.R-U	0	0	0	0	0	0	0	0	3.1.2-3	0	
		Ip	100	100	100	100	100	100	100	100	93	100	
	18	D.E.R-U	0	0	4.3.4-4	3.2.2-3	3.4.2-2	0	0	0	0	0	
		Ip	100	100	25	87	73	100	100	100	100	100	
	20	D.E.R-U	0	0	4.3.4-4	4.2.2-3	3.4.2-2	0	0	0	0	0	
		Ip	100	100	25	82	73	100	100	100	100	100	
4	5	D.E.R-U	0	0	0	0	0	0	0	0	0	0	EC
		Ip	100	100	100	100	100	100	100	100	100	100	
	6	D.E.R-U	0	0	0	0	0	0	0	0	3.1.2-2	0	
		Ip	100	100	100	100	100	100	100	100	93	100	
	7	D.E.R-U	0	2.1.2-2	0	0	3.2.2-2	0	2.1.1-1	0	0	0	

Table 20 continued…

STR REF No.	INSP ITEM	D.E.R-U / Ip	Abr	ASR	Rbc	Lch	Ds	Tc	Md	Ps	Cd	Stf	PR
			DETERIORATION MECHANISMS										
		Ip	100	96	100	100	87	100	98	100	100	100	
	11	D.E.R-U	1.1.1-R	0	0	0	0	0	0	0	0	0	
		Ip	99	100	100	100	100	100	100	100	100	100	
	13	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	14	D.E.R-U	0	0	3.2.3-3	2.1.2-1	0	0	0	0	0	0	
		Ip	100	100	76	96	100	100	100	100	100	100	
	18	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	20	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
6	5	D.E.R-U	2.3.2-2	0	0	0	0	0	0	0	0	0	GP
		Ip	87	100	100	100	100	100	100	100	100	100	
	6	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	7	D.E.R-U	0	2.1.1-3	0	0	3.2.3-3	1.2.1-1	0	0	0	0	
		Ip	100	98	100	100	76	98	100	100	100	100	
	11	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	13	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	14	D.E.R-U	1.2.1-1	0	0	2.2.1-1	1.1.2-2	0	0	0	3.1.1-2	0	
		Ip	98	100	100	91	98	100	100	100	98	100	
	18	D.E.R-U	0	0	0	0	0	0	0	0	0	0	

Table 20 continued…

STR REF No.	INSP ITEM	D.E.R-U / Ip	Abr	ASR	Rbc	Lch	Ds	Tc	Md	Ps	Cd	Stf	PR
			DETERIORATION MECHANISMS										
		Ip	100	100	100	100	100	100	100	100	100	100	
	20	D.E.R-U	0	0	3.1.3-3	0	2.1.2-1	0	0	0	3.1.2-3	0	
		Ip	100	100	88	100	96	100	100	100	93	100	
10	5	D.E.R-U	0	0	0	0	0	0	0	0	0	0	WC
		Ip	100	100	100	100	100	100	100	100	100	100	
	6	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	7	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	11	D.E.R-U	0	0	4.2.3-4	0	2.1.1-1	0	0	0	0	0	
		Ip	100	100	68	100	98	100	100	100	100	100	
	13	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	14	D.E.R-U	0	0	0	2.2.1-1	0	2.1.2-2	0	0	0	0	
		Ip	100	100	100	97	100	96	100	100	100	100	
	18	D.E.R-U	0	0	3.2.2-3	0	0	0	0	0	0	0	
		Ip	100	100	87	100	100	100	100	100	100	100	
	20	D.E.R-U	0	0	3.2.3-4	0	0	0	0	0	0	0	
		Ip	100	100	76	100	100	100	100	100	100	100	
14	5	D.E.R-U	3.3.2-2	0	0	0	0	0	0	0	0	0	WC
		Ip	80	100	100	100	100	100	100	100	100	100	
	6	D.E.R-U	0	0	3.2.3-4	2.2.1-2	0	0	0	0	0	3.2.3-4	
		Ip	100	100	76	97	100	100	100	100	100	87	
	7	D.E.R-U	0	0	0	0	0	0	0	0	0	0	

Table 20 continued…

STR REF No.	INSP ITEM	D.E.R-U / Ip	Abr	ASR	Rbc	Lch	Ds	Tc	Md	Ps	Cd	Stf	PR
			DETERIORATION MECHANISMS										
		Ip	100	100	100	100	100	100	100	100	100	100	
	11	D.E.R-U	0	0	2.1.2-3	0	0	0	0	0	0	0	
		Ip	100	100	96	100	100	100	100	100	100	100	
	13	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	14	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	18	D.E.R-U	0	0	2.3.2-3	2.2.1-1	0	0	0	0	0	0	
		Ip	100	100	87	91	100	100	100	100	100	100	
	20	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
17	5	D.E.R-U	0	0	0	0	0	0	0	0	0	0	WC
		Ip	100	100	100	100	100	100	100	100	100	100	
	6	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	7	D.E.R-U	0	0	0	0	2.1.1-1	0	0	0	0	0	
		Ip	100	100	100	100	98	100	100	100	100	100	
	11	D.E.R-U	0	0	4.2.3-4	0	0	0	4.1.3-4	0	0	0	
		Ip	100	100	68	100	100	100	84	100	100	100	
	13	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	14	D.E.R-U	0	0	0	0	0	2.1.2-2	0	0	1.1.1-0	0	
		Ip	100	100	100	100	100	96	100	100	99	100	
	18	D.E.R-U	0	0	0	2.2.1-1	0	0	0	0	0	0	

