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An Investigation of the Relationships between Inventory and Inspection Data of RC Bridges and RC Culverts in the Western Cape Province

Submitted in partial fulfillment of the requirements for the degree of

Master of Science in Civil Engineering

By

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Declaration

This dissertation is being submitted in partial fulfilment for the Degree of Master of Science in Civil Engineering at the University of Cape Town. I know that meaning of plagiarism and declare that all the work in this document, save for that which is properly acknowledged, is my own. I also affirm that this work has not been submitted in this or any other University for examination or any other purposes.

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Thuthukile Dumile Mbanjwa
October, 2014

To my loving father, Thulani Donald Mbanjwa, your strength and humility have been an inspiration to us all.

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Abstract

Bridges and Culverts form critical links within transportation networks. They are costly long term infrastructure investments and are essential to a country's economic functioning and society's everyday lives. They are prone to deterioration due to unfavourable chemical, mechanical and physical consequences. Nonetheless, they are expected to remain safe and functional for the duration of their design life; thus Bridge Management Systems (BMSs) have been fundamental to maintaining and preserving these. However, the data contained in BMSs have mainly been used for structural prioritization. Hence, the purpose of this study was to utilize the Struman Bridge Management System database to investigate the relationships that exist between the inventory and inspection data of the Reinforced Concrete (RC) bridges and culverts within the Western Cape Province.

The study involved conducting data mining activities to simplify the data and extract useful information. These activities encompassed investigating structural defects, their predominance and spatial distributions (in terms of district municipalities) as well as their relationships with inventory data such as structure type, age and span length/width. STATA 2011 logit models were used to investigate whether the relationships were statistically significant and to determine odds ratios of several 'events'. In addition to this, the average Condition Indices (CIs), numerical ratings based on the network structures conditions and integrity, were also investigated to assess the condition of the structures relative to the inventory data.

The results indicated that RC bridges and culverts had similar defects and consequently similar forms of deterioration. Several deterioration mechanisms and causes of defects were identified. These included chloride ingress and carbonation, traffic loading, undermining, inadequate routine maintenance and poor construction practises and techniques. There were no specific spatial distributions identified along the coast or inland as the prevalence of the predominant defects varied for the different district municipalities. Also, location (in terms of district municipalities) gave more meaningful results when considered with the mean age and the standard deviation of the age of the RC structures in that location. There were no statistically significant relationships identified between the predominant defects and the various structure types. However, findings from data mining suggested that the most dominant structure types had higher percentages of RC structures with defects and were in the worst condition. Moreover, there was a general increase in RC structures with predominant defects with increasing age categories as well as increasing span length/width. Also, the average structural condition showed a general decrease with age and span length/width. Nonetheless, the average CIs of the RC structures suggested that their average overall conditions were good.

Lastly, the study makes recommendations pertaining to the collection of BMS data, the analyses BMS software should be able to execute, the parameters CI indicators should take into consideration as well as further research that could be a continuation of the study.

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List of Acronyms

AAR	Alkali-Aggregate Reaction
AASTHO	American Association of State Highway and Transportation Officials
ASR	Alkali-Silica Reaction
BM	Bridge Management
BMS	Bridge Management System
CBD	Central Business District
CCAA	Cement Concrete Aggregates Australia
CI	Condition Index
COSMOS	Computerized System for the Maintenance of Structures
CSIR	Council for Scientific and Industrial Research
DM	District Municipality
EUL	Estimated Useful Life
FHWA	Federal Highway Administration
FI	Functional Index
HiSMIS	Highway Structures Management Information System
HMS	Highway Management System
IM	Infrastructure Management
LCC	Life Cycle Costing
LCCA	Life Cycle Costing Analysis
LRA	Logistic Regression Analysis
NBIS	National Bridge Inspection Standard
NPV	Net Present Value
OPI	Overall Priority Index
PGWC	Provincial Government of the Western Cape
PI	Priority Index
PVA	Present Value Analysis

RC	Reinforced Concrete
RMS	Roads Management System
RTMS	Roads Transportation Management System
SANRAL	South African National Roads Authority Limited
SIA	Structure Inventory and Appraisal
SMIS	Structures Management Information System
SSI	Stewart Scott International
TMS	Traffic Management System
TOMS	Traffic Observation Management System
WLCC	Whole Life Cycle Costing

Chapter 1

1 Introduction

1.1 Background to Study

Bridges and culverts form costly critical components of transport networks and society has always relied on transportation for survival. History shows that the earliest bridges were made from numerous objects in nature such as fallen trees, vines and ropes (Tang, 2007). They were initially developed as a means of transportation for people when walking or on horseback and with the evolution of technologies they were later redeveloped for mechanized transportation of goods and of people. Thus the performance of such public infrastructure should be of great concern as transportation systems not only have an impact on the country's economy, they also affect society's everyday lives (Adhikari *et al*., 2013).

Bridges are prone to defects and deterioration as a result of chemical, mechanical and physical consequences. An example of the latter is when the legal axle load was increased from 8.2 to 9 tonnes in South Africa in 1996 (Nordengen and De Fleuriot, 1998); deterioration of transportation infrastructure was expected to increase. In addition to this, society's expectations for infrastructure managers to meet new demands (such as higher axle loads; higher speeds; safety; less disturbances; reduced maintenance costs and increased availability and reliability of network structures) continue to steadily increase ((Bień *et al.,* 2007), (Arts *et al*., 2008) and (Schraven *et al*., 2011)). Nonetheless, Infrastructure Management (IM) is a basic service delivery to society and access to safe and reliable road networks is essential for improving the quality of lives. For these reasons as well as the reason of defects in bridges being sometimes difficult to identify and the consequences of failure being disastrous, it is apparent that an effective approach to bridge IM is necessary and should not be delayed (Nordengen, 2012).

Schraven *et al.* (2011) suggest that in the past, public agencies across the world were able to allocate large budgets for maintenance, renovations and reconstruction of infrastructure. However, in recent years, many of the agencies were challenged with budgetary constraints. Consequently, the quantity of structures receiving preventative and routine maintenance decreased and the need for their repair and rehabilitation became more pronounced. An example of this are the significant challenges pertaining to limited available funds for IM versus the large quantity of bridges and culverts requiring remedial work activities, experienced by South Africa prior to year 2000. Nordengen and De Fleuriot (1998) indicate that the constraint of limited funds for construction and maintenance led to more attention being given to the preservation of infrastructure in urgent need of remedial work activities as opposed to routine maintenance for all infrastructures.

IM is an approach that aims to achieve more value with fewer resources (Schraven *et al.*, 2011). Moreover, Moon *et al.* (2009) discuss the significance of IM and in doing so strongly believe the transition to the paradigm of transportation IM practices will serve as a tool to improve matters of budgetary shortfalls, performance of transportation infrastructure and the increased demand for accountability of transportation agencies. It may be noted that there are various methods of transportation IM, however this study will focus only on that of Bridge Management (BM) through the use of Bridge Management Systems (BMSs), as the investigation conducted was solely on bridges and culverts.

BMSs have long been in practice; evidence of this is the development of the National Bridge Inspection Standards (NBIS), a legislative requirement, as a response to the collapse of the Silver Bridge across the Ohio River in Virginia in the United States in 1967. The NBIS were implemented in the early 1970s and made it mandatory that all public bridges of the Federal Highway Administration (FHWA) had a Structure Inventory and Appraisal (SIA) conducted. The legislation formed the basis of formalised bridge inspections and maintenance in the United States and has since undergone further development (Small *et al.*, 1999). Subsequently, BMSs became standardised practices globally and are consistently undergoing technological advances and improvements to enhance decision making related to resource allocation (Yang *et al.*, 2011).

According to Nordengen and De Fleuriot (1998), the Struman Bridge Management System (BMS) was developed and implemented by the Division of Roads and Transport Technology of the Council for Scientific and Industrial Research (CSIR) together with Royal Haskoning DHV, previously Stewart Scott International (SSI) in 1995. Many of the road and rail authorities within Southern Africa did not have BMSs in place at the time and to this end adopted and employed the Struman Bridge Management System which provided a basis for bridge maintenance and rehabilitation. The system utilizes a prioritization algorithm to rank structures in order of need for rehabilitation and repair. Its particular focus is that of observed defects of the various bridge and culvert elements as opposed to the element's overall condition (Nordengen and Nell, 2005).

Included in the Southern African road and rail authorities to have adopted the Struman BMS in 2000 was the Department of Transport and Public Works of the Provincial Government of the Western Cape (PGWC); the study area. The PGWC is accountable for the management of 16 000 km of road. The network comprises 6 000 km of paved road, 10 000 km of unpaved road and is also inclusive of approximately 2 400 bridges and culverts distributed as shown in *Figure 1-1* (Nordengen and Nell, 2005).

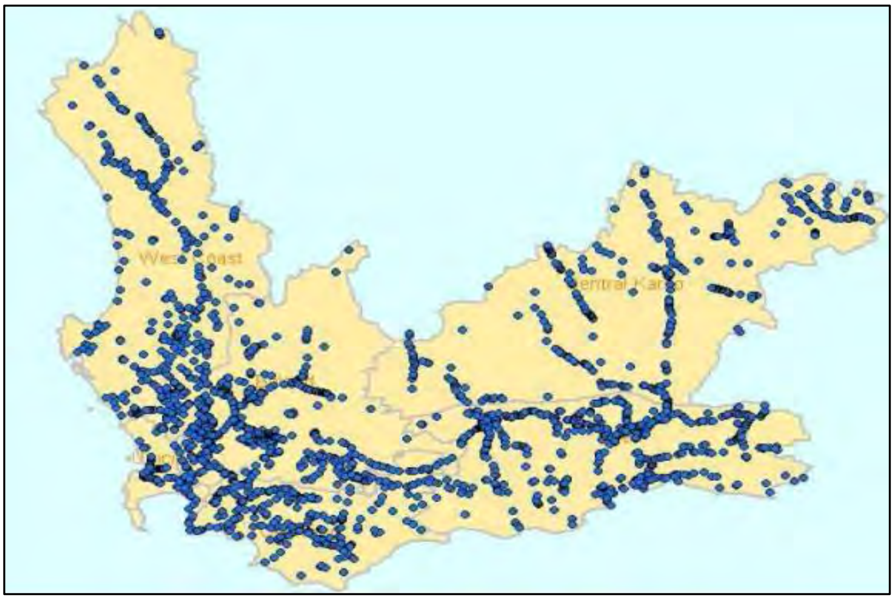

Figure 1-1: Map showing all Bridges and Culverts in the Western Cape Struman BMS Database

Source: Nordengen and Nell (2005)

1.1.1 Description of Study Area

The Province of the Western Cape is located on the south-western part of South Africa. It is the southernmost region of the African continent and is approximately 129 462 km^2 – in area. A large extent of the province lies within the Cape Fold Belt (an array of sandstone folded mountains) and has generally very fertile alluvial loamy to clay soils. The province's far interior falls within the Karoo Basin, (a semi-desert area containing an extensive and geographically recent arrangement of sedimentary and igneous rocks) a rather hilly zone. The south-east coastal region is bordered by the Indian Ocean and the western coastal region is bordered by the Atlantic Ocean. The influence the warm Agulhas Ocean current has on the east lessens the cold fonts of the cold Benguela Ocean current on the west making the climate vary. The majority of the province has a Mediterranean climate, the northern and western areas are semi-arid and the south coast has a maritime climate. Thus the topography and vegetation is extremely diverse (Gumbi and van Weele, 2013).

Figure 1-2 utilizes Thornthawaite's Moisture Index, briefly explained in *Table 1-1,* to provide an indication of the various climatic zones of South Africa, including the Western Cape Province (Committee of Transport Officials, 2013b).

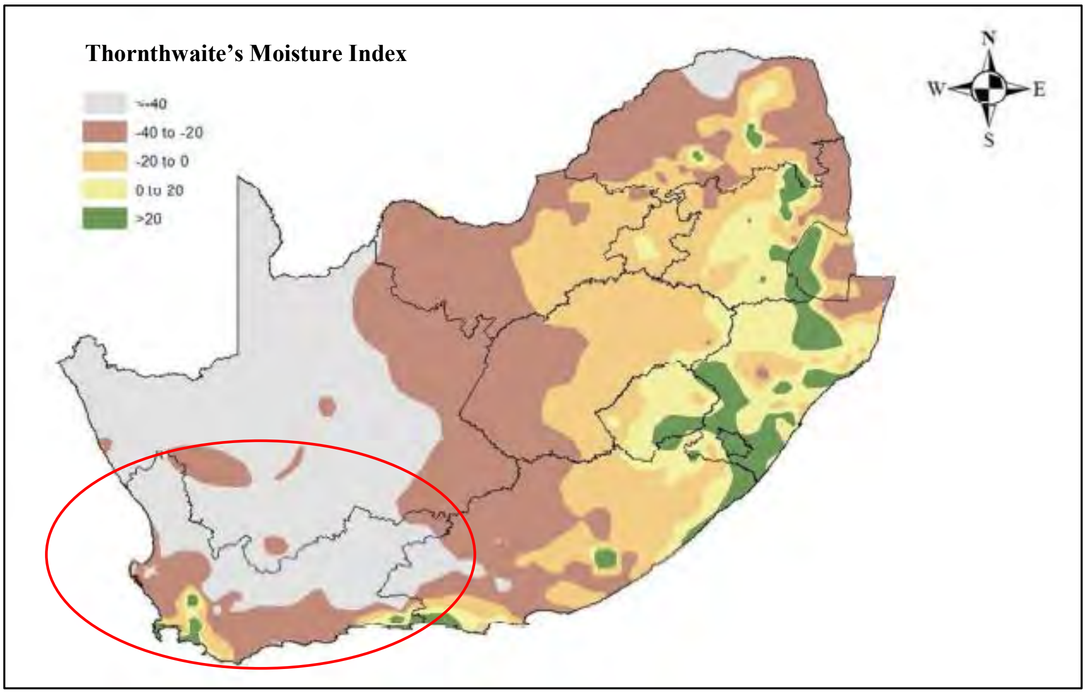

Figure 1-2: Thornthwaite's Moisture Index Diagram for South Africa

Source: Committee of Transport Officials (2013b)

Table 1-1: Thornthwaite's Moisture Index Range for each Climatic Zone of South Africa

Minimum Moisture Index	Maximum Moisture Index	Climatic Zone	Description
N/A	< - 40	Arid	Very low rainfall and high evaporation
- 40	- 20	Semi-arid	Low rainfall
- 20	0	Dry sub humid	Moderate rainfall or strongly seasonal rainfall
0	20	Moist sub humid	
> 20	N/A	Humid	Moderate, warm seasonal rainfall

The Western Cape consists of one metropolitan municipality and five district municipalities. The district municipalities are further sub-divided into 24 local municipalities, so as to ease the governance and running of the province. *Figure 1-3* illustrates a map of the Province of the Western Cape and the various local as well as the metropolitan and district municipalities listed in *Table 1-2*.

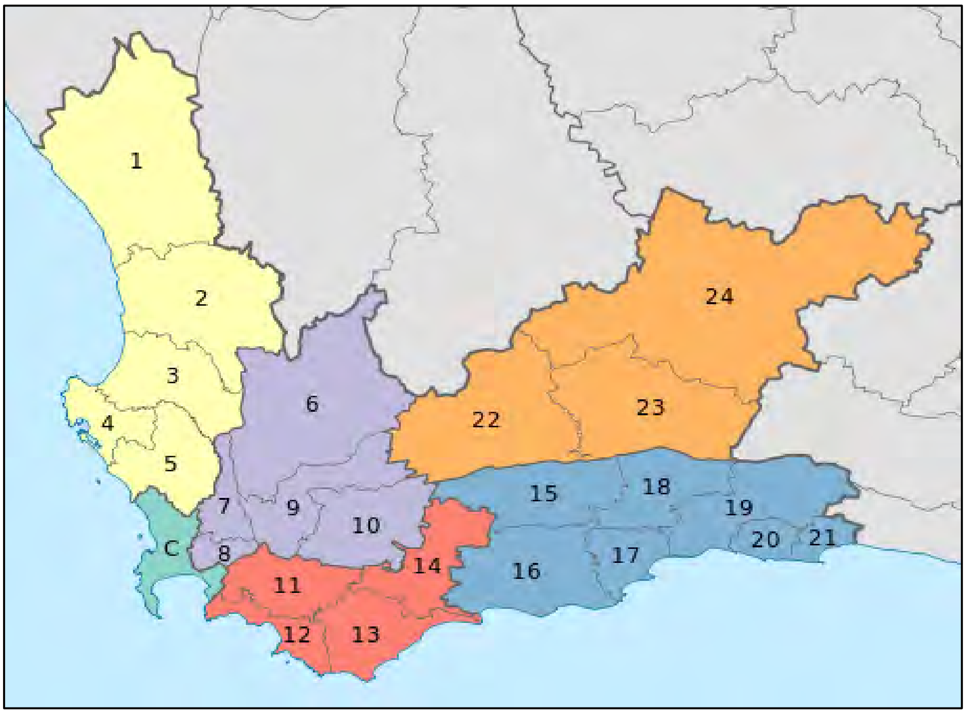

Figure 1-3: Map Showing the Western Cape Province of South Africa including the Divisions of Municipalities

Source: http://en.wikipedia.org/wiki/List_of_municipalities_in_the_Western_Cape

[Accessed 2014, March 12]

Table 1-2: List of Metropolitan and District Municipalities within the Western Cape Province

Key	Name of Municipality	Code	Seat	Area (km^2)
	City of Cape Town Metropolitan Municipality	CPT	Cape Town	2 460
	West Coast District Municipality	DC1	Moorreesburg	31 104
	Cape Winelands District Municipality	DC2	Worcester	22 309
	Overberg District Municipality	DC3	Bredasdorp	11 405
	Eden District Municipality	DC4	George	23 331
	Central Karoo District Municipality	DC5	Beaufort West	38 854

Source: http://www.localgovernment.co.za/provinces/view/9/western-cape

[Accessed 2014, March 12]

1.2 Problem Statement and Significance

The PGWC Struman BMS database contains a significant quantity of bridge and culvert (defined in *section 2.2.1*) data and has mainly been used for prioritization of structures for remedial work activities. It is however acknowledged that the BMS is also used for obtaining detailed inventory information and photos of structures; identification of structures on roads/areas of interest; determining and monitoring asset value etc. However, is believed that the data contained may provide useful information related to the defects on structures and the structure's conditions. This information could also be used to provide insight into predominant defects, their spatial distributions and their relationships with inventory data such as; location, structure type, age and span length/width. However, because there have been no investigations conducted, there is no evidence suggesting that relationships exist between the defects and the indicated inventory data. Also, the condition of the structures in relation to the indicated inventory data is not known. It is for this reason that the study was conducted and it intends to provide a deeper understanding of the condition of the structures in the Western Cape Province.

There are shortcomings of mainly utilizing data from visual assessments for prioritization of structures as other essential information, such as that mentioned earlier, is not taken into account. Transportation agencies stand to benefit from this information as it could be used to forecast structural deterioration; improve the decision-making process during resource allocation; inform future bridge and culvert design and advise future technological advances to the Struman BMS including its database.

1.3 Research Objectives and Research Questions

The research was focused on providing an indication of the state of the Reinforced Concrete (RC) bridges and culverts (the most common bridge and culvert material type) in the Western Cape Province by analysing the data contained in the Struman BMS database. This was achieved by fulfilling the primary objectives listed below:

1. Identifying the types of defects found on the RC structures.
2. Determining the most predominant defects on RC structures and discussing their possible causes and/or deterioration mechanisms.
3. Investigating relationships between defects and location (in terms of district municipalities) structure type, age and span length/width.

The research questions to be addressed were:

a) Do the RC bridges and culverts throughout the province have similar forms of deterioration?

b) What are the critical and most predominant defects of RC bridges and culverts and what possible causes and/or deterioration mechanisms can be associated with these?

c) Are there spatial distribution patterns of the defects based on the various environments in the Western Cape Province (i.e. coastal and inland)?

d) Do relationships exist between predominant defects and the structure location, the structure type, the age and the span length/width?