Table 20 continued…

STR REF No.	INSP ITEM	D.E.R-U / Ip	Abr	ASR	Rbc	Lch	Ds	Tc	Md	Ps	Cd	Stf	PR
			DETERIORATION MECHANISMS										
		Ip	100	100	100	97	100	100	100	100	100	100	
	20	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
20	5	D.E.R-U	0	0	3.3.2	4.3.3	0	0	0	0	0	2.1.2	WC
		Ip	100	100	80	36	100	100	100	100	100	96	
	6	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	7	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	11	D.E.R-U	0	0	2.1.1-1	0	0	0	0	0	0	0	
		Ip	100	100	98	100	100	100	100	100	100	100	
	13	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	14	D.E.R-U	0	0	3.3.2-2	3.3.1-2	0	0	0	0	0	0	
		Ip	100	100	80	80	100	100	100	100	100	100	
	18	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	20	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
21	5	D.E.R	0	0	0	0	0	0	0	2.2.2	0	3.2.2	WC
		Ip	100	100	100	100	100	100	100	91	100	87	
	6	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	7	D.E.R-U	0	0	0	0	0	0	0	0	0	0	

Table 20 continued…

STR REF No.	INSP ITEM	D.E.R-U / Ip	Abr	ASR	Rbc	Lch	Ds	Tc	Md	Ps	Cd	Stf	PR
			DETERIORATION MECHANISMS										
		Ip	100	100	100	100	100	100	100	100	100	100	
	11	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	13	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	14	D.E.R-U	0	0	4.3.4	2.2.1-1	0	0	0	0	2.2.2-3	3.2.3-2	
		Ip	100	100	25	97	100	100	100	100	91	84	
	18	D.E.R-U	0	0	0	2.2.2-2	0	0	2.1.2-3	0	0	0	
		Ip	100	100	100	91	100	100	97	100	100	100	
	20	D.E.R-U	0	0	3.2.3-4	2.2.1-1	0	0	0	0	0	0	
		Ip	100	100	76	97	100	100	100	100	100	100	
22	5	D.E.R	0	0	4.4.4	0	3.3.3	0	0	0	0	0	WC
		Ip	100	100	0	100	63	100	100	100	100	100	
	6	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	7	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	11	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	13	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	14	D.E.R-U	0	0	4.2.4-4	0	0	0	0	0	0	0	
		Ip	100	100	50	100	100	100	100	100	100	100	
	18	D.E.R-U	0	0	3.3.3-4	3.2.1-1	0	0	0	0	0	0	

Table 20 continued…

STR REF No.	INSP ITEM	D.E.R-U / Ip	Abr	ASR	Rbc	Lch	Ds	Tc	Md	Ps	Cd	Stf	PR
			DETERIORATION MECHANISMS										
		Ip	100	100	63	95	100	100	100	100	100	100	
	20	D.E.R-U	0	0	4.4.4-4	2.1.1-1	0	0	0	0	0	0	
		Ip	100	100	0	98	100	100	100	100	100	100	
23	5	D.E.R	0	4.2.3	4.3.3	3.3.3	0	0	0	0	0	0	WC
		Ip	100	68	51	63	100	100	100	100	100	100	
	6	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	7	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	11	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	13	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	14	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	18	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	20	D.E.R-U	0	3.2.3-4	4.4.4-4	3.3.2-2	0	0	0	0	0	0	
		Ip	100	76	0	80	100	100	100	100	100	100	

It is important to note that the maximum Ip is 100 and is assumed to indicate that the specific mechanism may have not been observed by visual inspection on a specific structure. A very low Ip is an indication that the structure has been affected severely by the specific mechanism.

The urgency rating on inspection items affected by reinforcement corrosion is very high as compared to the other defects especially on the beams and bridge decks as reinforcement corrosion compromises the structural integrity of bridge structure. Although mechanical damage was only observed on three bridges, one in the Eastern Cape and two in the Western Cape, the urgency rating is very low on the two cases but extreme on one case. This implies that it can be serious where it occurs despite its rare occurrence.

Two graphs have been generated using information from Table 20, which are Figure 68 and 69.

The total number of bridge structures showing specific defects is shown in Figure 69.

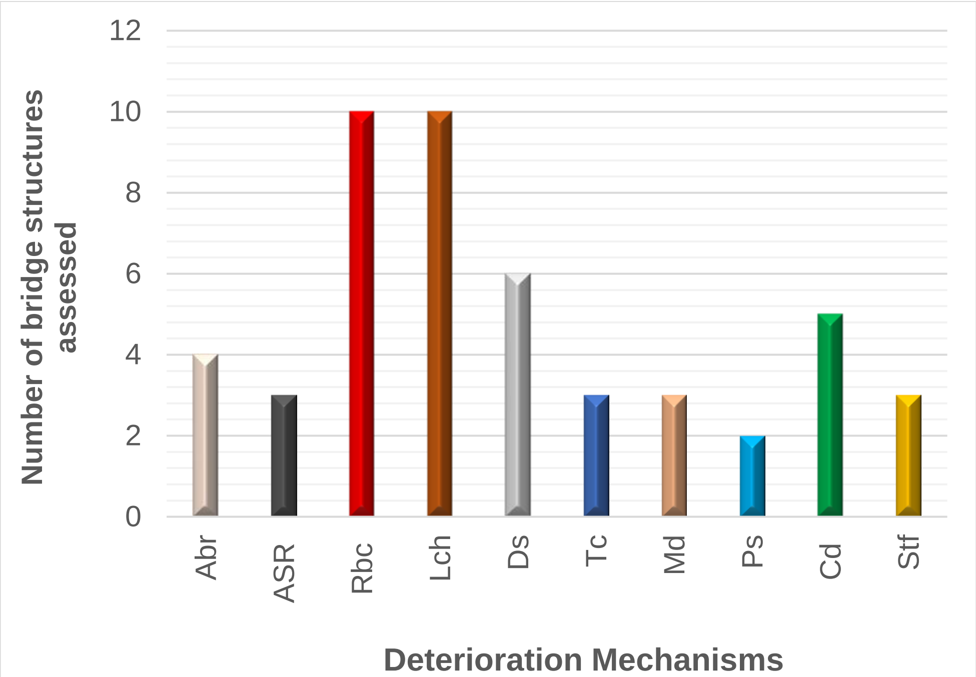


Figure 68: Number of bridges with specific defect observed

Figure 68 shows that amongst the ten mechanisms considered in this research in relation to the structures assessed; reinforcing bars corrosion and leaching have been identified as the most predominant mechanisms. This is because, all the ten bridge structures have been affected irrespective of the inspection items considered.

The total number of bridges with specific defects identified and grouped per province is as shown in Figure 69.

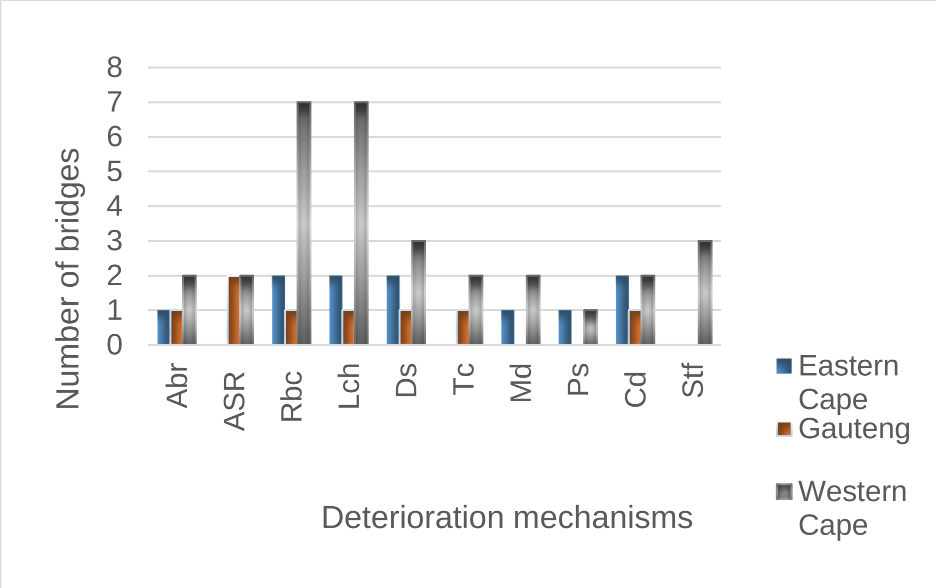


Figure 69: Number of bridges with specific defects per province

Figure 69 clearly shows that the Western Cape province has the highest number of bridge structures assessed and all the ten mechanisms considered in this research have been identified in the Western Cape; with reinforcement corrosion and leaching leading in terms of number of occurrences. Abrasion, mechanical damage and structural failure were not observed on the bridge structures located in the Gauteng province.

The priority condition indices for the inspection items on the bridges assessed have been determined by considering the defect with the highest relevancy rating as stipulated in draft TMH19, (2013). Summary of the priority condition indices for each inspection item is shown in Table 21.