1.4 Scope and Limitations

The study was based only on the RC bridge and culvert data obtained from the Struman BMS database. The investigation was confined to the Western Cape Province located in South Africa. The data used for analysis was limited to that from the Struman BMS database supplied by the Roads and Transport division of the CSIR Built Environment. Lastly, the study focused on the prevalence of the defects rather than their severity, nonetheless the condition of the structures was investigated and an indication of the general condition provided.

1.5 Organisation of the Study

The research study is presented in the five chapters detailed below:

Chapter 1

This chapter introduces the study by providing a background into the investigation and a description of the study area. The research problem is defined and its significance highlighted. The research objectives are then outlined followed by the research questions. A short paragraph outlining the scope and limitations is provided followed by an account of the organisation of the dissertation.

Chapter 2

Chapter 2 provides an indication of the common types of bridges and the defects found on bridges in the Western Cape Province as well as the contributors and causes of deterioration and deterioration mechanisms related to bridge and culvert defects. The chapter then discusses bridge and culvert valuation including the consequences and evaluation of bridge or culvert failure. It then progresses to contextualize the practice of BM and BMSs and provides a review of existing BMSs. Thereafter the Struman BMS is discussed in greater detail.

Chapter 3

This chapter provides a description of the method through which the data was attained and validated. It then describes the data mining techniques employed to extract information from the data. This includes outlining the principles, inputs and outputs of the logistic regression analysis software used; STATA 2011.

Chapter 4

Chapter 4 presents the findings from the data analysis. Initially inventory information pertaining to all the structures in the database is presented. Thereafter, the findings from the data mining activities and the STATA 2011 logit models are presented and critically discussed. Lastly, a brief summary of the findings is provided.

Chapter 5

Chapter 5 concludes the study and provides recommendations regarding the development of BMSs and BMS software as well as further research in the development of CI indicators and research which will be a continuation from the study.

Chapter 2

2 A Review of the Existence and State of Bridges and Culverts

2.1 Introduction

The review begins by clearly defining bridges and culverts followed by an account of the common types of bridge structures, including those common in the Western Cape Province. It then provides an indication of defects that are common to specific structure types. This is followed by a section that discusses contributors and causes of deterioration and deterioration mechanisms related to defects on bridges and culverts. Next, the risks associated with bridge or culvert failure are discussed; this includes Whole Life Cycle Costing (WLCC) and costing consequences of failure. Thereafter, an introduction to BM and BMSs is provided followed by a review of BMSs currently in existence. The review then progresses to provide a detailed description of the Struman BMS adopted and employed by the PGWC. Finally, a database analysis recently carried out on the Struman BMS database of Namibia is reviewed.

2.2 Bridges, Culverts and Related Defects

2.2.1 Bridge and Culvert Classification

According to the Committee of Transport Officials (2013a) a structure may be classified as a bridge should it fulfil the criteria summarized in *Figure 2-1.* This includes:

1. The length of any single span (measured horizontally at the soffit along the road or rail centre line or between the centres of its support) being equal to or greater than 6 m.

2. The individual spans exceed 1.5 m and the overall length exceeds 20 m.

3. The maximum vertical distance measured from the streambed or structure floor at the inlet or from the top of any base to the soffit of the superstructure is equal to or greater than 6m.

4. The structure being a road-over-rail/ rail-over-road structure, if the span is less than 6 m.

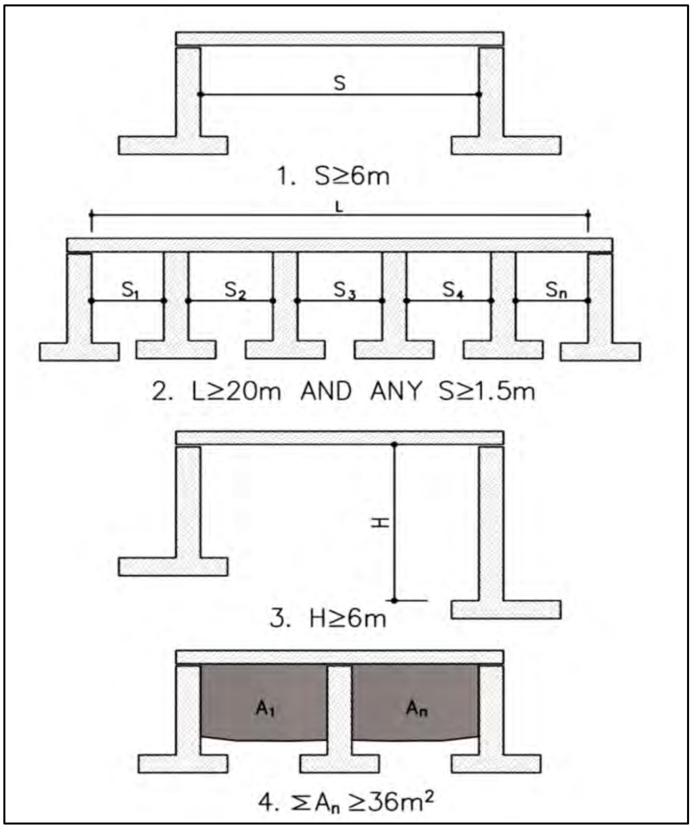

Figure 2-1: Bridge Classification Criteria

Source: Committee of Transport Officials (2013a)

A structure may be classified as a culvert should it be a cellular structure containing dimensions smaller than those defined for bridges. Additionally, it may also contain a span length/width or a diameter greater than or equivalent to 2.1 m or a culvert with cross-section opening greater than or equivalent to 5 m^2. Lastly, a structure may be classified as a lesser culvert should it contain dimensions less than those defined for major culvert (see *Figure 2-2* for culvert classification criterion).

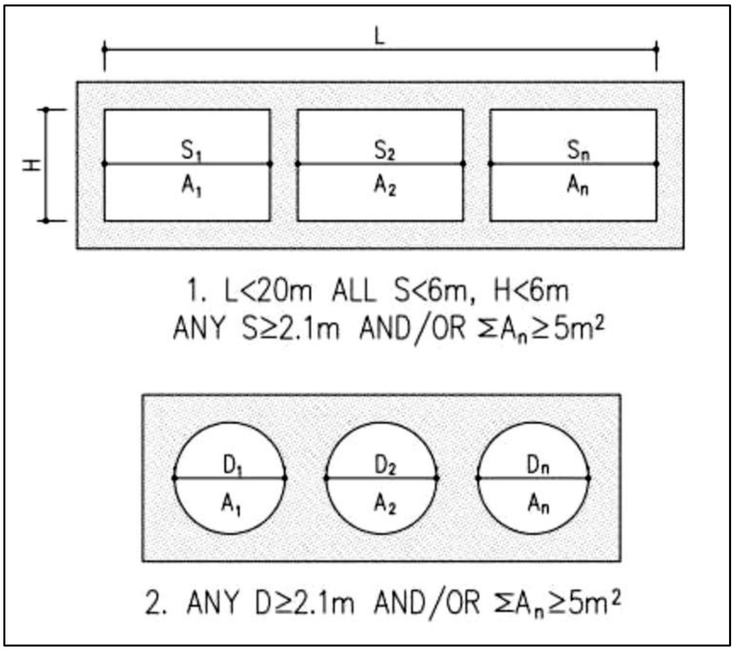

Figure 2-2: Culvert Classification Criterion

Source: Committee of Transport Officials (2013a)

All bridges and culverts contain three main components, namely; the superstructure, the deck and the substructure (see *Figure 2-3* and *Table 2-1*). However culverts are generally 'buried' with road fill or road slabs on top; thus elements such as deck slabs and abutments are not as easily identifiable as the apron slabs and cut-off walls (Committee of Transport Officials, 2013a).

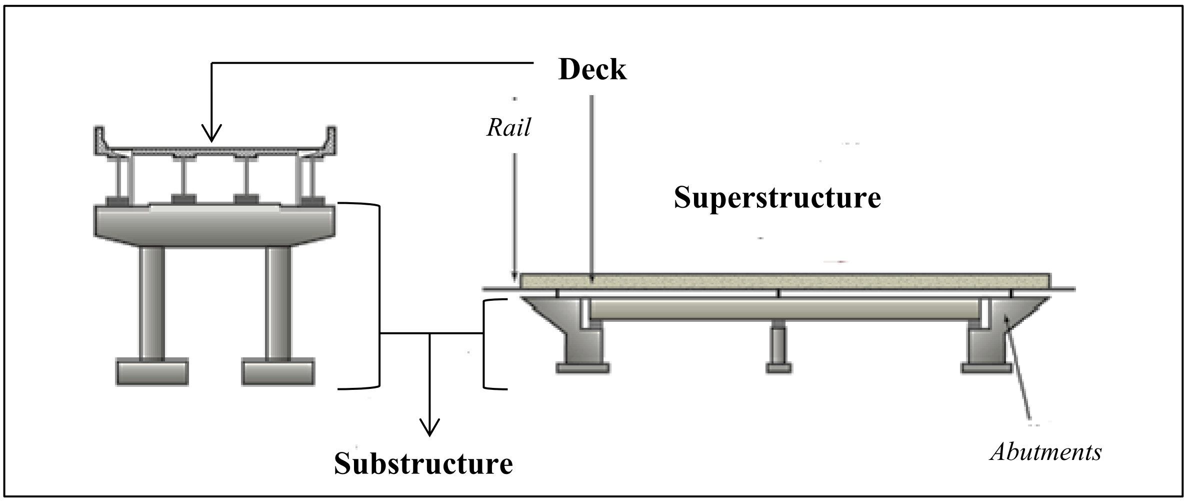

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Figure 2-3: Structural Elements of a Bridge Structure

Source: http://rmw.vtransprojects.vermont.gov/related_links/ [Accessed 2014, June 1]

Table 2-1: The Main Components of Bridge Structures and their Description

Component	Description
Superstructure/ Decking	The component of the bridge that carries the load which is the roadbed is the substructure (i.e. roadbed, truss or girder etc.). It then transfers the load to the substructure and thereafter, the ground. The substructure is the roadbed, truss or girder, etc.
Bearings	The bearings are components that ensure that the dead and live loads and are evenly distributed and transferred to the substructure.
Substructure	The substructure refers to the component of the structure that transfers the loads to the ground. This includes the abutments, piers, wing walls, foundations etc.

Bridges and culverts may be grouped into various structure type categories according to the main structural form from which the design is based (Bhattacharyya and Wright, 2013).

The most common bridge structure types include (Bhattacharyya and Wright, 2013):

1. **Simply Supported**
   Beam bridges are the most common type of bridge. They are typically a beam supported by two supports, such as piers or columns. The vertical forces on the bridge become shear and flexural loads on the beam and are transferred down its length to its piers or columns. Beam bridges may be sub-divided into two categories, namely; reinforced concrete and steel beam bridges.

2. **Continuous Bridges**
   If two or more beams are joined rigidly and extend over supports more than two supports, as seen in *Figure 2.4 (a)*, the beam bridge becomes a continuous beam bridge. Both tension and compression forces on the top and bottom of the beam are transferred from the beam to the ground via the abutments and the piers or columns. Continuous bridges may also be reinforced concrete or steel beam bridges.

3. **Truss Girder Bridges**
   The steel truss girder bridge is very similar to the beam bridge, except a truss is used. There are a variety of trusses that have been developed. These are usually

determined by the basis of the design and the location of the bridge. *Figure 2.4 (b)* shows an illustration of a Pratt steel bridge. The forces experienced by the bridge are dissipated through the entire truss. Truss members carry either compression or tension forces. These forces are transferred to the supports and then the ground.

4. **Arch Bridges**

   Arch bridges have an arch with pillars that support the road deck as seen in *Figure 2.4 (c)*. There are practically no tension forces on the underside of the arch; the forces on the deck are transferred to the top of the arch and are thereafter distributed to the abutments and consequently the ground.

5. **Cantilever Bridges**

   Cantilever bridges usually have two beams supporting another beam which is supported by piers or columns or as seen in *Figure 2.4 (d)*. The dead and live loads of the bridge are borne by the two outermost piers and then transferred to the ground through these pillars. This beam is usually the vehicle roadway and is composed of reinforced concrete.

6. **Suspension Bridges**

   Suspension bridges contain horizontal suspension cables connected to main cables that run over towers from one end of the bridge to the other as illustrated by *Figure 2.4 (e)*. The towers and deck are usually hollow steel boxes and reinforced concrete, respectively. Both the suspension cables and main cables carry a tension force resulting in the development of a compression force in the towers. The forces on the suspension cables and the towers balance each other out and the forces from the main cable are transferred to the ground, thus the bridge stays in equilibrium.

7. **Cable-Stayed Bridge**

   The cable-stayed bridge and the suspension bridge may appear very alike however, the cable-stayed bridge does not require any anchorage nor does it require two towers. *Figure 2.4 (f)* shows an illustration of a harp designed-cable stayed bridge in which the cables run parallel to each other and connect the deck to the towers. The loads experienced by the deck produce a tension force on the cables and a resultant compression force is experienced by the towers.

It may be noted that the bridges of the Western Cape Province range mainly between structure types (a) to (d).

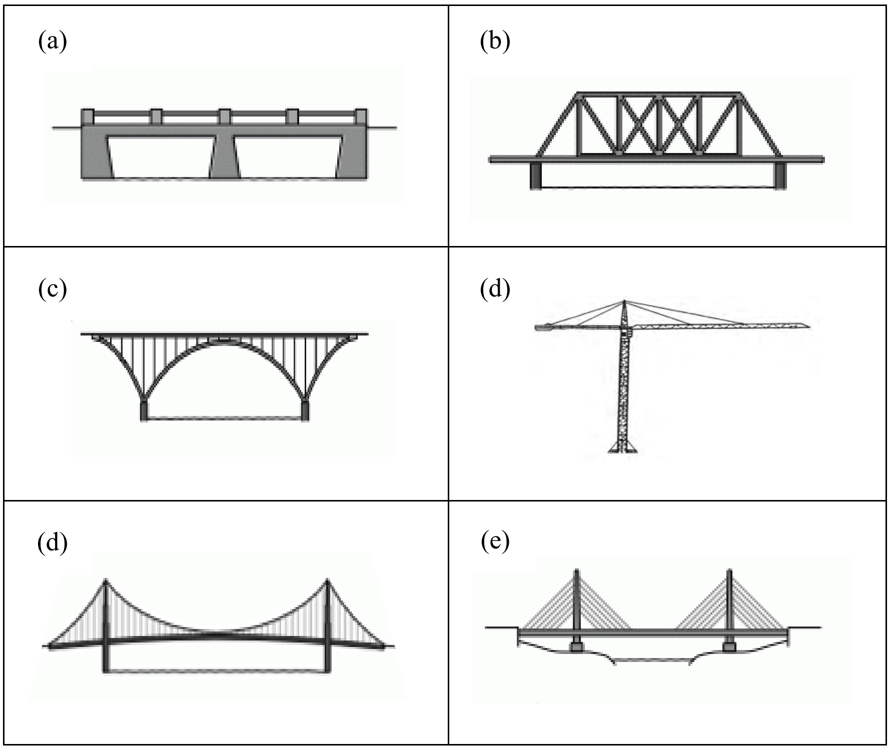

Figure 2-4: Structure Types: (a) Continuous Bridge; (b) Truss Bridge; (c) Arch Bridge; (d) Cantilever Bridge; (e) Suspension Bridge; (f) Cable-Stayed Bridge

Source: Bhattacharyya & Wright (2013)

It may be noted that there are also various beam decks, namely solid slab decks, voided slab decks, and twine spin beam decks (Rajagopalan, 2006). However the most common are solid slab beam decks, precast and cast in-situ beam-and-slab decks and concrete box girder decks (Rajagopalan, 2006).

There are a variety of materials used in bridge and culvert construction. These vary for various structure types and may also be determined by the structure's proposed appearance and required strength (Bhattacharyya and Wright, 2013). Goode *et al.*, (2006) state that reinforced concrete is the most commonly used material in the construction of bridges and culverts because of its versatility and its durability. Furthermore, while concrete is stronger in compression, steel is stronger in tension. Thus the concrete compensates for shortfalls in the steel and *vice versa*. Other bridge and culvert materials include composites constituting the combination of concrete, steel and timber as well as masonry materials. A large quantity of the bridges and culverts in the Western Cape are reinforced concrete. RC structures are suitable to an environment such as the coastal areas of the Western Cape Province as they do not expose the steel thus making it less susceptible to corrosion.

2.2.2 Structure Type and Element Specific Defects

Defects on bridges and culverts are identified through means of visual inspections. The primary objective of visual inspections is to determine the types and severity of defects that appear on the structure and the extent of the damage and/or deterioration. Frangopol *et al.,* (1999) and Ryall (2010) specify that prior to inspection, structures should initially be subdivided based on the 'physical landmarks' by which they are bounded. Thereafter, they are subdivided into constituent parts (namely the superstructure; substructure and foundations) and then those parts further subdivided into separate elements.

Various structural defects may be associated with certain structure types. Furthermore, according to Ryall (2010) inspection of the deck of a bridge is usually given the highest priority as it carries the traffic load directly and therefore there are particular defects that are expected to be found on it. *Table 2-2* lists various structure types along with the defects usually associated with them. The defects on a large majority of bridges and culverts of the Western Cape include, but are not limited to, those listed under RC and composite structures.

Table 2-2: Defects to Inspect on Various Structure Types

Structure	Concrete Decks and Slabs	Pre stressed RC structures	Composite Structures	Masonry Arch	Timber Bridges
Defects	Cracking	Cracking	Corrosion	Cracking	Corrosion
	Spalling	Spalling	Fracture	Ice	Decay
	Leaching	Leaching	Deformation	Water	Insects
	Deck surface damage	Deck surface damage	Protective system	Weathering	Loose decks
	Poor quality concrete	Poor quality concrete	Loose fixings	Vibration	Deck Separation
	Accidental damage	Accidental damage	Accidental damage	Accidental damage	Accidental damage
	Corrosion of Reinforcement	Corrosion of Reinforcement		Poor workmanship	
	Deck surface damage	Deck surface damage			
	Salt crystallization	Salt crystallization			

	Deflection and vibration	Deflection and vibration Condition of flanges & webs Condition of grout			

2.3 Contributors and Causes of Deterioration and Deterioration Mechanisms Related to Defects

2.3.1 Introduction

The defects summarized in the previous section may be attributed to a number of causes and imply several deterioration mechanisms. Presentations prepared by Beushausen (2013) and Moyo (2013) suggest that there are six main causes of defects. These causes are namely design inadequacies; poor construction practices and techniques; material performance shortfalls; physical actions; unfavourable environmental factors and the lack of appropriate and adequate maintenance programmes. The sole purpose of this section is to provide a better understanding the causes of deterioration and deterioration mechanisms implied by defects. It is important to note that the contents of the categories discussed in this section are broad and therefore only the main deterioration contributors identified in each category have been covered.

2.3.2 Design Inadequacies

Design inadequacies are largely due to design professionals failing to either understand the type of structure required (considering its purpose and environment) or produce comprehensively detailed and accurate designs and construction documents or both (Gatlin, 2013). The design phase is one that involves many assumptions, iterative calculations, in-depth detailing and analysis. This phase integrates a variety of disciplines such as architecture, civil engineering, geotechnical engineering, drafting etc. According to Al-hammad and Assaf (1997) design defects may therefore be grouped into six categories which have been tabulated in *Table 2.3.* This stage is fairly sensitive to errors, which, if go unnoticed, may be implemented in construction and later compromise the structures safety and/or integrity.