Table 21: Condition Indices per inspection item (Ip)

Inspection Item	EC		GP	WC						
	Structure reference number									
	2	4	6	10	14	17	20	21	22	23
5	100	100	87	100	80	100	36	87	0	51
6	100	93	100	100	76	100	100	100	100	100
7	93	87	76	100	100	98	100	100	100	100
11	100	99	100	68	96	68	91	100	100	100
13	100	76	100	100	100	100	100	100	100	100
14	93	100	91	96	100	96	80	25	50	100
18	25	100	100	87	87	97	100	91	63	100
20	25	100	88	76	100	100	100	76	0	0

Table 21 clearly shows that the Ip of 0 and 25 which reflect that the damage is severe have been encountered on bridge structures in the Western Cape and Eastern Cape provinces. Western Cape and Eastern Cape provinces are in the coastal areas. Gauteng province has the lowest Ip of 76 which reflects that the severity is medium. However, it should be noted that there is only one bridge structure assessed in the Gauteng region.

3.3.2.2 Buildings

DER-U rating system was developed for bridges and is not necessarily directly applicable to building structures. However, since no method for buildings exists, the method was considered an appropriate tool for a preliminary evaluation of the severity and significance of damage observed on buildings.

The inspection item numbering for buildings which was developed by the author as indicated in Table 22 was used for numbering the inspection items for buildings only. The inspection items were derived from the items assessed and information available in the assessment reports.

Table 22 : Inspection item numbering for buildings

Inspection item number	description		Inspection item number	description		Inspection item number	description
1	Foundation		3	Wall		5	Deck
2	Column		4	Beam			

All defects observed on the assessed buildings by the scholars were rated and are summarised in Table 23. It is important to note that all scholars were unable to assess the foundations of all the buildings assessed, hence the foundation inspection items were not included in Table 23.

Table 23 : Buildings DER-U rating

STR REF No.	INSP ITEM	D.E.R-U	DETERIORATION MECHANISMS										
		Ip	Abr	ASR	Rbc	Lch	Ds	Tc	Md	Ps	Cd	Stf	PR
1	2	D.E.R-U	0	0	4.4.4-4	0	0	0	0	0	0	4.2.4-4	EC
		Ip	100	100	0	100	100	100	100	100	100	50	
	3	D.E.R-U	0	0	4.3.4-4	0	0	0	0	0	0	0	
		Ip	100	100	25	100	100	100	100	100	100	100	
	4	D.E.R-U	0	0	4.3.4-4	0	0	0	0	0	0	0	
		Ip	100	100	25	100	100	100	100	100	100	100	
	5	D.E.R-U	0	0	4.4.4-4	0	0	0	0	0	0	0	
		Ip	100	100	0	100	100	100	100	100	100	100	
3	2	D.E.R-U	3.3.2-3	0	4.3.4-4	4.3.2-1	0	0	0	0	0	0	EC
		Ip	80	100	25	73	100	100	100	100	100	100	
	3	D.E.R-U	0	0	2.1.2-2	0	0	0	0	0	0	0	
		Ip	100	100	96	100	100	100	100	100	100	100	
	4	D.E.R-U	0	0	2.1.2-2	0	0	0	0	0	0	0	
		Ip	100	100	96	100	100	100	100	100	100	100	
	5	D.E.R-U	0	0	4.3.3-4	0	0	0	0	0	0	0	
		Ip	100	100	51	100	100	100	100	100	100	100	
5	2	D.E.R-U	0	0	3.3.3-3	3.1.2-2	0	0	0	3.3.2-1	0	2.1.2-2	GP
		Ip	100	100	63	93	100	100	100	80	100	96	
	3	D.E.R-U	0	3.2.3-4	0	0	0	2.1.2-2	0	0	0	0	
		Ip	100	76	100	100	100	96	100	100	100	100	
	4	D.E.R-U	0	0	3.2.3-3	0	0	0	0	0	0	0	
		Ip	100	100	76	100	100	100	100	100	100	100	

Table 23 continued…

STR REF No.	INSP ITEM	D.E.R-U / Ip	Abr	ASR	Rbc	Lch	Ds	Tc	Md	Ps	Cd	Stf	PR
	5	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
7	2	D.E.R-U	0	0	3.2.3-3	0	3.3.3-3	0	0	3.2.3-3	0	0	GP
		Ip	100	100	76	100	63	100	100	76	100	100	
	3	D.E.R-U	0	0	0	0	3.3.3	2.2.2	0	3.2.2-2	0	0	
		Ip	100	100	100	100	63	91	100	87	100	100	
	4	D.E.R-U	0	0	2.1.2-2	0	3.2.3-2	0	0	0	0	0	
		Ip	100	100	96	100	76	100	100	100	100	100	
	5	D.E.R-U	0	0	4.3.4-4	0	0	0	0	0	0	0	
		Ip	100	100	25	100	100	100	100	100	100	100	
8	2	D.E.R-U	0	2.1.2-2	2.3.2-2	0	2.2.2-2	0	0	0	2.2.2-2	2.1.2-2	GP
		Ip	100	96	87	100	91	100	100	100	91	96	
	3	D.E.R-U	0	3.2.3-4	0	2.1.2-2	0	0	0	0	0	0	
		Ip	100	76	100	96	100	100	100	100	100	100	
	4	D.E.R-U	0	0	2.2.2-2	0	0	0	0	0	0	0	
		Ip	100	100	91	100	100	100	100	100	100	100	
	5	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
9	2	D.E.R-U	0	0	2.1.2-2	0	0	0	0	0	0	0	GP
		Ip	100	100	96	100	100	100	100	100	100	100	
	3	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	4	D.E.R-U	0	0	3.2.2-3	0	0	0	0	0	0	0	
		Ip	100	100	87	100	100	100	100	100	100	100	
	5	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	

Table 23 continued…

STR REF No.	INSP ITEM	D.E.R-U / Ip	Abr	ASR	Rbc	Lch	Ds	Tc	Md	Ps	Cd	Stf	PR
			DETERIORATION MECHANISMS										
13	2	D.E.R-U	0	4.3.4	4.3.3-3	3.1.2	3.2.3	0	1.1.1-R	0	0	0	WC
		Ip	100	25	63	93	76	100	99	100	100	100	
	3	D.E.R-U	0	0	3.3.3	3.1.2	0	0	0	0	0	0	
		Ip	100	100	63	93	100	100	100	100	100	100	
	4	D.E.R-U	0	4.3.4	3.3.3	0	3.2.3	0	0	0	0	0	
		Ip	100	25	63	100	76	100	100	100	100	100	
	5	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
15	2	D.E.R-U	1.2.1-R	0	4.4.4-4	0	0	0	0	0	0	0	WC
		Ip	98	100	0	100	100	100	100	100	100	100	
	3	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	4	D.E.R-U	0	0	4.4.4-4	0	0	0	0	0	0	0	
		Ip	100	100	0	100	100	100	100	100	100	100	
	5	D.E.R-U	0	0	4.3.4-4	0	0	0	0	0	0	0	
		Ip	100	100	25	100	100	100	100	100	100	100	
16	2	D.E.R-U	0	0	0	0	0	0	1.1.1-R	0	0	0	WC
		Ip	100	100	100	100	100	100	99	100	100	100	
	3	D.E.R-U	0	0	0	0	0	2.2.2	0	0	0	2.1.2	
		Ip	100	100	100	100	100	91	100	100	100	96	
	4	D.E.R-U	0	0	4.3.4-4	0	0	0	0	0	0	0	
		Ip	100	100	25	100	100	100	100	100	100	100	
	5	D.E.R-U	0	0	3.2.3-4	0	0	0	0	0	0	0	
		Ip	100	100	76	100	100	100	100	100	100	100	
18	2	D.E.R-U	0	3.2.2	4.2.3	3.2.2	4.3.2	0	0	0	0	0	WC

Table 23 continued…

STR REF No.	INSP ITEM	D.E.R-U / Ip	Abr	ASR	Rbc	Lch	Ds	Tc	Md	Ps	Cd	Stf	PR
		Ip	100	87	68	87	82	100	100	100	100	100	
	3	D.E.R-U	0	0	0	3.2.2	0	0	0	2.3.1	0	0	
		Ip	100	100	100	87	100	100	100	95	100	100	
	4	D.E.R-U	0	0	4.2.3	3.2.2	4.3.2	0	0	0	0	0	
		Ip	100	100	68	87	73	100	100	100	100	100	
	5	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
19	2	D.E.R-U	0	0	2.1.2-2	3.2.2-2	0	0	0	0	0	0	WC
		Ip	100	100	96	87	100	100	100	100	100	100	
	3	D.E.R-U	0	0	4.3.3-4	3.2.2-2	0	0	0	0	0	0	
		Ip	100	100	51	87	100	100	100	100	100	100	
	4	D.E.R-U	0	0	4.3.3-4	3.2.2-3	0	0	0	0	0	0	
		Ip	100	100	51	87	100	100	100	100	100	100	
	5	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
24	2	D.E.R-U	0	0	4.2.3-4	3.1.2-2	4.2.3-3	0	0	0	0	0	WC
		Ip	100	100	68	93	68	100	100	100	100	100	
	3	D.E.R-U	0	0	3.3.3-3	3.2.1-1	4.2.3-3	0	0	0	0	0	
		Ip	100	100	63	76	68	100	100	100	100	100	
	4	D.E.R-U	0	0	4.3.3-4	3.2.2-2	4.2.3-3	0	0	0	0	0	
		Ip	100	100	51	87	68	100	100	100	100	100	
	5	D.E.R-U	0	0	3.3.3-4	0	0	0	0	4.1.3-4	0	0	
		Ip	100	100	63	100	100	100	100	84	100	100	

The urgency rating on inspection items affected by reinforcement corrosion is very high as compared to the other defects. Even though ASR damage was only observed on the building in Western Cape and the urgency rating is very high, this indicates that it can be serious where it occurs despite its rare occurrence.