Table 2-3: Categories of Design Defects and their Description

Design Defect Category	Description
Defects in architectural design	- Ignoring climate effects on exterior shapes and finishes. - Designing inadequate joints between finished surfaces.
Defect in civil design	- Insufficient provision made for movement and expansion. - Disregarding environmental conditions and biological effects. - Improper structural design for required load capacity and existing soil conditions. - Exceeding allowable deflection limits - Ignoring wind effects. - Insufficient cover.
Design defects in maintenance practicality and adequacy	- Ignoring site access for maintenance equipment as well as maintenance equipment availability. - Ignoring maintenance requirements in the drawing.
Defects due to consultant firm staff and administration	- Lack of designer field technical background and experience. - Lack of training programmes for designers. - Misunderstanding of user's intended use.
Defects due to construction specifications	- Unclear specifications and mix design. - Inadequate definition of material types. - Overlooking construction procedures.
Defects due to construction drawings	- Insufficient or conflicting detail. - Lack of references.

Chong *et al.* (2006) suggests that the defects traced back to the design process could be associated with incorrect design assumptions; lack of experience and design knowledge as well as information given to designers; errors in calculations alongside other organisational and management factors. Additionally, Blaney and McGlone (2011) state that problems in design mainly result from insufficient knowledge and understanding of building codes and good building practices from designers and the contractors who implement those designs. Furthermore, other defects that later appear result from the design defects and could have initially been avoided (Chong and Low, 2006).

2.3.3 Poor Construction Practices and Techniques

Defects may arise during design, construction and even after the project has been completed (Gatlin, 2013). Blaney and McGlone (2011), claim recent studies have shown that 70 % of defects identified on structures may be traced back to poor construction site practices and techniques. Furthermore, the most problematic of these defects are those said to be related to the intrusion and build-up of moisture within the building envelope.

According to Gatlin (2013) there are two types of construction defects: those visible to the eye (also known as patent defects) and as well as those concealed within the structure and are not as easily identified, also known as latent defects (see examples in *Table 2-4*). The development of these defects is important as sometimes it is only after they manifest that it may be determined whether the defect was a patent or latent defect. Thereafter further investigations may be carried out. Chong and Low (2006) indicate that it is very difficult to detect latent defects due to the timeframe in which they develop. The consequences of these defects range from mild to extreme and may be experienced instantly and/or after long periods of time.

Table 2-4: Latent Defects vs Patent Defects

Patent Defects	Latent Defects
Reinforcements visible on concrete structure	Incorrect placement of reinforcement
Lack of drainage including incorrect slopes	Improper compaction of soil
Improper consolidated concrete	Improper consolidated concrete
Missing control/ expansion joints	Insufficient cover
Cracking and signs of distress	Inadequate curing
	Poor workmanship and quality

2.3.4 Material Performance Shortfalls

The performance of a material may be hindered by its lack of coherence resulting from inconsistencies in the mix as a result of improper mixing during construction; inadequate curing; environmental factors which may have gone undetected or neglected during the design phase as well as incorrect material selection also during the design phase. Such performance shortfalls include high shrinkage as well as Alkali-Aggregate Reactions (AAR) and Alkali-Silica Reactions (ASR) and according to Moyo (2013), are usually identifiable by irregular deflections, cracking and other defects during and even after construction.

After the casting, the free water of the fresh concrete evaporates and shrinkage due to loss of capillary water takes place (Lees, 1992). Shrinkage results in cracking which does not often necessitate repairs if the concrete is properly cured because the volume changes are generally insignificant. The hardening concrete experiences plastic shrinkage the first few hours after casting followed by drying shrinkage two to four weeks later (Hans 2013). Plastic shrinkage cracks are usually randomly positioned whereas dry shrinkage cracks run parallel to each other and are more distinct *(see Figure 2-5)*.

The process of drying shrinkage causes an increase in tensile stresses. This may result in cracking, internal warping and external deformations. The extent of drying shrinkage is to a large extent governed by the properties and portions of the constituents, the method of curing, the environmental conditions and the sizes of the members. Cement Concrete Aggregates Australia [CCAA] (2002) suggest that dry shrinkage should not pose a threat should a mix consisting of suitable constituents and properties be used and if the hardening concrete has been allowed sufficient space for movement. Lees (1992) states that the volume of aggregates used in the concrete mix have a major influence on the concrete's shrinkage. This is because of the restraint it may have on the shrinkage of the gel fraction and because more aggregate implies less paste, therefore less shrinkage. Consequently, design inadequacies such as errors in calculations of the volumes of constituents to be used could result in shortfalls in material performance, such as too much shrinkage leading to severe cracking and crack propagation over time.

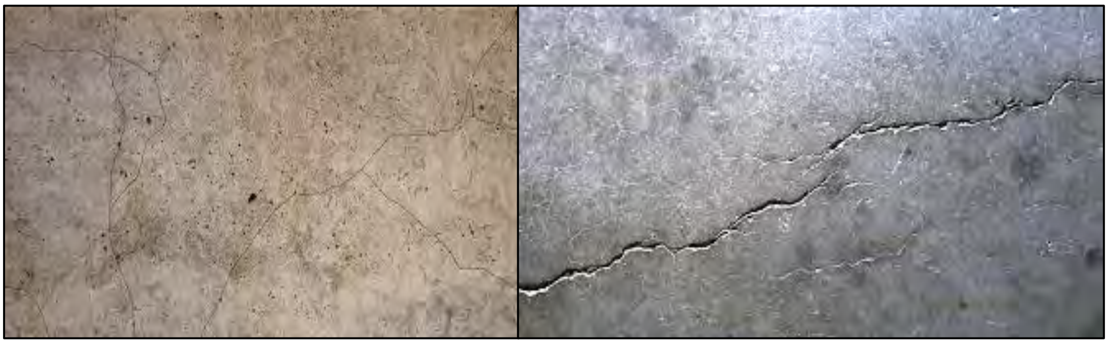

Figure 2-5: Random Plastic Shrinkage (left) and a Horizontal Dry Shrinkage Cracks (right)

In addition, ASR, the most common form of AAR, may take place due to growing swelling pressures caused by the reactive interactions between the alkali hydroxides (usually derived from the cement) and reactive forms of silica in the aggregate particles (Swamy, 1992). The result is map cracking as seen in *Figure 2-6.*

Figure 2-6: Map-cracking of Concrete due to ASR

Source: http://enr.construction.com/infrastructure/transportation/2012/0227-officials-will-soon-seek-designer-for-historic-la-bridge-replacement.asp [Accessed 2013, October 12]

According to Farny and Kosmatka (1997), the cement paste between the aggregates within the concrete contains interconnected microscopic pores through which water and ions in solution may manoeuvre. The result of the reactions is the formation of a gel reaction product which swells when it comes into contact and absorbs the water migrating through these pores. Moyo (2013) and Farny and Kosmatka (1997) indicate that high swelling gels may cause abnormal expansion and map-cracking of the concrete, should the pressures resulting from the gel swelling exceed the tensile strength of the concrete. It may be noted that for this reason, the presence of gel may not always be harmful or destructive as some gels experience very little or no expansion. Destructive ASR however, may largely be associated with the selection of unsuitable building materials. In order for ASR to take place, three conditions are required. These conditions are: the pore solution must contain a sufficiently high amount of alkali; reactive forms of silica are present in the aggregate and there is an adequate amount of moisture/ water in the concrete (Winter, 2005).

Jones and Clark (1998) carried out a study concerned with the effects of ASR on the properties of concrete. They found that the tensile and compressive strength of concrete were reduced by ASR (with the reduction being directly proportional to the increase in expansion) and the Young's modulus could decrease significantly (due to the micro-cracks as opposed to the expansion). Furthermore it was determined that the prestress developed by the ASR expansion enhanced the shear strength and stiffness of beams. Moreover, Haha (2006) found that the flexural strength of concrete structures located in environments of high temperatures was very sensitive to the effects of ASR as it decreased linearly depending on the content of alkali in the concrete.

2.3.5 Physical Actions

Damage and deterioration of bridge structures caused by physical actions is not always easy to account for during design as it is often unpredictable and may therefore so easily be disregarded. Nonetheless, such damage is important and should be anticipated as it puts not only the structure at risk but the users as well. Damage and deterioration from physical actions resulting in further bridge deterioration and even failure include impact, overloading, scour and undermining and foundation settlements.

Fu *et al.*, (2011) investigated and assessed structural damage of bridges due to impact from over height vehicles in Maryland, one of the least most densely populated states in the United States of America. Information was gathered from several sources including the over height vehicle detector, the states accident database and the bridge inventory. The results from the analysis carried out on the data showed that most of the bridges struck were either located in the urban areas, which had more traffic, and in the rural areas in cases of bridges with low clearances. Also, the most common types of loads that caused the accidents were enclosed box trucks and trailers and approximately half of these accidents resulted in damage. Accidents that caused severe damage to the bridges were approximately 20 % of those analysed. It was also established that the accidents caused by over height vehicles have increased over the years and if Maryland was a representation of such accidents around the nation and globe, then they too could be assumed to be escalating. Mclinn (2009) found that impact was the most common cause of major bridge failures in the US.

In addition to this, Mclinn (2009) identified overloading as one of the two leading causes of bridge failures internationally (the other being failure during construction). Overloaded road structures are generally more susceptible to fatigue failure. Leander (2010) provides a survey of methods for fatigue life prediction of bridge structures. Such methods include but are not limited to the Palmgren-Miner Rule (one of the most widely used approaches), the Nominal and Notch Stress Approach as well as fracture mechanics. It may be noted that although fatigue has been in known for many years now. None of the methods attain results as accurate as those attained from testing of real components using the known/realistic time history of the load. Nonetheless, using the Palmgren-Miner Rule, the damage is calculated may be calculate using *Equation 2.1* and *2.2.*

$$D = \sum \frac{n_i}{N_i} \qquad \text{... (2.1)}$$

Where, D is the damage effect is index; n_i is the number of cycles in the stress range N_i is the number of cycles to fracture.N_i may be calculated using the Basquin relation:

$$N = \sum \left(\frac{C}{S_r}\right)^m \qquad \text{... (2.2)}$$

Where, C is the coefficient; m is the slope of the log-log S-N curve and S_r is the stress range.

Other mechanisms of bridge and culvert deterioration capable of resulting in failure are scour and undermining. These two modes of bridge deterioration decrease the lateral restraint as well as the bearing area of the spread footing. According to Ruocci *et al.* (2008) undermining and scour pose serious threats to RC structures, especially arch bridges as a result of their sensitivity to bearing loss. Bridges that have their piers located in riverbeds and culverts which are submerged for long periods are highly prone to scour and undermining as high velocities and the formation of vortices erode the submerged pier elements. If the foundations are not sufficiently deep the material under the substructure is washed away. Furthermore, when the scour depth increases beyond a certain level, the lateral strength in the foundations may become insufficient to prevent the piles from yielding. Consequently, the probability of the foundations being subjected to damages is increased if they are exposed to seismic activity (Song *et al.*, 2005).

The continuous asymmetric loss of base soil due to undermining may ultimately lead to rotations and differential settlement of the foundations (Ruocci *et al.*, 2008). Dupont and Allen (2002) indicate that the dynamic traffic loading and static embankment loading may also contribute to foundation settlement and in cases of undermining these loads may even accelerate settlement. Thus a bridge or culvert's mechanical failure may be attributed to crack development within the substructure and superstructure as a result of scour and the foundation settlement resulting from undermining.

2.3.6 Unfavourable Environmental Factors

All bridges and culverts are exposed to and thus influenced by various environmental factors. Moncmanová (2007) suggests that designing concrete structures without taking their environments into consideration is a seriously flawed practice and may result in a reduced service life for the structure as well as be uneconomical in the long-term. Severe climatic conditions may result in corrosion of the steel reinforcements within the RC structures. Thus during the design process the appropriate codes and specifications should be referred to for the appropriate concrete mix design, material selection, cover etc. in order to ensure durability and minimize adverse environmental impacts. It is however important to remember that there are factors that are more complicated to design and cater for due to the uncertainty in the likelihood of their development as there is always the probability that they may or may never develop. Such environmental factors are those resulting from natural disasters.

Other unfavourable environmental conditions include chemical and biological attack. Natural disasters (which include seismic hazards, floods; fires, snow, hurricanes etc.) not only cause deterioration and damage to infrastructure, but can also cause adverse social effects, harsh economic impacts, serious injuries and fatalities. The consequences of these can be great as they may result in a direct economic loss, a term used to refer to losses which are quantified based on the repair or replacement cost of the damaged infrastructure (Tirasirichai and Enke, 2007). According to the website Pavement Interactive (2012), flooding is the most common of natural disasters and may lead to scouring, soil erosion, weakened foundations as well as other defects and ultimately failure. Lastly, bridges and culverts are at higher risks of cracking, differential settlement and failure during natural disasters.

Mohamed (2008) indicates that there are two main reasons for steel corrosion, namely; chloride ingress and carbon dioxide penetration, and that the extent of rebar corrosion due to these may also compromise the structures safety and integrity. This is because after the steel begins to corrode there is high potential of cover concrete falling and subsequent spalling as corrosion perpetuates. Consequently, the structure experiences a decrease in strength due to the reduction in both the concrete and rebar's cross-section. The two phenomena's are unusual in that they do not attack the concrete, as their aggressive agents infiltrate the concrete and attack the steel. Rather, the damage due to the steel subsequently becomes the primary cause of the concretes deterioration.

The two mechanisms of carbonation and chloride ingress are completely independent of one another. Nonetheless, they may take place simultaneously in the same RC structure. In further detail, McKinley (2005) describes the carbonation process as one that is usually (depending on the porosity and permeability of the concrete) slower than chloride ingress. Carbon dioxide (CO_2) from the atmosphere penetrates through the porous concrete and dissolves to form carbonic acid which neutralizes the alkali in the pore water. Calcium hydroxyl exits and the concrete's pH level decreases causing the oxide film to become unsteady. Carbonates attack and spread throughout the concrete reacting with calcium hydroxide ($Ca(OH)_2$) to form calcium carbonate ($CaCO_3$). Provided there is enough oxygen and moisture, the steel will begin to corrode as the concretes alkalinity has been reduced. *Figure 2-8* shows a simple illustration of the carbonation process's progression.

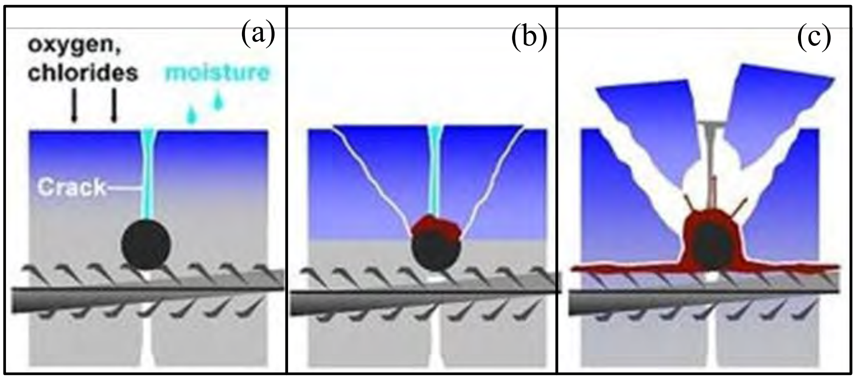

Figure 2-7: Carbonation Process: (a) Penetration of Carbon Dioxide; (b) Reaction between CO_2 and $Ca(OH)_2$, Formation of $CaCO_3$ and Decreases in pH; (c) Localised Corrosion, Cracking and Spalling

Source: Friend (2013)

On the other hand the process of chloride ingress *(see Figure 2-9)* causes corrosion at a much faster rate than the carbonation process (McKinley, 2005). Chloride ions exist in the concrete if saltwater had been used in the concrete mix and if the additives or water used contained chloride contents over those specified. Chlorides are also able to penetrate the concrete if it is exposed to saltwater spray and if there are chlorides present in the chemicals that attack the structure (Mohamed, 2008; Ghods *et al.*, 2007). According to McKinley (2005) once the chloride threshold concentration is reached, the chloride begins to break down the passive film protecting the reinforcement. The removal of the passive film initiates the galvanic corrosion process whereby the steel begins to corrode. The distribution of chloride through the RC structure is not uniform hence the corrosion is localised.

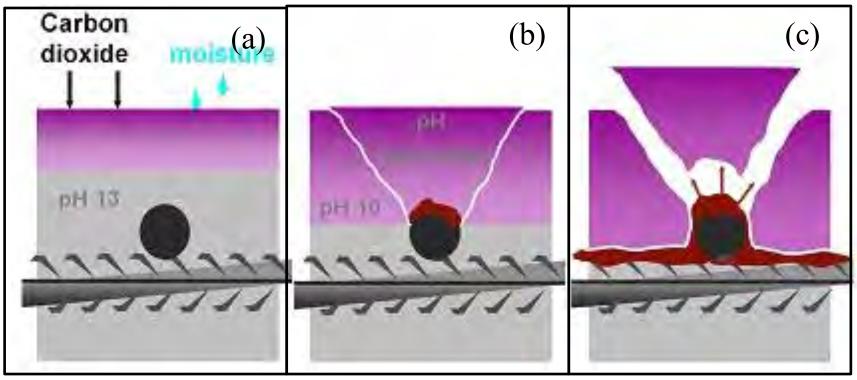

Figure 2-8: Chloride Ingress: (a) Chloride Ions Infiltrate; (b) Removal of Passive Layer and Initiation of Galvanic Corrosion Process; (c) Localised Corrosion, Cracking and Spalling

Source: (Friend, 2013)

Lastly, biological attack may take place in conjunction with chemical attack, making it difficult to distinguish the amount for which the various deterioration mechanisms are responsible. From the various definitions of biological deterioration provided by Jayakumar (2012) it is clear that it involves the loss of structural capacity and quality over time due to live organisms or material leaching. Cwalina (2008) states that material deterioration of RC structures resulting from biological attack by microorganisms (such as bacteria, microscopic fungi, algae etc.) as a whole, is estimated at a range of approximately 30 %. According to Márquez *et al.* (2013), bacteria is said to be the most aggressive microorganism to RC components. The bio-deterioration process may be subdivided into two phases namely the initiation phase and the secondary or development stage, both of which are described in *Table 2-6.*

Table 2-5: Phases of the Bio-deterioration Process

Phase	Description
1. Initiation Stage	Initially the concrete is conditioned for microorganisms. This is achieved by a lowered pH, moisture gain and subsequently biofilm formation. Consequently the bio-receptivity of the concrete due to these conditions makes it ideal for microbial growth. Also, minor changes in the aesthetic appearance may be noted.
2. Secondary/ Development Stage	During the development stage, microorganisms grow and spread. As a result of this the structure becomes more prone to chemical attack

(Adapted from Márquez, Sanchez-Silva and Husserl, 2013.)

2.3.7 Lack of Appropriate Maintenance Programmes

Lack of appropriate maintenance programmes are able to contribute to the further development of defects. Therefore a good maintenance strategy can prevent excess spending in the future and almost guarantee the structures functionality throughout its service life. There are different approaches for addressing the issue of maintenance; such approaches include reactive, predictive, preventative as well as proactive maintenance and have been described in more detail in *Table 2-7* (Emerson Process Management Publication, 2003). It may be noted that all of these strategies are relevant but not necessarily always applicable.

Table 2-6: Maintenance Strategies

Maintenance Strategy	Description	Adverse Implications
Reactive Maintenance	The reactive maintenance approach is one that refers to a "run-till-failure" philosophy. A structure is therefore not repaired or replaced until it fails.	Structural safety and integrity is compromised. High and unplanned expenditures are incurred.
Predictive Maintenance	The preventative maintenance approach is time based and planned. The aim of the approach is to keep the structure in a healthy condition.	Sometimes it may be wasteful and costly as the structure is repaired prematurely.
Preventative Maintenance	The preventative maintenance approach is one in which maintenance is dictated by the structures condition as opposed to time intervals.	The root cause of failure may be misinterpreted or not identified at all.
Proactive Maintenance	The proactive maintenance strategy relies on information provided by predictive methods to identify problems and isolate the source of failure.	Repairs often made while something is not broken.