The number of buildings assessed and showing specific defects are shown in Figure 70.

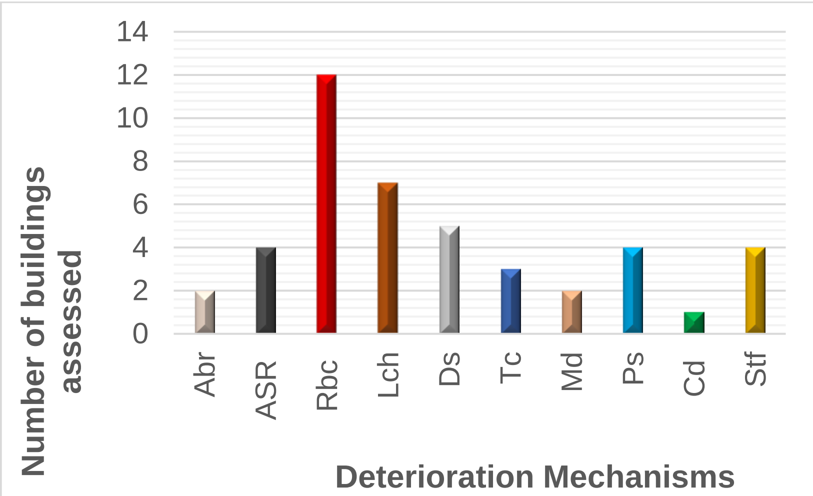


Figure 70: Number of all buildings with specific defects observed

Figure 70 shows that for the ten mechanisms considered in this research and in relation to the buildings assessed; reinforcing bars corrosion has been identified as the most predominant mechanism as all the twelve building structures have been affected irrespective of the inspection items considered.

The buildings with specific defects were grouped per province, where the structures were located and are shown in Figure 71.

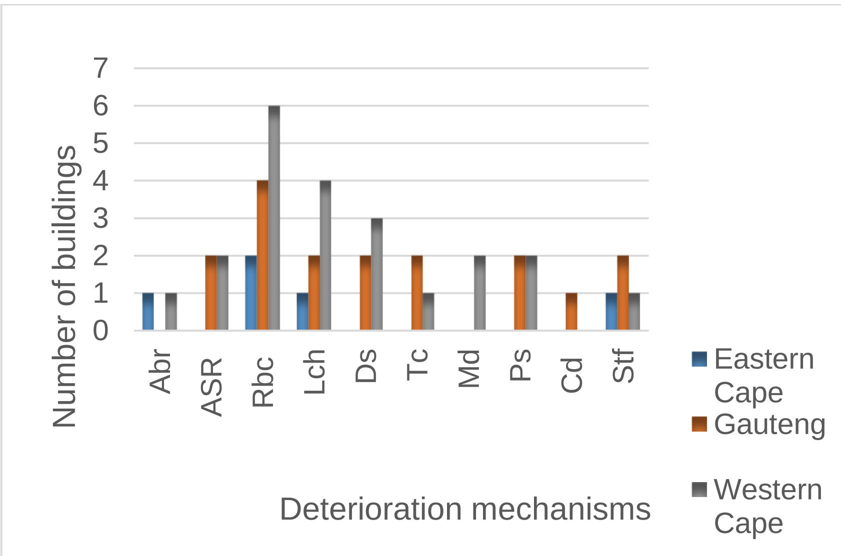


Figure 71: Number of buildings with specific defects grouped per province

Figure 71 shows that Western Cape has the highest number of building structures assessed and only the construction defect was not observed in the Western Cape; with reinforcement corrosion leading in terms of number of occurrences.

The priority condition indices for the inspection items on the buildings assessed have been determined by considering the defect with the highest relevancy rating as stipulated in draft TMH19, (2013). Summary of the priority condition indices for each inspection item is shown in Table 24.

Table 24: Condition indices per inspection item

Inspection Item	**EC**		**GP**				**WC**					
	Structure reference number											
	1	**3**	**5**	**7**	**8**	**9**	**13**	**15**	**16**	**18**	**19**	**24**
2	0	25	63	63	87	96	25	0	99	68	87	68
3	25	96	76	63	76	100	63	100	91	87	51	63
4	25	96	76	76	91	87	25	0	25	68	51	51
5	0	51	100	25	100	100	100	25	76	100	100	63

Table 24 clearly shows that the Ip(s) of 0 which reflect(s) that the damage is severe have been encountered on building structures in the Western Cape and Eastern Cape provinces. Although an Ip of 25 which is also considered very low has been calculated in the Gauteng, Western Cape and Eastern Cape provinces, the frequency is less in the Gauteng province.

3.3.2.3 Retaining walls

The conventional inspection item numbering for the retaining walls as per COTO, (2013) and as indicated in Table 14 have been used for numbering the inspection items for retaining walls only. All defects observed were rated and are summarised in Table 25.