While some may argue that one strategy has more advantages over another or one is more suited over the other, it is important to realize that the combination of two, three or even four of these strategies is more effective than the selection of only one. It is acknowledged that various combinations will be suited to different scenarios and are dependent on the maintenance aims and objectives. According to the Emerson Process Management (2003) publication the more critical and the more potential for extensive deterioration, the more the selected strategy should move towards predictive and proactive maintenance.

RC structures that are exposed to severe environments and aren't regularly inspected and maintained usually require repairs after 15 to 25 year. These repairs are costly and extensive due to that the harsh environmental conditions result in unacceptable levels of damage, usually corrosion damage, within the structure. The structures integrity and/or safety are compromised. It then requires extensive and costly repairs to return it to an acceptable quality. *Figures 2-10* and *2-11* provide an indication of the implications of irregular maintenance and regular maintenance, respectively (Beushausen, 2013).

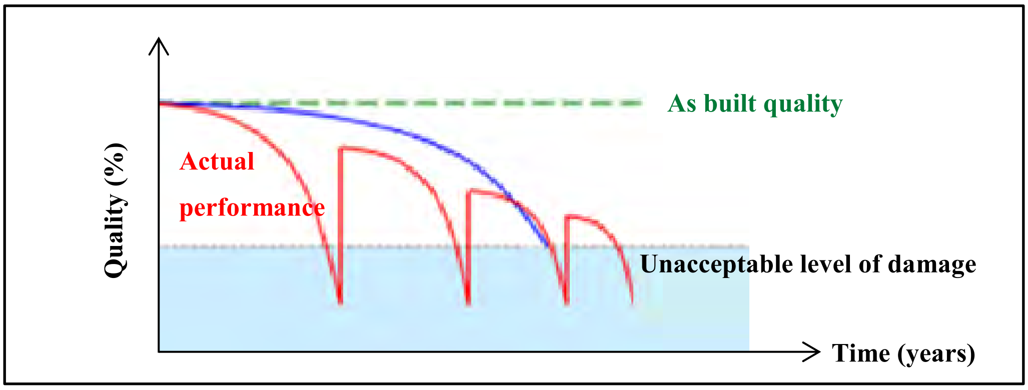

Figure 2-9: Service Life of RC Structures only Subjected to Irregular Maintenance and/or Repairs

Source: Beushausen (2013)

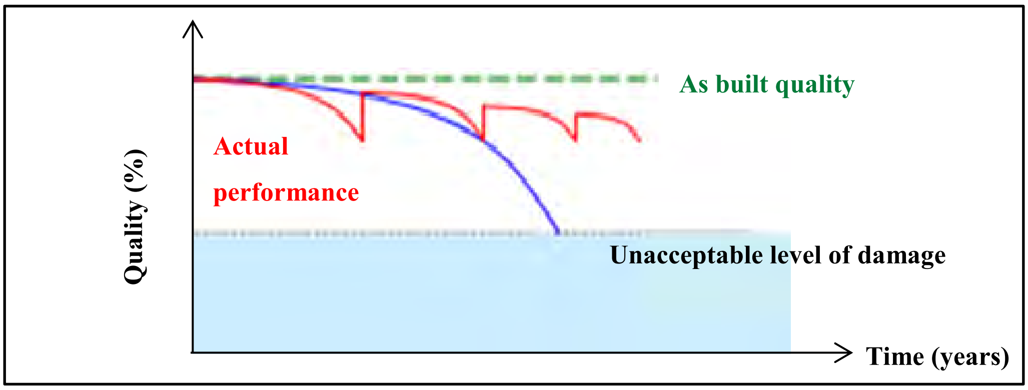

Figure 2-10: Service Life of RC Structures which Receive Regular Inspections and Maintenance

Source: Beushausen (2013)

2.4 Costs associated with Bridges and Culverts

2.4.1 Introduction

Bridges and culverts can and will fail if not properly designed, constructed or maintained. In hindsight most of the bridge and culvert failures that have occurred could have been prevented (Christian, 2010). Bridge and culvert failures (which in the context of the study refer to collapse or the loss of a structural component or severe damage compromising to the structures safety and integrity) may be attributed to a confluence of contributing events or underlying causes. Furthermore, their consequences may range from a simple inconvenience to a tragedy involving fatalities. This section discusses the concept of Whole Life Cycle Costing (WLCC), including its history and definition. It then discusses the consequences of structural failure and attempts to provide insight into estimating and quantifying the consequences of failure.

2.4.2 Whole Life Cycle Costing

The development of the WLCC theory can be dated back to the 1970's when terotechnology governed many decisions made by most professionals at the time. Terotechnology involved basing capital investment costs primarily on capital costs. It was believed that the long-term would bear significant cost savings when compared with less costly option. According to Boussabaine and Kirkham (2004) certain sectors appreciated that investment decisions could not be based on this ideology alone. During the 1970's it was decided that the operational expenditure of an asset should also be considered. The term 'cost-in-use', which referred to the latter, began to appear more frequently in literature and in industry. It was later realised that this method did not facilitate future forecasting and it became increasingly apparent that an alternative method was required.

In the 1970's, Life-Cycle Costing (LCC) was derived as a solution to the shortfalls of cost-in-use. By 1977, LCC had developed significantly and become very popular as it encompassed a number of techniques (engineering, mathematical, accounting and statistical methods) to determine the net expenditure associated with the life-cycle of an asset. Researchers continued to denhance LCC by developing models and data collection frameworks such that it became a recognized standard in the UK under the British Standard BS 3843 (Green, 2009). LCC became defined as "a technique which enables comparative cost assessments to be made over a specified period of time, taking into account all relevant economic factors both in terms of initial capital costs and future operational costs." Finally, during the late 1990's, the term of WLCC emerged (Boussabaine and Kirkham, 2004). The evolution of WLCC is represented in *Figure 2-12.*

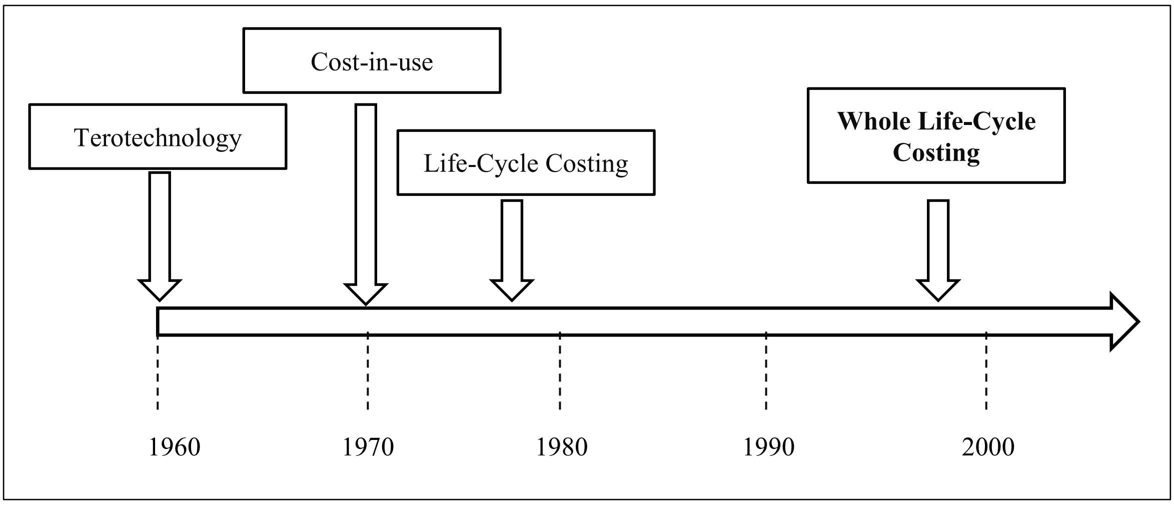

Figure 2-11: The evolution timeline of Whole-Life cycle costing

According to existing literature, it may be argued that WLCC and LCC are merely synonyms of each other, with WLCC as the modern equivalent of LCC. Others may argue that WLCC and LCC are terms that have been so often interchangeably used, their meanings have become confused. Boussabaine and Kirkham (2004) confirm that some researchers view WLCC as the advancement of LCC, as in their opinions it seeks to confront some of the concerns of LCC. Such concerns include the fact that WLCC takes into account the cost of the asset's entire life span, hence the term 'whole'. On the other hand, LCC is based on a predefined time interval (also known as the period of commercial interest), that should be specified in the LCC model.

The BS ISO 15686-5 draws a clear distinction between LCC and WLCC. It defines LCC as *"... a methodology for the systematic and economic evaluation of the life cycle costs over a period of analysis…*" and WLCC as "…*a methodology for the systematic economic consideration of all the whole life costs and benefits over the period of analysis…*" As seen in the *Figure 2-13,* it may be deduced that WLCC takes into consideration aspects beyond of the scope of LCC. Thus, in the context of this study, WLCC will be viewed as the evolution of LCC (Green, 2009).

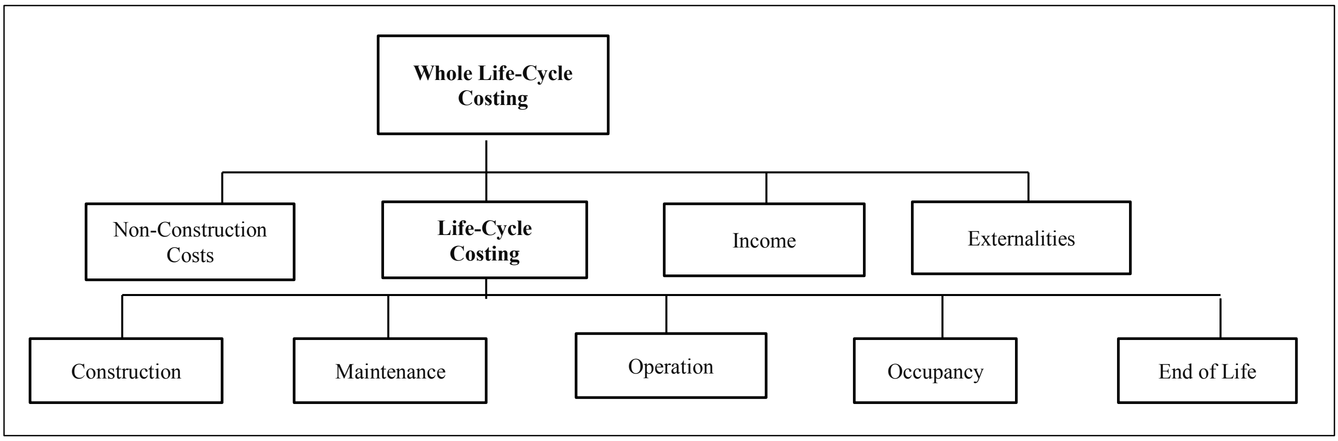

Figure 2-12: LCC a Component of WLCC

(Adapted from BS ISO 15686-5, 2008*)*

Nonetheless, Life Cycle Cost Analysis (LCCA) remains a significant tool as it informs bridge owners and managers about the potential consequences of their decisions in present monetary terms. According to Dhillon (2010) the LCC of a bridge may be expressed by *Equation 2.3* below.

$$LCC_{br} = CONC + INSC + DESC + FAIC + RAMC \qquad \ldots (2.3)$$

Where, LCC_{br} is the life cycle cost of the bridge, *CONC* the construction cost, *INSC* the inspection cost, *DESC* the design cost, *FAIC* the failure cost and *RAMC* the repair and maintenance cost.

WLCC aims to determine the total cost of a bridge from its initial conception and feasibility to the end of its service life. A Present Value Analysis (PVA) is conducted to estimate the assets worth over its whole life time thus providing investors with an indication of how much the asset is going to cost in the long term. During the PVA, the net present value (NPV) which estimates the cost of the asset in the long term in present monetary terms, is computed based on a simple investment formula as shown by *Equation 2.4* (Ryall, 2010).

$$C = P(1 + r)^n \qquad \ldots (2.4)$$

Where, C is the expenditure, P the net present value, r the interest rate and n the number of years.

However, for a bridge structure, a number of items of expenditure (i.e. abutments, piers, beams etc.) need to be accounted for, thus the cumulative NPV would be computed as indicated in *Equation 2.5* (Ryall, 2010).

$$\sum P = \frac{\sum C}{(1+r)^n} \quad \ldots(2.5)$$

Boussabaine and Kirkham (2004) indicate that WLCC takes into consideration the characteristics of the asset, its reusability, sustainability, maintainability and obsolesce. WLCC also takes into account capital, maintenance, operational, financial, and residual and disposal costs. It is from these factors that an asset's total cost may be determined. In the case of a bridge structure, the costs that would be taken into account include those of design, construction, repair, maintenance and upgrading, traffic management and the cost of demolition. Thus, Ryall (2010) concludes that the NPV of a new bridge structure would be calculated by *Equation 2.6*.

$$P = C + \sum_{n=1}^{n=N} \frac{M_n + TD_n + TB_n}{(1+r)^n} - CL_N + SV_N \quad \ldots(2.6)$$

Where, P is the net present value, n is the year of service life considered, r is the discount rate, N is the service life, C is the construction cost, M is the maintenance cost, TD is the traffic disruption, TB is the traffic betterment, CL is the capital loss and SV is the scrap value.

2.4.3 Case Study on Causes and Consequences of Bridge Failure

The consequences of a bridge or culvert failure may vary from structure to structure and is dependent on a wide variety of factors. The consequences of failure may thus be measured in terms of that which is damaged, destroyed or lost assets and utilities such as goods, services, lives etc. These may be divided into direct and indirect consequences. The direct consequences are those that may be associated with component or structural element failure whereas the indirect consequences are due to the latter and result in a decrease in or complete loss of the system and its functionality. Thus the impacts of the two are varied and may both be categorized under human, social, economic and environmental. *Table 2-8* provides a summary of the manner in which Ryall (2010) and Koh (2012) suggest consequences of bridge and culvert failures may be grouped.

Table 2-7: Categorization of Bridge and Culver Failure Consequences

	Consequence Category	**Description**
1.	Human Factors	Human factors include injuries, fatalities and psychological damage.

2.	Social Implications	The stakeholders or authorities who are considered accountable for the failure may lose their reputations and the public may no longer have confidence in them.
3.	Economic Factors	Economic factors include replacement/ repair costs, loss of functionality and consequently traffic delays and re-routing costs, traffic management costs, clean-up costs, rescue costs, loss of production and/or business for entrepreneurs expecting or delivering goods, investigations, compensations and infrastructure interdependency costs.
4.	Environmental Damage	Bridges and culvert failures may cause unforeseen environmental damage such as the release of pollutants from in the case of spillages, energy use and CO_2 emissions.

The direct and indirect factors may also be further categorized according to the categories of consequences as shown in *Table 2-9*. It may be noted that these factors have different units of measure which increases the complexity of quantifying the consequences. Furthermore, it is sometimes difficult to express factors such as loss of life or injuries in monetary units (Koh, 2012).

Table 2-8: Direct and Indirect Failure Consequences Considerations

Category	Direct Consequence	Indirect Consequence
Human Factors	Injuries Fatalities	Psychological damage
Economic	Repair of initial damage Replacement/ repair of initial contents (as a result of initial damage)	Demolition Replacement/ repair of nearby structures affected Disruption of services Traffic delays Traffic management and re-routing Losses in business or production Clean up cost Investigations

		Compensation
Environmental	CO_2 emissions Toxic releases	Environmental studies/ repair Environmental clean up
Social		Loss of reputation

2.4.4 Case Study on Causes of Bridge Failure

Imam and Chryssanthopoulos (2012) conducted a study based on the causes and consequences of metallic bridge failure. They investigated a total of 87 cases and found that the most recurrent causes of failure were due to design errors. *Figures 2-14 (a)* and *(b)* present the distribution of failure causes and modes for collapsed bridges in the USA and the UK from the early nineteenth century to the early twentieth century. It may be noted that design errors, limited knowledge and natural hazards account for almost two thirds of the failures of which just over half of these could have been prevented. Post the year 1991, it was found that limited knowledge was no longer a contributing factor however natural hazards and design errors were still found to be the main causes of bridge failure.

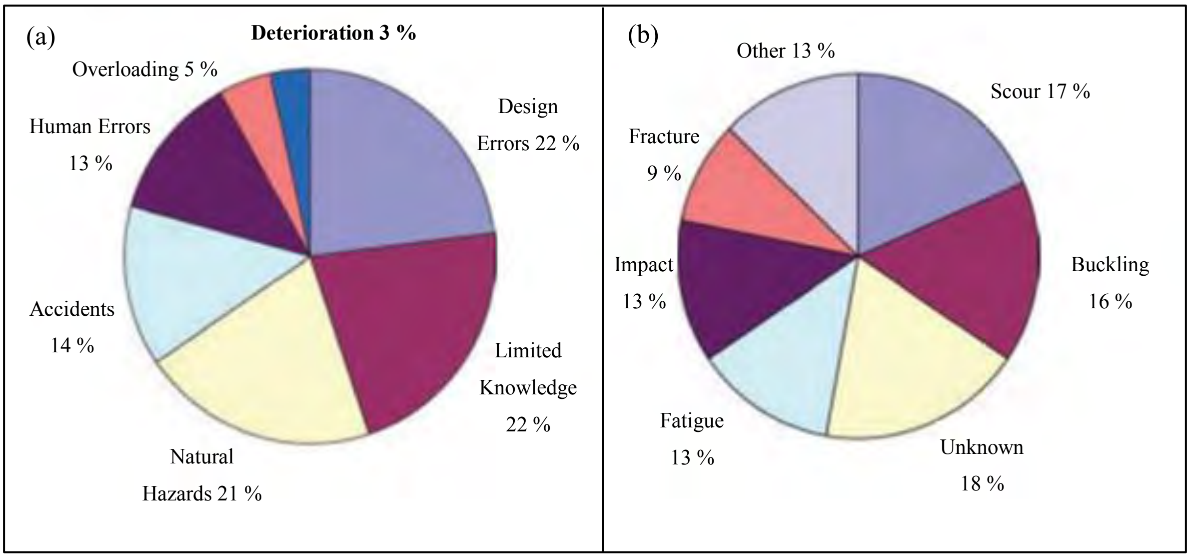

Figure 2-13: (a) Causes of Failure for Steel Bridges; (b) Failure Modes Associated with Steel Bridge Failures

Source: Imam and Chryssanthopoulos (2011)

It cannot be clearly stated why reinforced concrete bridges and culverts failed. However, much of the literature links most reinforced concrete bridge failures to durability and makes several references to failure during construction. Ryall (2010), Imam and Chryssanthopoulos (2012) as well as Moore (2011) conducted extensive reviews on bridge collapses dating as far as the mid-1800s right up to the 21^{st} century. Their findings reveal that most of the RC bridges they discuss failed due to a loss of strength as a result of concrete cracking, reinforcement corrosion and scour. The failure modes are varied from overloading to natural disasters.

2.5 Introduction to Bridge Management Systems

2.5.1 Contextualizing BM and BMSs

Bridges are designed to provide a service for a specified duration of time and because of this they cannot last forever and cannot be expected to last forever. Nonetheless measures to ensure that they are fully functional throughout their service life should be taken as with time, bridges are subjected to various types of deterioration (refer to *section 2.3*). Such measures include the practice of BM such as utilizing BMSs. BM is a term that refers to all the actions that should be conducted to ensure that a bridge remains safe to use during its design life. Thus the broad definition of a BMS is one that defines a BMS as any method or inter-linked, collective series of engineering and management functions comprising the actions and activities essential to managing a bridge network; its performance as well as its risks and expenditures over its lifecycle (Gattulli and Chiaramonte, 2005). Refining this definition, one may define a BMS as a combination of activities and a computer-based database tool which may be used for planning and monitoring infrastructure.

The system is based on inventory information of a bridge network and the condition of the bridges in a network. This information is collected through visual inspections and existing inventory data and drawings. The main purpose of a BMS is to facilitate the storage, manipulation, management and recovery of information pertaining to bridges, culverts, gantries, retaining walls etc. According to Wium and Rautenbach (2004) BMSs serve as a database and contain relevant inventory, condition and maintenance data and transport agencies therefore utilize BMSs to inform decision-making. The information assists them with (Nordengen, 2012):

1. Early detection and diagnosis of structural defects.
2. Determining and understanding maintenance needs of the bridges and/or culverts being managed.
3. Prioritising maintenance and rehabilitation activities in terms of their state of deterioration significance relative to the overall transportation network.
4. Allocating funds in a structured manner such that the cost benefit ratio is optimised.
5. Predicting future budget requirements and bridge network conditions.

6. Assessing the value of the bridges and/or culverts by assigning certain performance related indices to them after inspections.

2.5.2 Contents, Limitations of and Opportunities for BMSs

Ryall (2010) indicates that for the smooth functioning of any BMS, the BMS database should contain as much relevant information about the bridge and/or culvert as possible. Of course, the quantity of information is limited to the size and complexity of the system. Nonetheless, all systems should have basic modules which are an inventory module; an inspection module; a condition module; a costs module and a maintenance module.

The limitations of BMSs include threats to cities such as terrorists who may gain access to BMS data and plan attacks hence it is closely guarded and restricted. Shirolé (2010) suggests that the on-going revolution of technologies has gained significant momentum in the past decade. It is predicted that beyond the year 2020 advances in BMSs will include advances in bridge data storage and retrieval capabilities, security and access to sensitive information as well as other improvements to the database applications. Hearn *et al.*, (2009) indicates that in the future BMSs will focus on quantitative assessments of bridge performance and conditions as opposed to visual inspections and condition ratings. A number of wireless sensors powered by a wireless network will be used to collect data that will be analysed automatically by computer applications. This will address the matter of uncertainty and bias in the data as well as reduce the cost of data collection.

Furthermore, numerous analytical tools have been made available for infrastructure management; however their effectiveness has been supressed by the lack of available historical bridge data. It is expected that by 2020 information pertaining to bridge element condition, factors affecting rates of deterioration and costs as well as information pertain to maintenance and service life improvements will be available to address such limitations (Shirolé, 2010).

Lastly, some BMSs contain only bridge and culvert data. In the future, most BMSs are expected to be integrated with other information systems such that data is combined and analysed on a much larger, more detailed scale. Examples of such include the South African National Roads Agency Limited's (SANRAL's) Road Transportation Management System (RTMS) and the Namibian Roads Management System (RMS). Integrating information systems will also enhance communication and coordination within various divisions of an organisation (Hearn *et al.*, 2009).

2.6 Types of Bridge Management Systems

2.6.1 Introduction

Transportation agencies are accountable and responsible for keeping road structures within their respective areas of jurisdiction accessible, safe and reliable. Many have therefore developed and adopted various BMSs. Consequently, there are numerous types of BMSs currently being implemented throughout the globe. This section intends to provide a short review on four BMSs employed by different transportation authorities so as to provide an indication of the types of BMS technology available and how they came about.

2.6.2 The Pontis BMS

Minchin *et al.* (2006) stated that the Pontis BMS was established by the Federal Highway Administration (FHWA) in 1990. In 1994 it then become a product of the American Association of State Highway and Transportation Official (AASTHO).

Minchin *et al.* (2006) described the system as a computerized Windows programme that executed programme simulations to determine the infrastructures current condition, as well as predict future performance. It was also said to be able to record bridge inventory and inspection data, replicate structural conditions and produce a preservation policy and an overall bridge programme. As a result of its ability to characterize the entire structure as a set of elements and assess each elements condition, it has been able to provide detailed inspection data. According to Ryall (2010), it may be noted that its ability to optimize costs by means of an incremental benefit/cost logarithm has made it of great value. *Figure 2-15* depicts the manner in which information flows in the system.

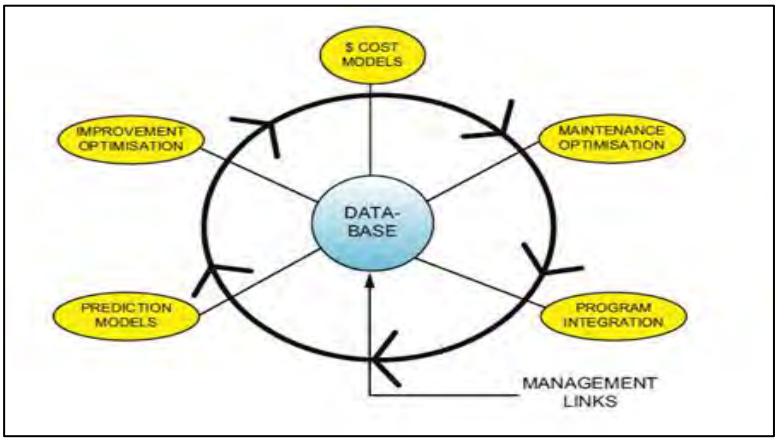

Figure 2-14: Information flow in the Pontis BMS

Source: Ryall (2010)

2.6.3 SMIS

The Highways Structures Management Information System (HiSMIS) was developed in the United Kingdom during the year 1990 and thereafter adapted and modified and renamed the Structures Management Information System (SMIS). The HiSMIS was reported the most popular BMS within the country (Ryall, 2010). It was a computerized BMS consisting of five input modules and six output modules. The input modules were controlled via the system administration module and the output modules via the enquiry and reporting suite (Ryall, 2010).

The systems versatility enabled it to not only store bridge data but to also store data pertaining to retaining walls, lighting systems, flood ways and various other types of infrastructure. Over time, it was realized that the enormity and nature of the data required by the system resulted in it being inefficient. The system was later improved and thereafter called the SMIS which was more efficient and more suited to meet their infrastructure management objectives. *Figures 2-16* and *2-17* show the configurations of the HiSMIS and the SMIS, respectively.

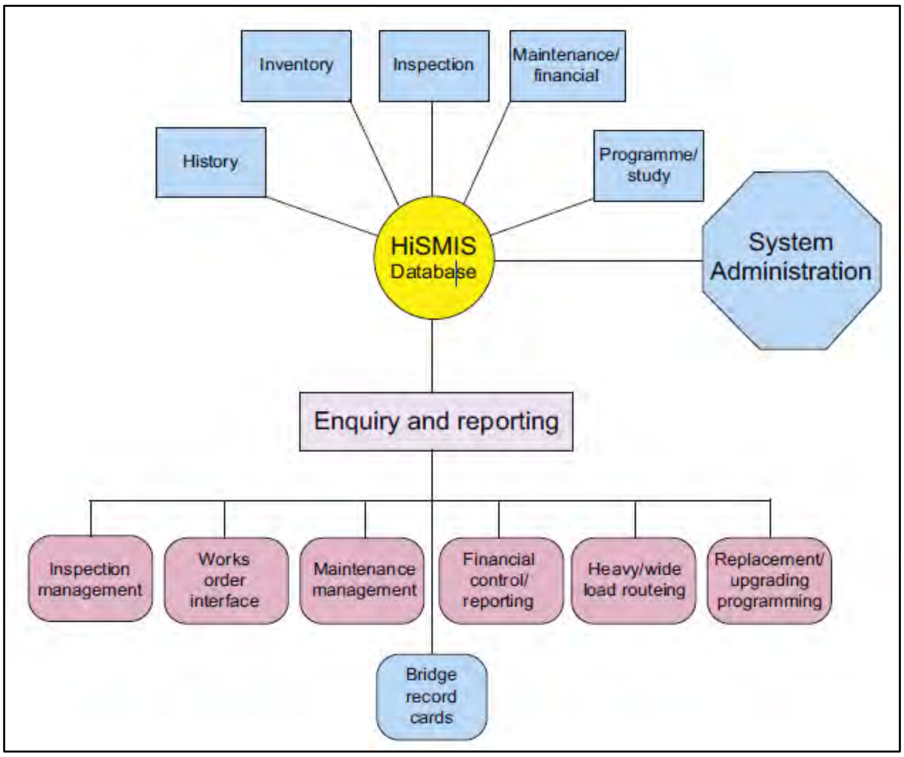

Figure 2-15: Information flow in the HiSMIS

Source: Ryall (2010)

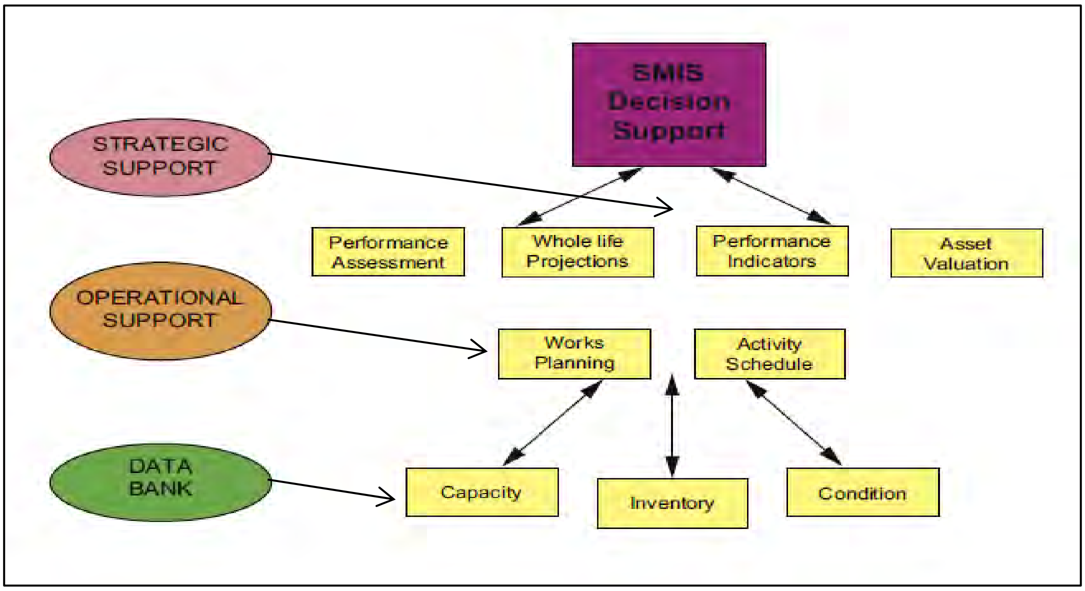

Figure 2-16: Structure of SMIS

Source: Ryall (2010)

2.6.4 Pennsylvanian BMS

The Pennsylvania BMS has been functional since 1986. This system was developed by McDonnell Douglas Professional services, together with the Pennsylvania Department of Transport (PennDOT) and is unique to the State of Pennsylvania environment (Minchin *et al.*, 2006).

During bridge inspections, maintenance needs for each bridge are identified and thereafter, each need is prioritized (Rossow, 2013). The Pennsylvania BMS, ranks the infrastructure on a scale ranging from 0 to 100. The system then assesses the bridges and proceeds to rank them with regards to their need for maintenance, repair and rehabilitation (Minchin *et al.*, 2006). Also, the system employs a model to predict the future condition of the bridges and culverts. Unfortunately the system is not able to predict preservation maintenance needs and improvement needs. However, the system has been developed to interface with the Pontis BMS. Its elements are shown in *Figure 2-18*.

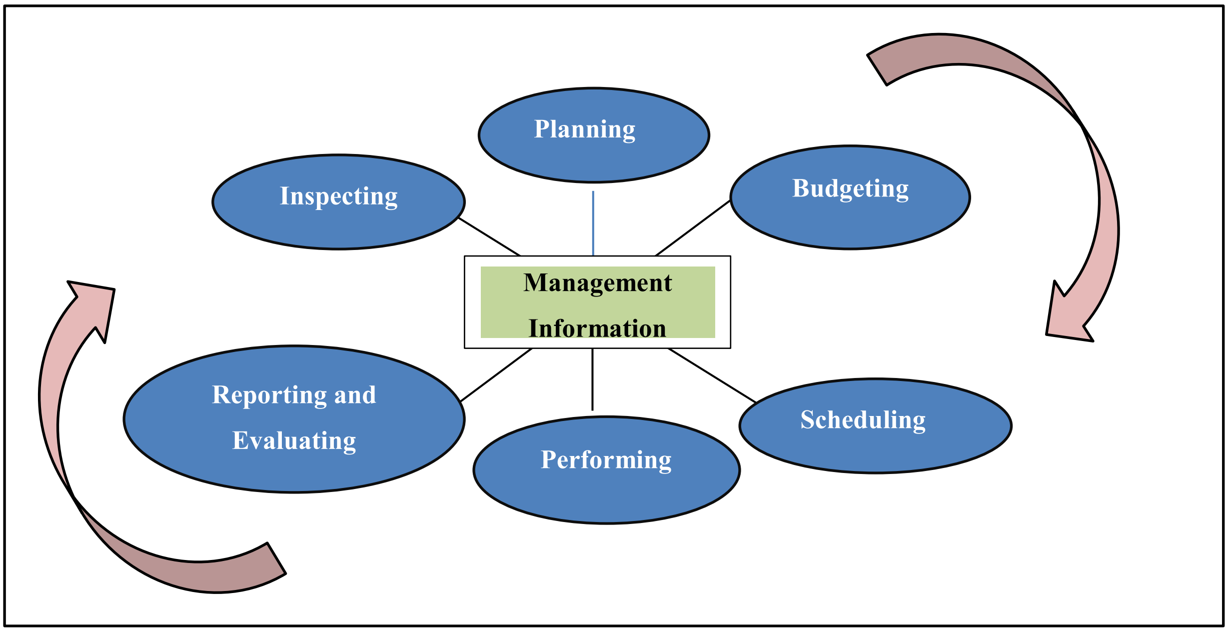

Figure 2-17: Elements of the Pennsylvania BMS

2.6.5 COSMOS

The Surrey County Council developed the Computerised System for the Management of Structures (COSMOS) during 1980 and later modified it. Initially the system was partly paper and partly electronically based. Later it became more a more electronically based database and proved to be useful for transportation agencies with limited resources managing bridge networks. *Figure 2-19* shows a representation of the information flow within the modified system. The system evaluated maintenance requirements according to a maintenance priority factor algorithm that was based on structural location, structural elements, the road link as well as the importance of the link (Ryall, 2010).

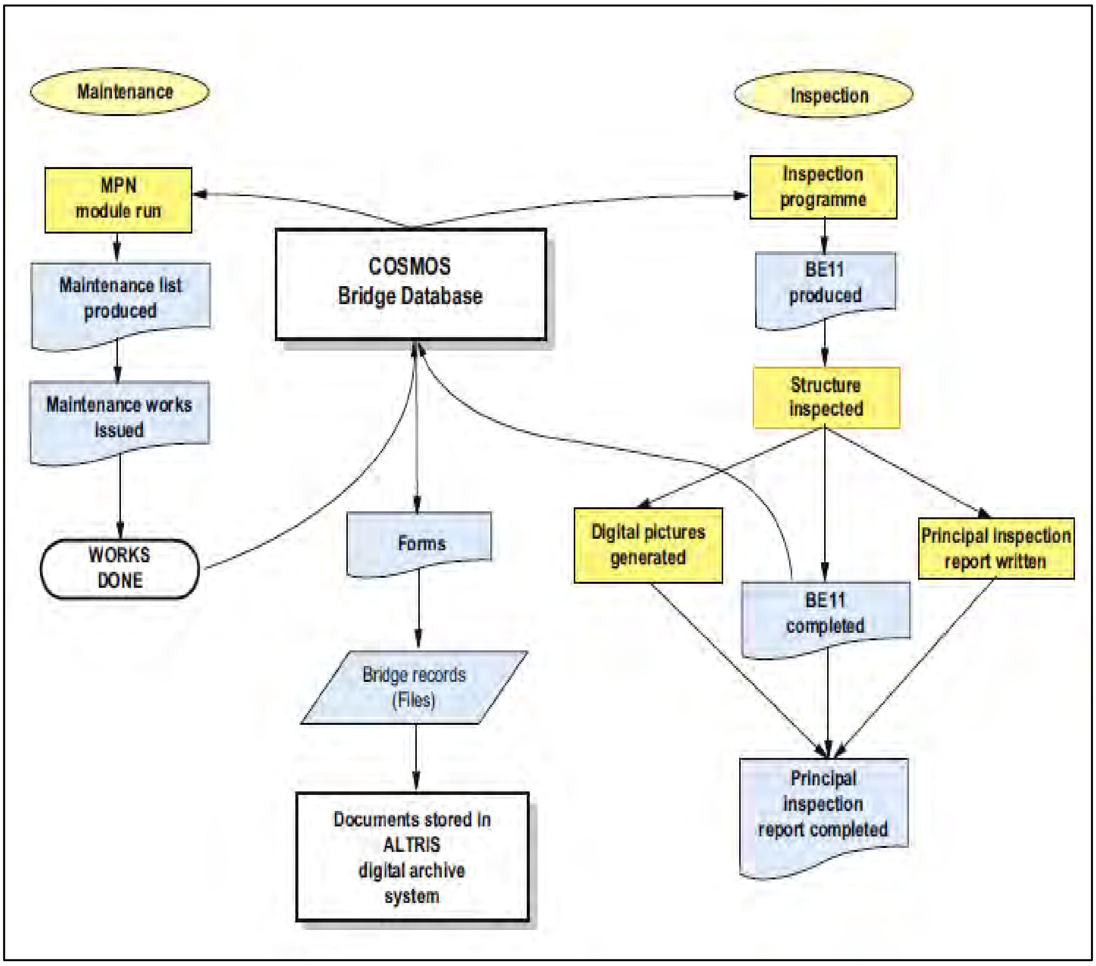

Figure 2-18: The Modified COSMOS

Source: Ryall (2010)

2.7 The Struman Bridge Management System

2.7.1 Introduction

Referring back to *Chapter 1*, the Struman BMS was initially developed by the Division of Roads and Transport Technology of the CSIR in partnership with Royal HaskoningDHV during 1995, using existing BMSs previously developed for the City of Cape Town and Spoornet (Nordengen and De Fleuriot, 1998). The system is a computerised database that consists of five modules namely the inventory; inspection; condition; budget and maintenance modules. It utilizes visual assessment data to prioritize maintenance projects. The BMS is currently a standalone system and has been used for the analysis of bridge data only however, according to Ryall (2010) in the near future, it is expected to be integrated with management systems such as the Highway Management System (HMS), the Traffic Management System (TMS), the Traffic Observation Management System (TOMS) as well as other management systems so as to better facilitate the management of asset information.

2.7.2 System Components

As in most BMSs, the Struman BMS consists of the inventory, inspection, condition, budget and maintenance modules. These modules are informed by each other and are therefore interlinked as shown in *Figure 2-20*.