Table 25: DER-U rating for retaining walls

			DETERIORATION MECHANISMS										
STR REF No.	INSP ITEM	D.E.R-U / Ip	Abr	ASR	Rbc	Lch	Ds	Tc	Md	Ps	Cd	Stf	PR
11	1	D.E.R-U	0	0	0	0	0	0	0	0	0	0	WC
		Ip	100	100	100	100	100	100	100	100	100	100	
	2	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	3	D.E.R-U	0	0	4.3.2-2	0	0	0	0	0	0	0	
		Ip	100	100	73	100	100	100	100	100	100	100	
	4	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	5	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	6	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
12	1	D.E.R-U	0	0	0	0	0	0	0	0	0	0	WC
		Ip	100	100	100	100	100	100	100	100	100	100	
	2	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	3	D.E.R-U	2.3.2-1	0	3.3.2-2	0	4.2.2-2	0	0	0	2.3.1-1	0	
		Ip	87	100	63	100	82	100	100	100	87	100	
	4	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	5	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	
	6	D.E.R-U	0	0	0	0	0	0	0	0	0	0	
		Ip	100	100	100	100	100	100	100	100	100	100	

Although the degree ratings in most cases are high, the urgency ratings are low as the relevancy of the defects on the retaining walls is low.

The number of retaining walls assessed and showing specific defects are shown in Figure 72.

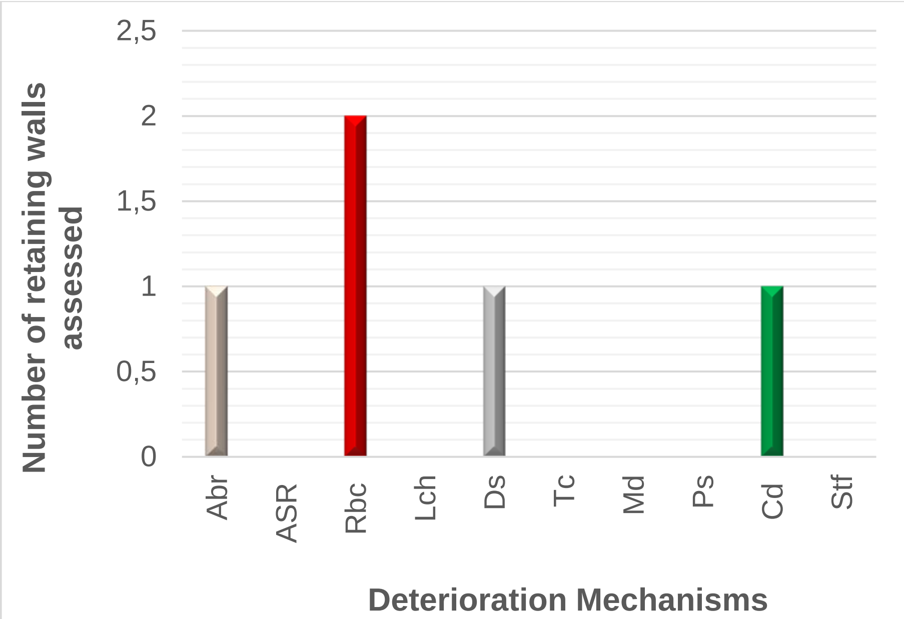


Figure 72 : Number of all retaining walls with specific defects observed

Figure 72 shows that reinforcing bars corrosion has been identified as the most predominate mechanism as both retaining walls have been affected irrespective of the inspection item considered.

The retaining walls were both in the Western Cape province as shown in Figure 74.

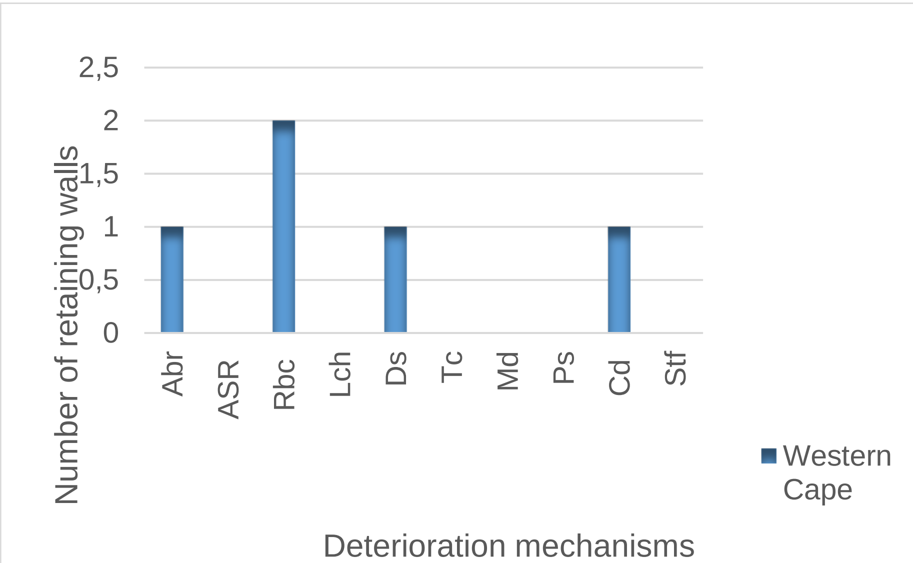


Figure 73 : Number of buildings with specific defects grouped per province

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The priority condition indices for the inspection items on the retaining walls assessed have been determined by considering the defect with the highest relevancy rating as stipulated in draft TMH19, (2013). Summary of the priority condition indices for each inspection item is shown in Table 26.