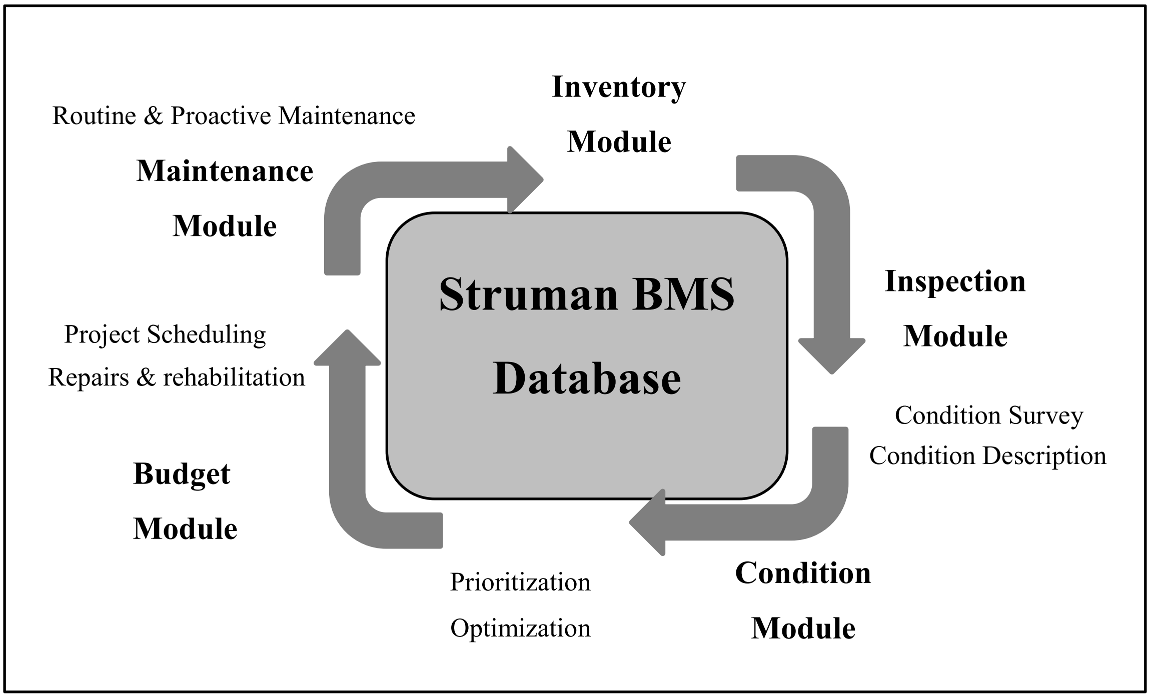

Figure 2-19: Information Flow in the Struman BMS

Inventory Module

The inventory module is the most basic module of the BMS, consisting of a list of all the structures in the network as well as comprehensive inventory data of each structure type.

The main components of this module are:

1. Location details
2. Contract details
3. Structural features
4. Hydraulic data
5. Dimensions and geometry
6. Service details
7. Road configuration and traffic volumes
8. Archive details
9. Rehabilitation history
10. Factors influencing field inspections
11. Inventory photos

Inspection Module

The inspection module is an archive of data collected during visual inspections. It is the main distinction between other BMSs and the Struman BMS. According to Nell *et al.* (2008) the inspection module focuses on the observed defects on the elements as opposed to rating the overall condition of each element. It may be noted that culverts, gantries, retaining walls etc. are sub-divided according to their unique structural elements and that the number of elements vary for the various structures. For example a bridge is sub divided into the 21 elements listed in *Table 2-10.* These are individually assessed and rated. It may be noted that the inspection sheet shown in *Figure 2-21* is used to document the on-site inspection findings.

Table 2-9: Inspection Item list

Inspection Items		
1. Approach embankment	8. Surfacing/ ballast	15. Bearings
2. Guardrail	9. Superstructure drainage	16. Support drainage
3. Waterway	10. Kerbs/ Sideways	17. Expansion joints
4. Abutment foundations	11. Parapet/ handrail	18. Longitudinal members
5. Abutment	12. Pier protection works	19. Transverse members
6. Wing/ retaining walls	13. Pier foundations	20. Deck slab
7. Kerbs/ Sidewalks	14. Piers and columns	21. Miscellaneous items

SA National Roads Agency Ltd BRIDGE MANAGEMENT SYSTEM		BRIDGE Field Inspection Sheet		No.	N001_01N_B699	
				Name	Agter Paarl Road over Road Bridge	
				GIS: X		
				Y		
Inspection Type:		**Inspector**	**Firm**	**Date**	**Route/Section**	N001 / 01N
Current	PR	M Smuts	VKE CTN	07-May-99	**Route km**	47.29
Last Principal	PR	M Smuts	VKE CTN	07-May-99	**Other Bridge No**	4453
Last Monitoring	MO				**N Route Over/Under**	Under
Last Maintenance	MA				**Feature Name**	Agter Paarl Road
Last Verification	VE				**Feature Rd No**	

Bridge Type	Simply supported	**No of spans**	4	**Min Vertical Clearance**	**Pos/Span**	NBC /	NBC /	SBC /	SBC / Right
Year constructed	01-Jan-70	**Overall length**	112.4		**Min height**	8.395	7.5	6.33	5.21
Bridge orientation	North/South	**Angle of skew**	58	**Direction of river flow**					
Time (Hours) Inventory	0	**Inspection**	0	**Reporting**	0	**Capturing**	0		

INSPECTION ITEM		D	E	R	INSPECTION ITEM		D	E	R	INSPECTION ITEM	D	E	R
1. Approach Embankment	N A	2	1	1	5. Abutment Foundations	N A	U			9. Superstructure Drainage	X		
	S A	0				S A	U						
2. Guardrail		2	1	1	6. Abutments	N A	3	3	3	10. Kerbs/ Sidewalks	0		
						S A	3	3	3				
3. Waterway		X			7. Wing/Retaining walls	N A	3	2	2	11. Parapet	3	3	2
						S A	3	2	2				
4. Appr.Emb. Prot.Works	N A	0			8. Surfacing		0			21. Miscellaneous Items	X		
	S A	0											

SUPPORTS	12 Pier Protection Works			13 Pier Foundation			14 Piers & Columns			15 Bearings			16 Support Drainage			17 Expansion Joints			SPANS	18 Longitudinal Members			19 Transverse Members			20 Decks and Slabs		
	D	E	R	D	E	R	D	E	R	D	E	R	D	E	R	D	E	R		D	E	R	D	E	R	D	E	R
A S	-	-	-	-	-	-	-	-	-	3	4	2	U			4	4	3	S 1	4	3	3	X			3	2	2
A N	-	-	-	-	-	-	-	-	-	3	4	2	U			4	4	3	S 2	4	3	3	X			3	2	2
P 1	0			U			3	2	2	2	1	1	U			4	4	3	S 3	4	3	3	X			3	2	2
P 2	0			U			3	2	2	2	1	1	U			4	4	3	S 4	4	3	3	X			3	2	2
P 3	0			U			3	2	2	2	1	1	U			4	4	3										

Item	Position	Activity	Qty	Unit	U	MS	Remarks	Monitor Freq	Photos
17.	AL	2. ? Replace concrete nosing	90	m	2	No	All expansion joints are leaking - to be replaced	0	23-27
18.	AS	2. Seal, repair cracks > 0,3 mm	380	m	4	No	Major longitudinal cracks in soffit - 10mm max	0	28-38
18.	AS	4. Apply protective coating	850	m2	2	No	Pattern cracking due to AAR	0	32-39
18.	AS	6. Clean concrete surface	850	m2	2	No	Concrete stained	0	28-39
19.	BA	2. Seal, repair cracks > 0,3 mm	8	m	4	No	Horizontal cracks	0	40,41
19.	BA	4. Apply protective coating	25	m2	2	No	Pattern cracking due to AAR	0	40,41
19.	BA	5. Clean concrete surface	25	m2	2	No	Concrete stained	0	40,41
20.	AS	2. Seal, repair cracks > 0,3 mm	5	m	2	No	Cracks	0	43-45
20.	AS	4. Apply protective coating	250	m2	2	No	Pattern cracking due to AAR	0	42-45
20.	AS	7. Clean concrete surface	250	m2	2	No	Concrete stained	0	42-45
20.	S2	1. Repair spalled concrete	0.5	m3	1	No	None	0	42
1.	NA	4. Inlets/outlets - clean	1	no	1	No	Inlet blocked	0	01
1.	NA	10. Side drains - clean	10	m	1	No	Vegatation on verge	0	02
2.	P1,P3	2. Replace rails	15	m	1	No	Collision damage	0	03,04
6.	BA	9. Apply protective coating	26	m2	2	No	Pattern cracking due to AAR	0	05-08
6.	BA	13. Clean concrete surface	26	m2	2	No	Severe staining	0	05-08
7.	AL	3. Seal, repair cracks > 0.3 mm	4	m	2	No	Horizontal cracks	0	10
7.	AL	7. Apply protective coating	6	m2	1	No	Pattern cracking due to AAR	0	09-11
7.	AL	13. Clean concrete surface	6	m2	1	No	Staining	0	09-11
11.	AL	12. Reconstruct parapet (Not NJ)	270	m3	2	No	Pattern cracking due to AAR	0	12,13
11.	W	20. Replace steel/aluminium handrail	6	m	1	No	Collision Damage	0	14
14.	AP	4. Apply protective coating	280	m2	2	No	Pattern cracking due to AAR	0	15-19
14.	AP	7. Clean concrete surface	280	m2	2	No	Concrete stained	0	15-19
14.	P1	1. Repair spalled concrete	0.5	m3	1	No	Western column	0	15
14.	P2	2. Seal, repair cracks > 0,3 mm	6	m	2	No	Verticle cracks	0	17,18
15.	AL	8. Clear obstructions to movement	70	no	1	No	Clean gap around bearings	0	20-22

Inspector's assessment of structure condition and further comments:

Major longitudinal cracks in deck soffit - up to 10mm wide - needs urgent attention.
All exposed concrete surfaces are stained and covered with pattern cracking due to AAR.
All exposed concrete surfaces to be painted with a protective coating.

Further inspection needed ? Y/N	No	**IF FURTHER INSPECTION REQUIRED IS Y:**
Was UBIU used ? Y/N	No	Then please indicate any special requirements ie. 6m Ladder, Bush
Is the UBIU needed for future insp's? Y/N	No	cutting, UBIU, better weather etc. If nothing please state "none"

D - DEGREE							E - EXTENT				R - RELEVANCY				U - URGENCY					
NA	UA Insp	None	Minor	Fair	Poor	Severe	Local	>Local	<Gnl	General	Min	Moderate	Major	Critical	Record	Monitor	Routine	<5 yrs	<2 yrs	ASAP
X	U	0	1	2	3	4	1	2	3	4	1	2	3	4	R	0	1	2	3	4

Figure 2-20: Sample Inspection Sheet

Condition Module

The bridges and culverts in the condition module are prioritised for repair and rehabilitation based on their Condition Index (CI) and Priority Index (PI); parameters predetermined by the database itself. Roux *et al.* (2001) as well as Nell *et al.* (2008) mention that the CI and PI are derived from the ratings assigned to the elements of the structure during the inspections. The CI takes into account the overall structures' physical condition (as opposed to their need for maintenance) whereas the PI accounts for the structures ability to function by considering the defects on the structures critical elements. Structural elements such as abutments, piers and decks with high degree (D) and relevancy (R) ratings have adjustable weighing factors built into the database's prioritization algorithms to ensure that essential items have a larger impact on the on the Priority Index (PI). The CI and PI range from 0 to 100 and the higher the rating the better the condition of the structure (see *Table 2-11*). Lastly, the Functional Index (FI) combined with the PI are used to evaluate the Overall Priority Index (OPI) which determines the strategic significance of the structure and/or route and hence its importance (Roux *et al.*, 2010).

Table 2-10: PI and CI Level Indicators

Condition Category	Index Range	Condition Index (CI)	Priority Index (PI)
Very Good	85 – 100	Structure is still new and in very good condition.	Structure is considered in excellent condition.
Good	70 - < 85	Structure requires only routine maintenance to retain its condition.	Structure is considered in good condition with very limited maintenance requirements.
Fair	50 - < 70	Some clearly evident deterioration. Structure requires preventative maintenance or renewal of isolated areas.	Structure is considered at warning level and is prioritized.
Poor	30 - < 50	Structure requires significant renewal/ rehabilitation to retain its structural integrity.	Structure is considered critical and given a high priority.
Very Poor	0 - < 30	Structure is not safe to use and exhibits significant deterioration. Thus it will require substantial renewal or upgrading and has less than 10 % of its estimated useful life (EUL) remaining.	

Adapted from: Roux *et al.* (2010) and Committee of Transport Officials (2013b)

Budget Module

Nordengen and Nell (2005) suggest that the unit costs of the remedial work activities assigned to the bridges and culverts during inspections are used in the budget module to estimate the total cost of repair activities for each structure. The budget is optimized by comparing the estimated cost of repair with the relevancy of the defect through a benefit-cost ratio. Structures are reprioritized based on this as it is said to be more beneficial to repair items that have high reductions in risk for road users and are estimated to be low cost. It may be noted that the budget module is able to overwrite the optimisation procedure when structures are manually prioritized. Lastly, repairs are then allocated to the current budget year or the subsequent budget years.

Maintenance Module

The maintenance module contains a detailed account of all the maintenance activities that have been carried out and completed. A preventative maintenance section also exists (usually routine in nature) and informs the bridge manager of the work to be done on certain structures should the bridge maintenance team carry out any other additional remedial works.

2.7.3 The DERU Rating System and the CI

The visual inspections are conducted in accordance to a 4-point defects based system, known as the DERU Rating System (defined in *Table 2-12*). The system requires the bridge inspector to assign a rating (between 1 and 4) to the worst defect (highest relevancy or highest degree for the same relevancy) identified on the bridge/culvert element. This then becomes the rating of the inspection element. The defect is rated based on its Degree (D), Extent (E) and Relevance (R). Consequently, based on the findings an Urgency (U) rating for proposed remedial work activities is assigned to the defect. The latter takes into account all possible future events which could have an adverse effect on the defect and deduces the time limit or rather urgency for the repair procedures. Although not all the defects are rated in terms of DER, it may be noted that they are all recorded and assigned urgency ratings for repair activities.

Furthermore, by rating the degree and extent of each defect separately more accurate deterioration rate calculations may be carried out as the actual variation in the degree and extent of defects is easily observable from one inspection to the next. The relevancy rating provides an indication of the consequence of the defect with respect to the structural integrity and safety. It may also be used to optimize the budget based on the reduction in risk for the user (Nordengen and De Fleuriot, 1998).

Table 2-11: The DERU Rating System

Category	Description	Rating
Degree (D)	The severity of the defect.	U - Unable to inspect 0 - No defect 1 - Minor 2 - Fair 3 - Poor 4 - Severe
Extent (E)	The prevalence of the defect on the inspection item.	1 - Local 2 - More than local 3 - Less than General 4 - General
Relevancy (R)	The consequences of the defect with regards to the structures safety and integrity.	1 - Minimum 2 - Moderate 3 - Major 4 - Critical
Urgency (U)	'Time' limit on repair requirements.	X - Make Safe U - Record 0 - Monitor 1 - Routine 2 - < 10 years 3 - < 5 years 4 - ASAP

Adapted from Nordengen (2012)

The value of the CI is derived from the inspection data. Firstly the CI value (Ic_{ij}) of each sub-item of the 21 main structure items (recall *Table 2-10*) is calculated as follows (Li *et al.*, 1998):

$$Ic_{ij} = 100 - 100\frac{(D+E)\,R^a}{(4+4)4^a} \qquad \text{... (2.5)}$$

Where D, E and R are the values associated with the Degree, Extent and Relevance associated with defect under consideration on the sub-item, correspondingly. The parameter *a* usually ranges between 1 and 2 and is determined by the significance and importance of the highway in which the structure is located.

The CI for the 21 sub-items is then calculated as follows:

$$Ic_i = \frac{\sum Ic_{ij}}{n} \qquad \text{... (2.6)}$$

Where n is defined by the number of sub-items (i.e. 21). Finally, the CI of the may be calculated using:

$$\text{CI} = \frac{\sum Ic_{ij} \text{x } w_i}{\sum w_i}, \qquad where \sum_{i=1}^{21} w_i = 100 \qquad \dots (2.7)$$

Where w_i is each inspection sub-items weighing.

2.8 Database Analysis of the Namibian Struman BMS

2.8.1 Introduction

Roux *et al.* (2001) prepared a paper describing the development and implementation of the Struman BMS for the Namibia Roads Authority (NRA). In doing so provided a few statistical summaries of general inventory information on the bridges in Namibia. A total of 1 430 road structures were inspected (483 bridges and 947 culverts). In addition to these 1 290 large culverts were found. They were summarised per maintenance region and then in the context of Namibia.

2.8.2 Database Analysis

The combined length of the bridges was approximately 27 km and the culverts 9.5 km. The average bridge length was 48 m and approximately 96 % of the bridges in Namibia had a maximum span length not greater than 20 m. It was also found that 95 % of the culverts had a span length between 0 m and 5 m. *Table 2-14* shows the number and class of structure within the various Namibian maintenance regions.

Table 2-12: Class and Number of Structures per Maintenance Region

Structure Class	Maintenance Region				Namibia
	Keetmanshoop	Oshakiti	Otjiwarongo	Windhoek	
Large Bridge	41	44	27	45	**157**
Medium Bridge	103	31	82	101	**317**
Small Bridge	24	6	30	28	**88**
Sub-total Bridges	**168**	**81**	**139**	**174**	**562**
Major Culverts	207	120	281	260	**869**
Large Culverts	444	113	426	307	**1289**
Total	**819**	**314**	**846**	**741**	**2720**

An age analysis of the bridges and culverts was conducted (refer to *Figure 2-22*). The analysis showed that most of structures were constructed during the period of 1960 -1980

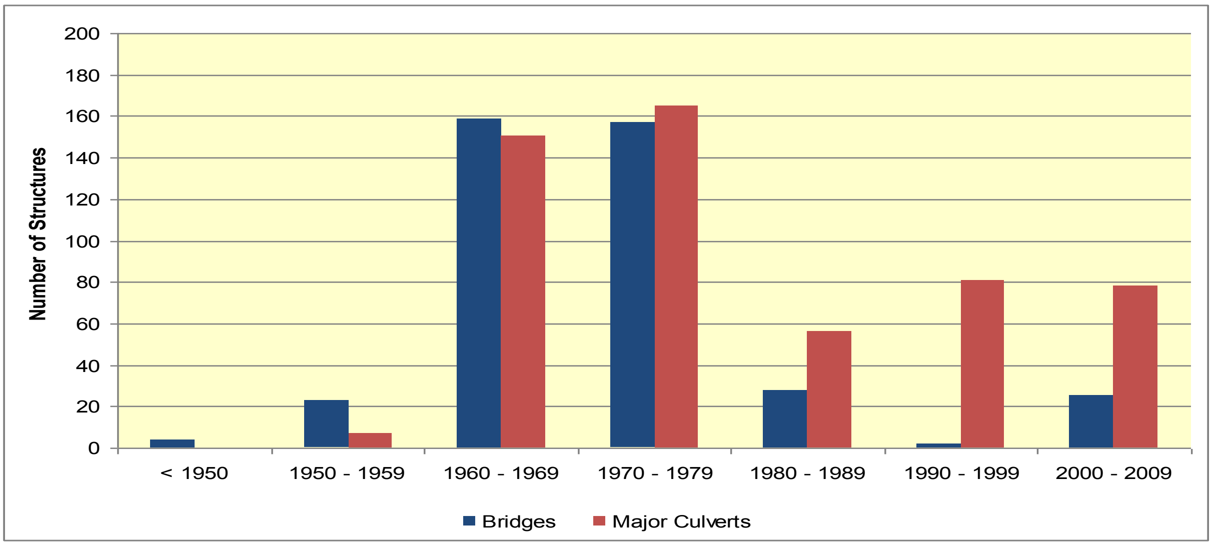

Figure 2-21: Distribution of Year of Construction of Structures

Source: Roux *et al.* (2001)

The condition ratings of all the structures were then investigated (see *Figure 2.24)*. Findings showed that approximately 200 structures had a condition ratings less than 70 and a PI ratings less than 87. The average CI ratings for all bridges and culverts were calculated to be 79.8 and 84.3 respectively, indicating that most of the structures were still in fairly good condition. Oshakiti, the maintenance region with the largest quantity of recently constructed structures had the highest structural average CI in Namibia, recorded to be 92.0 (Roux *et al.* 2001). The 200 structures in the warning and critical conditions were located mainly in the two southern maintenance regions, Keetmanshoop and Windhoek.

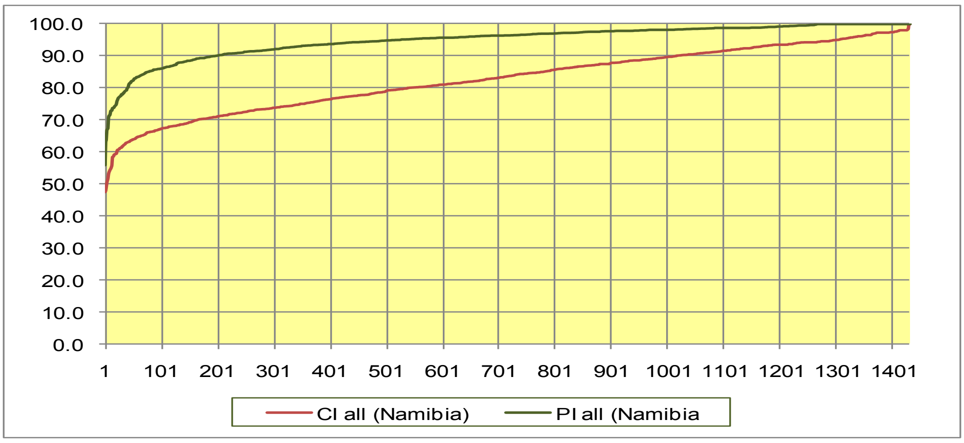

Figure 2-22: Condition Ratings of all Inspected Structures

Source: Roux *et al.* (2001)

Finally repair costs, based on urgency were estimated for each region *(refer to Figure 2.26)*. According to Roux et al (2010), the total costs of repairs and rehabilitation for the structures of Namibia were calculated to be N$ 152 million, of which N$ 106 million would be assigned to bridges and N$ 46 million for major culverts.

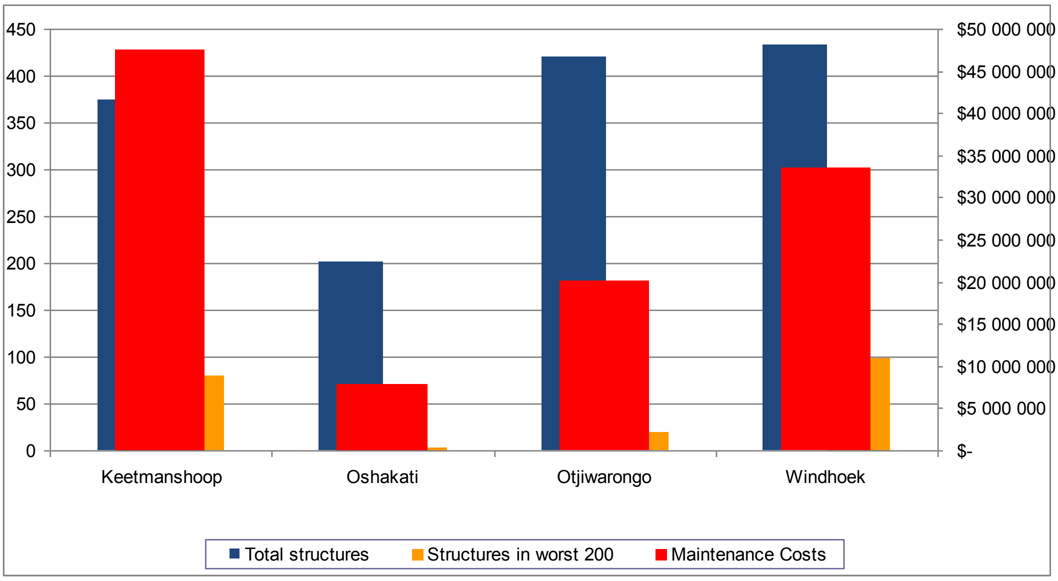

Figure 2-23: Estimated Maintenance Costs per Maintenance Region

Source: Roux et al (2001)

Roux *et al.* (2001) used the Struman BMS database to extract useful information pertaining to bridges and culverts in Namibia so as to gain a better understanding of their state. However, there were large gaps in the paper as the types of structures found in the various regions were not mentioned, nor were the defects found on the structures investigated. Thus the predominant structural defects as well as their causes and associated deterioration mechanisms are not known.

The Struman BMS has not only been implemented in Namibia and the Western Cape Province, several road and rail authorities across Southern Africa have also adopted this system. These are listed in *Table 2-15* (Nordengen, 2012).

Table 2-13: Implementation of the Struman BMS across Southern Africa to Date

Road and Rail Authorities
1. The Botswana Roads Department
2. The cities of Johannesburg, Cape Town, Port Elizabeth and Pietermaritzburg
3. The Kwa – Zulu Natal Department of Transport
4. SANRAL

5. The Taiwan Area National Freeway Bureau
6. The Western Cape Provincial Administration
7. Mangaung Metro
8. Mpumalanga Provincial Government
9. N3 Toll Concession Ltd, TRAC & Bakwena
10. Namibia Port Authority (NamPort)
11. Namibia Roads Authority
12. Nelson Mandela Metro,
13. Sasol (Secunda)
14. Swaziland Ministry of Public Works & Transportation
15. Spoornet – The South African railway authority

2.8.3 Summary

It may be concluded that the causes of structural defects and consequently deterioration may be grouped into three main topics, namely mechanical; physical and chemical. Nonetheless the phenomena are interrelated as multiple deterioration mechanisms may take place simultaneously. Moreover, some often result in or accelerate the others. Furthermore, regular maintenance and inspections serve as effective means to ensure that a structure achieves an acceptable level of performance for the duration of its Estimate Useful Life (EUL).

The costs associated with bridges and culverts may be estimated through WLCC as it takes into consideration the bridge or culvert construction costs, maintenance costs, etc. which should be accounted for during its required service life. However, these costs do not take into account costs that would arise in the event of an unexpected failure occurring before the structure reaches the end of its service life. These costs may be determined by attempting to quantify the consequences of failure should the structure fail at random. However, in attempting to do so, it is more difficult than it is convenient as some of the consequences are intangible and are of differing measurement units. Attention should be given to unforeseen structural failures. Attempts to quantify such events should be made so as to forecast and keep records of predicted costs as their impact can be large and the consequences expensive, especially when unplanned for.

BMSs have served as useful tools in monitoring the condition and performance of structures such as bridges and culverts so as to ensure that they are safe and functional for their entire EUL. However, they rely extensively on visual inspection data and for this reason there are opportunities for their development. The BMSs discussed in this section had various levels of detail as some were more developed than others. Nonetheless they all served the same primary function, which was to prioritize structures for remedial work activities. It may be concluded that there are many well-developed BMSs throughout the world which have been modified and adapted with time.

The Struman BMS developed by the CSIR in partnership with SSI is a well-developed BMS comprising of five interdependent modules, namely the inspection, inventory, condition, budget and maintenance modules. It thus contains all the necessary components of a well-established BMS. It has been able to utilize visual assessment data collected using a defect based system, otherwise known as the DER Rating methodology, not only to prioritize structure maintenance and rehabilitation needs but to also monitor structural deterioration rates. However, it may be noted that computation of the CI does not take into consideration the structure type, material type or climatic region of the bridge or culvert structure. Thus there are opportunities to further develop and refine the CI. Over the years the system has become increasingly popular and thus adopted and implemented by a number of road and rail authorities within Southern Africa. Lastly, in the near future the system is expected to be integrated with other infrastructure management systems with the intention to improve the management of asset information and inform decision-making pertaining to asset rehabilitation and repair projects.

Chapter 3

3 Research Methodology

3.1 Introduction

The procedure implemented to investigate the relationships between predominant defects and inventory data, such as location; structure type; age and span length/width, have been detailed in this chapter. Firstly, the data was attained and verified through comparisons of inspection data to inventory photos. Thereafter, it was clustered into several categories based on similarities so as to ease the analysis procedure. Next, the set of defects from which the predominant defects would be identified were selected on the basis of available inspection data and inventory photos. Two binary matrices containing these defects were formulated for both RC bridges and RC culverts and used to identify the predominant defects on the structures. These were also used to conduct several logistic regression analyses so as to identify statistically significant relationships between the predominant defects and the inventory data. In other words, highlight independent variables that significantly impacted the presence of the dependant variables while at the same time identify those which had less of an impact. In addition to this, data mining activities were conducted to identify relationships between the predominant defects and inventory data. Lastly, the condition of the RC structures in relation to the inventory data was investigated and findings from all activities discussed thereafter.

The data used for the study was that of RC bridges and culverts in the Western Cape Province. It was collected through visual assessments by bridge inspectors during the period from 2001 to 2004. It was extracted from the inventory, inspection and condition module of the Struman BMS database (see *Figures 3-1* and *3-2*) provided by the Roads and Transport division of the CSIR Built Environment.

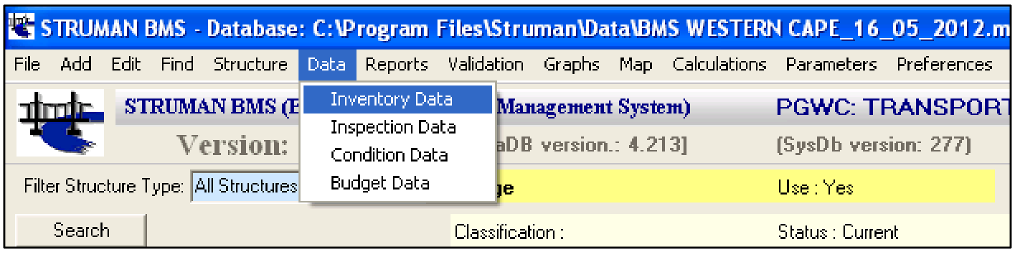

Figure 3-1: Modules of the Struman BMS

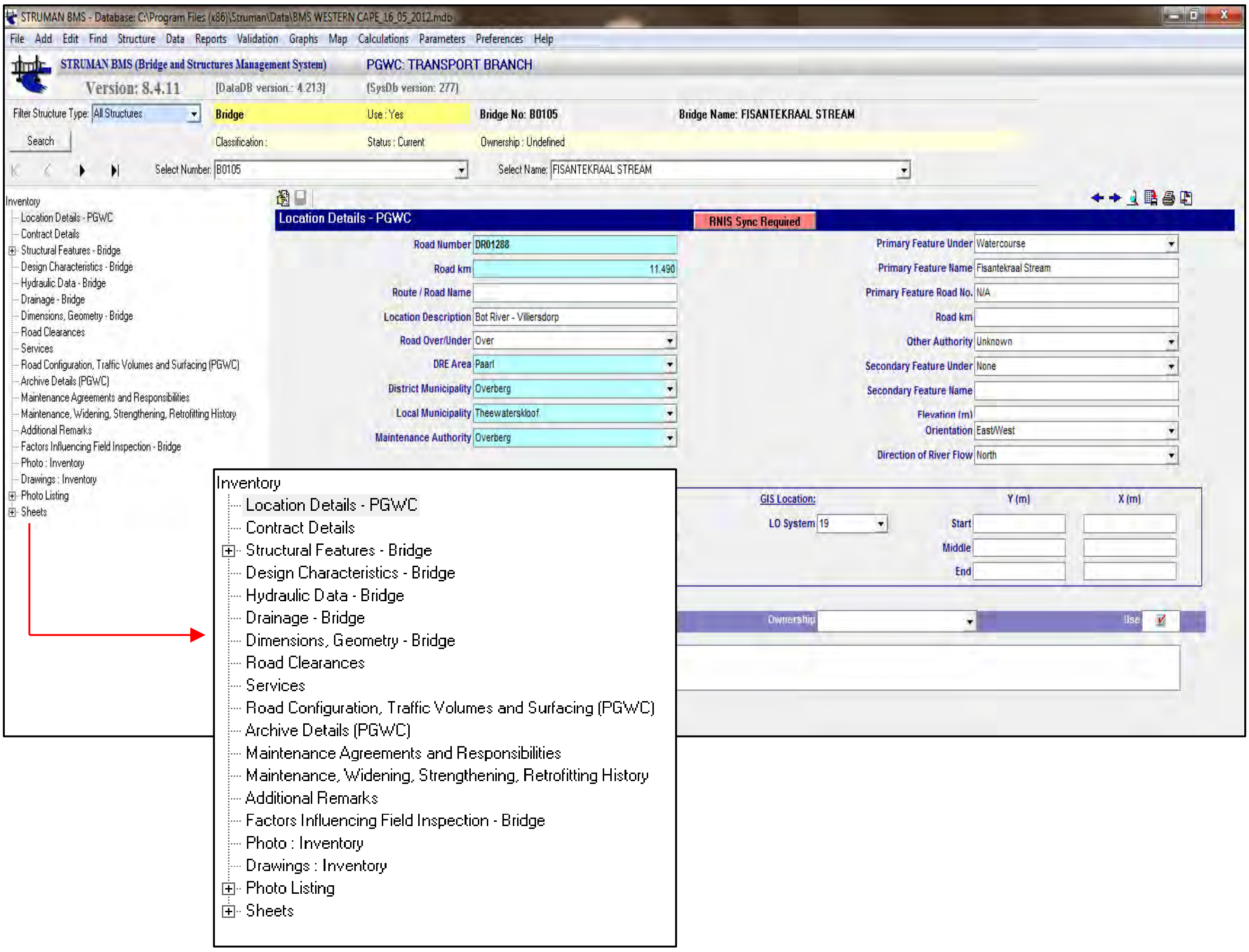

Figure 3-2: Inventory Data Module

A random sample of approximately 10 % of the RC bridges and culverts were re-inspected using the inventory photos taken at the time of inspection. This was done to ensure the inspection data was consistent with the inventory photos. This method of data validation was assumed appropriate because the timeframe in which the data had been collected (approximately 10 years ago) and the study conducted was long enough to allow for some bridges and culverts to have further deteriorated while others may have had remedial work activities. The findings were closely related to the data in the inspection module, therefore the inspection data was considered acceptable.

3.2 Raw Data Extraction, Clustering and Data Mining

The inventory data listed in *Table 3-1* was extracted from the Struman BMS database. It was from this data that each bridge and culvert's material type, location, structure type, span length/width and overall length were determined. In the case of absent information, inventory photos were used and when no inspection data or inventory photos were available the bridge or culvert was discarded. Thereafter, the inventory data was clustered as seen in *Table 3-2 and 3-3* and the RC structures were separated from the other structures.

Table 3-1: Inventory Data obtained from the Struman BMS Database

	Inventory Data
1.	Structure ID
2.	Structure Location - District Municipality - Maintenance Municipality - Road Name
3.	Structure Type
4.	Material Type
5.	Year of Construction
6.	Length of Structure - Total Length - Bridge Span Length - Culvert Width

Table 3-2: Material Type Categories

	Category	Description
1.	Composites	Concrete structures that had a combination of steel and timber and concrete or brickwork etc.
2.	Masonry	Brick work or any sort of individual units bound together by mortar.
3.	Steel and concrete	Concrete structures that had external steel elements.
5.	Reinforced Concrete	All reinforced concrete structures.

Table 3-3: Structure Type Categories

	Category	Description provided in BMS
1.	Arch	Armco Arch Armco pipe Arch Open Sprandel Arch Open Ribbed Arch Solid Sprandel filled Arch
2.	Armco Pipe	Armco Pipe
3.	Cantilever	Cantilever Cantilever with drop in span
4.	Cellular	In-situ cellular
	Concrete Pipe OG	Concrete Pipe OG Concrete Pipe Spigot and Socket
7.	Continuous	Continuous Continuous in-situ
8.	Precast Portal frame	Precast Portal Frame
9.	Simply Supported (note: *simply supported in-situ portal frame culverts are always specified)*	Simply Supported In-situ Portal frame Simply Supported
10.	Truss Girder	Lattice Truss Girder

The RC structures were then separated into bridges and culverts based on whether they had been defined as a bridge or culvert in the Struman database. This was done irrespective of the bridge or culvert criteria they satisfied so as to ensure that the findings from the analysis were consistent with the information in the database.

The defects investigated were reinforcement corrosion; cracking (structural and non-structural), including cracking due to settlement, torsion, shear stress, bearing stress, prestress restraint etc. It is acknowledged that structures that are generally in good condition are seldom expected to fail due to defects resulting from the loss of press or failure of a beam. Nonetheless, cracking due to these was investigated so as to provide a basis from which the various types of cracking may be investigated for future studies (see recommendations). Other defects under investigation included spalling; concrete erosion; approach settlement; surface erosion of deck; bearing defects; scour (including undermining); alkali-silica reaction (ASR); impact damage; honeycombing; embankment erosion; joint defects; movement and rotations and fire damage. The types of cracking were not always specified and the inventory photos were not always available, the structural and non-structural cracks were not separated. Furthermore, even though some of these defects were inter-related they were investigated individually as found in the database.

Two binary matrices containing the defects and the inventory data, shown in *Figure 3-3,* were formulated; one for RC bridges and the other for RC culverts. The defects were dichotomous. In other words, they had two jointly exhaustive mutually exclusive outcomes; the RC structure either had the defect or did not have the defect. Using the binary matrices, the frequencies of the defects were determined and the most predominant defects (i.e. defects with the highest frequency) were identified. Thereafter, relationships between these and the inventory data (location, structure type, age and span length/width) were investigated using several Logistic Regression Analysis (LRA) models in STATA 2011. Subsequently, data mining activities were employed to investigate relationships between the predominant defects and inventory data.

Finally, the average condition indices were investigated relative to the inventory data so as to ascertain whether the findings from LRA and data mining were an indication of the RC structures average condition. In addition to this, descriptive statistical analyses of the average condition indices were provided to quantitatively summarize the condition index data. It may be noted that the theory of LRA is further discussed in the next section.