Table 26 : Condition indices per inspection item

	WC	
Inspection Item	**Structure reference number**	
	11	**12**
1	100	100
2	100	100
3	73	63
4	100	100
5	100	100
6	100	100

The inspection item condition indices are relatively low on affected areas. Table 26 clearly shows that the inspection item 3 have the lowest Ip(s) on both retaining walls and Ip(s) of 100 on all other inspection items, which reflects that the defects were only observed on the walls. It is likely that the retaining wall inspection items with Ip of 100 were not inspected and recorded, and therefore the ratings underestimated.

The calculation of inspection item indices has assisted in identifying inspection items which are severely affected by various deterioration mechanisms. The determination of urgency rating has indicated the structures with critical inspection items based on the available data. Furthermore, generalisations could not have been possible due to limited available data and smaller samples which results in an undesirable wider error margin.

However, the limitations have been discussed under Section 1.5 of this research. The recommendations for future research and to develop a more useful guide has been discussed under Section.

CHAPTER 4

4. Summary and conclusion

The objective of the dissertation was to review literature on the predominant causes of deterioration of concrete structures. It also aimed at analysing the condition assessment results done by other scholars of which the visual assessment findings have been correlated to the common deterioration mechanisms. The rating of the defects was done using the DER-U rating system for bridges and retaining walls. Furthermore, the rating system for buildings was developed exploiting the bridges rating system, which is an available method, as there is no available established rating system for buildings. The trends of concrete deterioration mechanisms were established using only the limited number of structures visual assessed with emphasis on the South African context. It is important to note that the literature review and case studies were analysed used the available data and available rating methods, taking note of all the limitations.

In conclusion, the literature review has indicated that reinforced concrete deteriorates due to, but not limited to the following;

- the ingress of deleterious substances such as chlorides and carbon dioxide which react with the cement matrix,
- reinforcing bars corrosion,
- mechanical effects,
- physical effects,
- structural damages,
- poor construction practices, or
- due to a combination of these factors.

The ultimate effect of concrete deterioration is the reduction of the expected service lives of the concrete structures. The severity of damages is also dependent on the age. Furthermore, it was also noted that the severity of the damage increased with age of a structure. As an example, the 85-year-old building in the Eastern Cape showed major damages as compared to 15 – 30-year-old structures in the same vicinity. It has been noted that the rate of deterioration is exacerbated by harsh environmental conditions, which implies that the type of mechanism and the rate of deterioration is dependent on the location of the structures.

The following trends have been established from the preliminary evaluation using the available assessment reports;

- Reinforcing bars corrosion has been identified as the most common deterioration mechanism aggravated by inadequate cover and honeycombs. Corrosion was more pronounced in the marine and coastal environment where the Ip(s) were relatively low on the buildings, bridges and retaining walls assessed. The same conclusion is also as researched and confirmed by (Otieno et al., 2015).
- Abrasion was mainly observed on the concrete structure inspection items situated in the river stream and tidal zones.
- Drying shrinkage cracks have been manly observed in the Gauteng province which may be related to very high temperatures in summer and most probably construction was done in the hot seasons and curing was compromised.
- Severe leaching has been observed on all structures exposed to humid environments.
- Urgency rating is affected by the relevancy which depends on the inspection item position.
- ASR damage observed on a building in Western Cape had a high urgency rating, which implies that it can be serious where it occurs despite its rare occurrence, although petrographic analysis as an additional investigation was required to ascertain ASR

Recommendations

i) Practical recommendations

- Surface hardness is of paramount importance for such structures to reduce abrasion.
- Material testing of aggregates to reduce the use of reactive aggregates thereby minimising the risk of ASR.
- Strict monitoring during construction to minimise construction defect.
- Durability Index approach as recommended by Beushausen and Alexander (2008) must be implemented to reduce the maintenance costs of the structures for the entire service lives of the concrete structures.

ii) Future research

- Full scale condition assessments to ascertain the damage mechanisms as observed defects must not be based on the visual assessment in isolation, but should be complemented by other assessment techniques so that informed maintenance strategies can be developed.
- Increased size of samples of similar structures from different environmental conditions is required for accurate trends determination.
- Large and uniform samples from all provinces are required in order to establish trends of deterioration mechanisms across the entire country.
- The proposed inspection items for buildings deliberated on, so that a conventional numbering can be used for all buildings inspections.

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