Defects were denoted as follows:

Let x = "*a defect*"

$$\text{Defect } (x) = \begin{cases} x \; = 1 \text{ , } when\ the\ defect\ was\ present \\ x \; = 0 \text{ , } when\ the\ defect\ was\ absent \end{cases}$$

Independent Variables

Dependant Variables

								DEFECTS														
No.	Structure ID	District	Material	Age	Structure Type	Longest Span	Overall length	Corrosio	cracking	Spalling	Conc Erosio	App Sttlmnt	Surf erosio	Bearing	Scour	AAR	Impl Dam	ion Con	mb errosio	Joints	Mov/ Ro	Fire
1	B0967	CITY OF CAPE TOWN	Reinforced concrete	32	Simply supported	11.3	42.3	0	1	1	0	0	0	0	0	1	1	0	1	1	0	0
2	B2167	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	7.3	371	0	1	1	0	0	1	1	0	0	1	0	0	1	0	1
3	B2168	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	7.3	48.2	0	1	1	0	0	0	1	0	0	0	0	0	1	1	0
4	B2169	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	7.9	64	1	0	1	0	0	1	0	0	0	0	0	0	0	0	0
5	B2170	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	15.7	62.8	0	1	0	0	1	1	1	0	0	0	0	0	0	0	0
6	B2170A	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	15.7	62.8	0	1	1	0	0	0	1	0	0	0	0	0	0	1	0
7	B2171	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	7	51.6	1	1	1	0	0	0	0	0	0	0	0	0	0	0	0
8	B2172	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	16.3	54.7															
9	B2175	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	7.5	15	0	1	1	0	0	0	1	0	0	0	0	1	0	1	0
10	B2177	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	16.6	66.7	0	1	1	0	0	0	0	0	0	0	0	0	1	0	0
11	B2178	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	7.3	48.2	0	1	1	0	0	1	1	0	0	0	1	0	1	0	0
12	B2179	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	7.5	59.4	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0
13	B2180	CITY OF CAPE TOWN	Reinforced concrete	115	Continuous	7.4	52.4	0	0	1	0	0	0	0	0	0	0	0	0	1	0	0
14	B2181	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	10.3	20.6	0	1	1	1	0	0	0	0	0	1	0	0	0	0	0
15	B2181A	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	10.3	20.6	0	1	0	1	0	0	0	0	1	1	0	0	1	0	0
16	B2182	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	7.9	54	0	0	0	0	0	0	0	0	0	1	0	1	1	0	0
17	B2183	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	9.6	64.3	0	1	1	0	0	0	0	0	0	0	0	1	1	0	0
18	B2184	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	9.6	39	0	1	1	0	0	0	0	0	0	0	0	1	1	0	0
19	B2184A	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	9.6	39	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0
20	B2603	CITY OF CAPE TOWN	Reinforced concrete	115	arch	9.5	20.1	0	1	1	1	0	0	0	1	0	0	0	0	0	0	0
21	B2604	CITY OF CAPE TOWN	Reinforced concrete	115	arch	45.7	65.6	0	1	1	1	0	0	0	0	0	0	1	0	0	0	0
22	B2927	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	10.3	78.65	0	1	1	0	0	0	0	0	1	0	0	0	0	0	0
23	B2927A	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	10.3	78.65	0	1	1	0	0	0	1	0	0	0	0	0	1	0	0
24	B2960	CITY OF CAPE TOWN	Reinforced concrete	42	Simply supported	11	89.25	0	1	1	0	0	1	1	0	1	0	0	0	1	0	0
25	B2970	CITY OF CAPE TOWN	Reinforced concrete	42	Simply supported	14.4	14.4															
26	B2978	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	11.9	45	0	1	0	0	1	0	1	0	0	0	0	0	1	0	0
27	B3903	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	10.5	58.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
28	B4013A	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	10.7	63.2	0	1	1	0	0	0	0	0	0	1	0	1	1	0	0
29	B4113	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	18.5	74	0	1	1	0	0	0	0	0	0	1	0	0	1	0	0
30	B4179	CITY OF CAPE TOWN	Reinforced concrete	10	Simply supported	9.8	29.48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
31	B4246	CITY OF CAPE TOWN	Reinforced concrete	22	Simply supported	11.2	72.4	0	1	1	0	0	0	0	0	1	0	0	0	0	0	0
32	B4247	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	11.7	45.15	0	1	0	0	0	1	0	0	0	0	0	0	1	0	0
33	B4340	CITY OF CAPE TOWN	Reinforced concrete	115	Continuous	12.2	137.75	0	1	1	0	0	0	1	0	1	1	0	0	0	0	0
34	B4439	CITY OF CAPE TOWN	Reinforced concrete	115	Continuous	9	63	0	1	0	0	0	0	0	0	0	1	0	0	1	0	0
35	B4456	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	5.7	5.7	0	1	1	1	0	0	0	0	0	0	0	0	0	0	1
36	B4750	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	26.5	26.5	0	1	1	0	1	1	0	0	0	1	0	0	1	0	0
37	B4879	CITY OF CAPE TOWN	Reinforced concrete	36	Simply supported	12.2	53	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0
38	B4946	CITY OF CAPE TOWN	Reinforced concrete	115	Continuous	15.6	51.3	0	1	1	0	0	1	0	0	1	0	0	0	1	0	1
39	B4947	CITY OF CAPE TOWN	Reinforced concrete	115	Continuous	6.7	13.4	0	1	0	0	0	1	1	0	1	0	0	0	1	0	0
40	B4949	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	6.7	33.5	0	1	1	0	0	1	0	0	1	0	0	0	0	0	0
41	B4950	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	10.2	61.3	0	1	1	0	1	1	0	1	1	0	0	0	1	0	0
42	B4951	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	11	22	0	1	0	0	0	0	0	0	1	0	0	0	1	0	0
43	B4984	CITY OF CAPE TOWN	Reinforced concrete	115	Continuous	15.3	153	0	1	0	0	0	1	0	0	1	0	0	0	0	0	0
44	B4984A	CITY OF CAPE TOWN	Reinforced concrete	115	Continuous	15.3	153	0	1	0	0	0	1	0	0	1	0	0	0	1	0	0
45	B4985	CITY OF CAPE TOWN	Reinforced concrete	56	Simply supported	6.3	19.5															
46	B5104	CITY OF CAPE TOWN	Reinforced concrete	38	Simply supported	16	32	1	1	0	0	1	0	0	0	0	0	0	0	0	0	0
47	B5109	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	14	69.35	1	1	1	0	0	1	0	0	0	0	0	0	1	0	0
48	B5232	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	14.2	14.2	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0
49	B5233	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	13.8	27.5	0	1	0	0	0	0	0	0	1	0	0	0	0	0	0
50	B5234	CITY OF CAPE TOWN	Reinforced concrete	115	Simply supported	24	48	0	0	0	0	0	0	0	0	0	1	0	0	1	0	0
51	B5236	CITY OF CAPE TOWN	Reinforced concrete	36	Simply supported	8	64.1	0	1	0	0	0	0	0	0	0	0	0	0	1	0	0
52	B5237	CITY OF CAPE TOWN	Reinforced concrete	36	Simply supported	8	64.1	0	0	0	0	0	0	0	1	1	0	0	0	1	0	0

dges / Eden / Overburg / CPT / Central Karoo / Winelands / West Coast

Figure 3-3: RC Bridge Binary Matrix

3.3 Logistic Regression Analysis Theory

The logistic regression model also known as the logit model is a probabilistic statistical classification model that can be used when predicting a binary response of a dependant variable on the basis of an independent variable (Premph, 2009).

The response variable y_i is always binary and can therefore assume the values of either 0 or 1. Thus it has a binomial distribution which may be defined as:

$$y_i \sim B(n_i \pi_i), (i = 0 \ or \ 1)$$

Where n_i is known and represents the number of Bernoulli trials. Whereas π_i is unknown and indicates the possibility of the dependant variable being in one group (i.e. 1), while $(1 - \pi_i)$ indicates the possibility of it being in the other group (i.e. 0) (Premph, 2009).

π_i, is defined as follows:

$$\pi_i = \frac{1}{(1 + e^{-\theta i})} \qquad \text{... (4.1)}$$

and the variable θ_i is given by the linear regression expression:

$$\theta_i = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \cdots + \beta_k x_k \qquad \text{... (4.2)}$$

Whereby, the computed value of π_i is the probability in the range of 0 to 1, β_0 is the intercept and $\beta_1, \beta_2, \beta_3, \ldots, \beta_k$ are the unknown regression coefficients of the independent variables of interest denoted $x_1, x_2, x_3, \ldots, x_k$, correspondingly. The unknown regression coefficients indicate the size of the contribution of the independent variable. A positive coefficient therefore indicates an increase in the impact on the probability of the outcome and a negative coefficient indicates a decrease. The larger the coefficient the greater the independent variables influence on the probability of the outcome (Kleinbaum and Klein, 2010).

Using (4.1) and (4.2), π_i may alternatively be formulated to make the probability of the dependant variable equal a case equivalent to the exponential function of the linear regression expression as seen below:

$$f(\theta_i) = \pi_i = \frac{1}{(1 + e^{-\beta_0 + \beta_1 x_1 + \beta_2 x_2 + \cdots + \beta_k x_k})} \qquad \text{... (4.3)}$$

The linear predictors ($\beta_i x_i$) are able to take on any real value between negative infinity and infinity, whereas the output ($f(\theta_i) = \pi_i$) is confined between 0 and 1, hence it is interpretable as a probability. There is therefore no guarantee that the predicted values will be in the correct range as restrictions have not been imposed on the coefficients. Rodríguez (2007) indicates that the probability is therefore transformed over two steps to remove the range restrictions and model it as a linear function of the covariates. Firstly it is converted to the odds ratio (which provides a maximum likelihood estimation relative to the base category):

$$odds_i = \frac{\pi_i}{(1 - \pi_i)} \qquad \text{... (4.3)}$$

defined as the probability to its compliment, meaning the ratio of favourable to unfavourable cases. Thereafter logarithms of the odds ratio are taken to calculate the logit (i.e. log-odds or natural logarithm of the odds) which is equivalent to the linear regression expression:

$$\text{logit}\,(odds_i) = \log\frac{\pi_i}{(1 - \pi_i\,)} = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \cdots + \beta_k x_k \qquad \text{... (4.4)}$$

The transformation removes the restrictions and the resultant logit may be represented as seen in *Figure 3-4.* It may be noted that as the probability reduces down to zero, the odds approach zero and the logit approaches negative infinity. On the other hand, when the probability approaches one, the odds approach positive infinity and so does the logit.

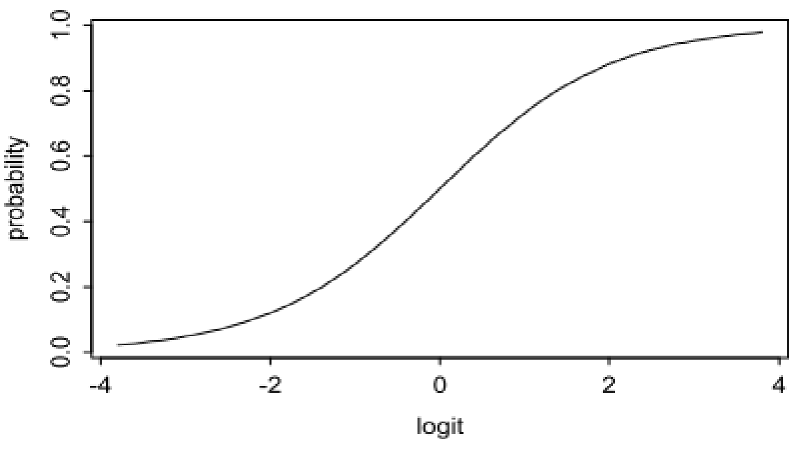

Figure 3-4: Logistic Function

Source: Rodríguez (2007)

Finally, considering *Equations 4.3* and *4.4* it may be noted that the logit serves as a link between the probability and linear regression expression.

3.4 Logistic Regression Analysis

According to Pyke and Sheridan (1993) and Premph (2009), LRA is particularly appropriate for modelling relationships between independent and dichotomous dependant variables; such as those in the study.

The selection of variables was determined primarily based on the research questions and objectives. The variables were grouped as follows:

Dependant variables – The dependant variables were the predominant defects. Those investigated for RC bridges were cracking; spalling; joint defects; surface erosion; movement/rotation as well as reinforcement corrosion. Similarly those investigated for RC culverts were cracking; spalling; defective concrete; scour; movement/rotations and reinforcement corrosion.

Continuous independent variables – The RC structure ages and span lengths/ widths constituted the independent variables as they were the predictor variables being manipulated to observe effects on the dependant variables. Furthermore, they were classified as continuous as they were numerical and could be ordered sequentially.

Categorical independent variables – The RC structure types and district municipalities were also part of the independent variables for the same reasons as the RC structure ages and span lengths/widths. However, they were classified as categorical as they did not take on a particular value/order and could be further categorised.

Thus the method of LRA used (refer to *Figure 3-5*) was employed to:

1. Determine statistically significant relationships between dependant and independent variables.

2. Quantify (through means of an odds ratio) the maximum likelihood of a dependant variable to move from one group to the other in comparison to the base category, i.e. for a defect to go from “absent” to “present” on a RC structure relative to the reference RC structure.

3. Determine the variance in the dependant variables explained by the independent variables. For example use the odds ratio to discuss the effect the various locations had on the presence of a specific defect on a RC structure.

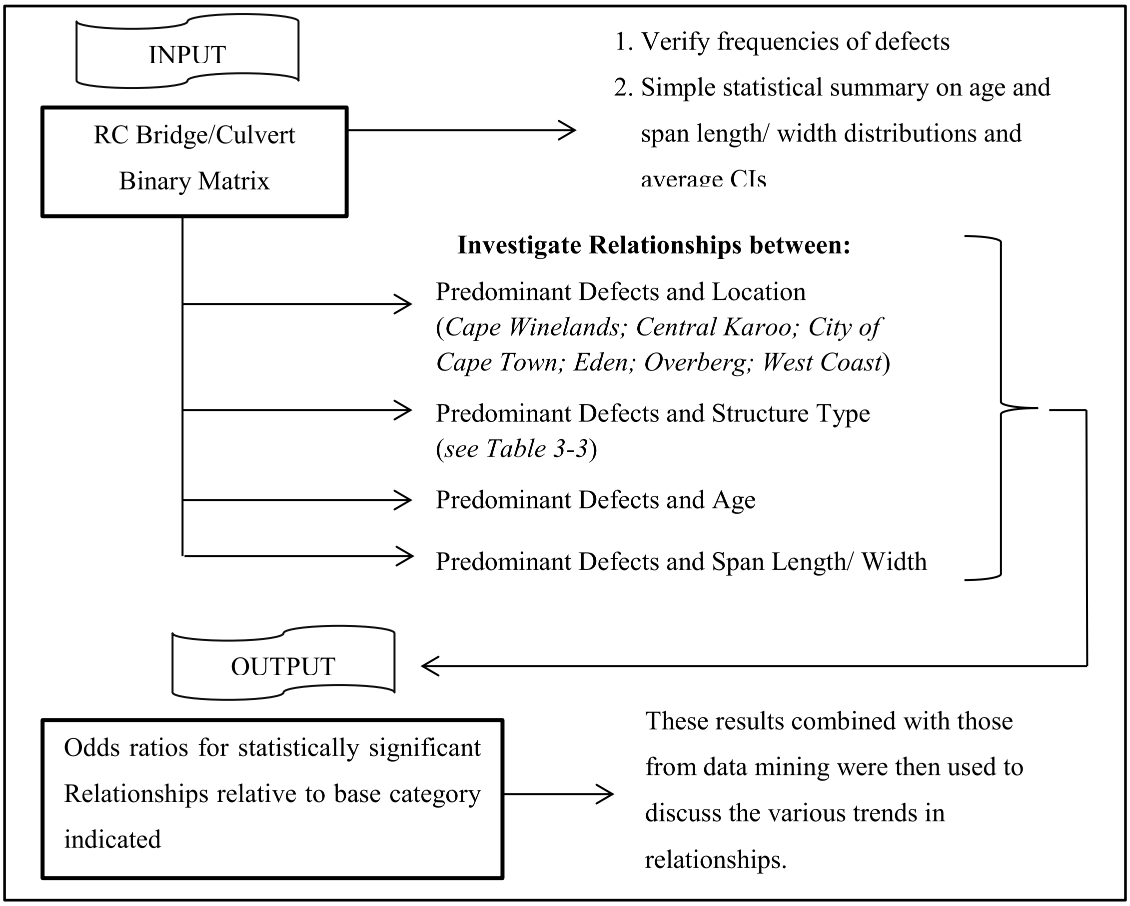

Figure 3-5: Analysis Process using STATA 2011

The codes used to generate the STATA 2011 logit models so as to investigate the relationships of interest were the following:

1. Relationship between predominant defects (dependant variable) and location (categorical variable):

```
encode District, gen(distr)

xi:logistic "insert defect" i.distr

E.g. xi:logistic cracking i.distr
```

2. Relationship between predominant defects (dependant variable) and structure type (categorical variable):

```
encode StructureType, gen(type)

xi:logistic "insert defect" i.type

E.g. xi:logistic cracking i.type
```

3. Relationship between predominant defects (dependant variable) and structure type and location (categorical variables):

```
tab type

xi:logistic “insert defect” i.distr if “insert type”

E.g. xi:logistic cracking i.distr if type==4

(Where type 4 is a simply supported RC bridge)
```

4. Relationship between defects (dependant variable) and age (continuous variable and categorical variable):

```
    • Age (Continuous Variable)

Logistic “insert defect” Age

E.g. logistic cracking Age

    • Age (Categorical Variable)

egen agegroup=cut(Age), at(0,10,20,30,40,50,60,70,80,90,100)

xi:logistic “insert defect” i:agegroup

E.g. xi:logistic cracking i:agegroup
```

5. Relationship between RC bridge Span Length and defects (dependant variable):

```
    • Span Length (Continuous Variable)

Logistic “insert defect” SpanLength

E.g. logistic cracking SpanLength

    • Span Length (Categorical Variable)

egen spangroup=cut(SpanLength), at(0,5,10,15,20,25,30,35,40,100)

xi:logistic “insert defect” i.spangroup

E.g. xi:logistic cracking i.spangroup
```

The present study was particularly concerned with the odds ratio, defined in the previous section. The logistic regression analyses were conducted at the 0.05 level of statistical significance characterized by p-values less than 0.05, 95 % confidence intervals of odds ratios not containing 1 and odds ratios differing from 1.

An example of the interpretation of LRA results from STATA 2011 has been provided in *Figures 3-6* and *3-7*. It may be noted that the results in these two figures are the same; however, they are presented in different ways. The first figure reports coefficients with respect to the logit (log of odds) while the second reports the odds ratios.

Output variable (Y)

Predictor variable (X) i.e. district

_Idistr_1 – Cape Winelands (the Base Category)

Code

```
. xi:logit cracking i.distr
i.distr           _Idistr_1-6         (naturally coded; _Idistr_1 omitted)

Iteration 0:   log likelihood = -512.96203
Iteration 1:   log likelihood = -490.35517
Iteration 2:   log likelihood = -490.05996
Iteration 3:   log likelihood = -490.05948
Iteration 4:   log likelihood = -490.05948

Logistic regression    (a)                    Number of obs   =        770   (b)
                                              LR chi2(5)      =      45.81   (c)
                                              Prob > chi2     =     0.0000   (d)
Log likelihood = -490.05948                   Pseudo R2       =     0.0446   (e)

------------------------------------------------------------------------------
    cracking |      Coef.   Std. Err.      z    P>|z|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
   _Idistr_2 |   1.589496   .3138435     5.06   0.000     .9743741    2.204618
   _Idistr_3 |   1.578778   .3312738     4.77   0.000     .9294932    2.228063
   _Idistr_4 |   .6716601   .2205063     3.05   0.002     .2394757    1.103845
   _Idistr_5 |   .7327563   .2539196     2.89   0.004     .2350831     1.23043
   _Idistr_6 |   .5284559   .2134599     2.48   0.013     .1100821    .9468296
       _cons |  -.1573922   .1451633    -1.08   0.278    -.4419069    .1271226
------------------------------------------------------------------------------
```

The iterative process in estimating the maximum likelihood (listing of the log likelihoods at each iteration). The model converged at 490.05948.

Dependant variable

Independent variables

_Idistr_2 – Central Karoo
_Idistr_3 – CPT
_Idistr_4 – Eden
_Idistr_5 – Overberg
_Idistr_6 – West Coast

Recall *Equation 4.4*

logit (cracking)

= -0.157 + 1.59(Central Karoo) + 1.58(City of Cape Town) + 0.67(Eden) + 0.73(Overberg) + 0.53(West Coast)

It may be noted that the intercept (β_0) is not significant however we are more interested in the predictor variables and their coefficients ($\beta_i X_i$).

Two tailed p-values test the null hypothesis which is $H_0 : x_i = 0$.

To reject this a p-value less than 0,05 (for the 95 % confidence interval) is required.

The z-value test the null hypothesis which is $H_0 : \beta_i = 0$.

To reject this, a z-value greater than 1.95 (for the 95 % confidence interval) is required.

Figure 3-6: Reporting Logistic Regression using the Logit Command

The other parameters listed in *Figure 3-6* include:

(a) The log likelihood of the fitted model used in the likelihood ratio Chi-Square test.

(b) The number of observations used by the model; 770 RC bridges. These are dependent on the binary matrix.

(c) The LR chi2(5) test that at least one of the predictors' coefficients is not equal to zero. The number in the parentheses indicates the degrees of freedom of the Chi-Square distribution used to test the LR Chi Square statistic. It is defined by the number of models (1) multiplied by the number of predictors in the model (5).

(d) Prob> chi2 is the p-value of the model and it should be lower than 0.05 to show a statistically significant relationship between X (independent/predictor variable) and Y (output/dependent variable). It indicates the reliability of X to predict Y under the null hypothesis; all of the regression coefficients are simultaneously equal to zero.

(e) Pseudo R2 indicates the amount of variance of Y explained by X. In the example provided the model explains 4.46 % of the variance in cracking.

In the case of the study the main focus was the odds ratio thus the other parameters, though considered, were not been discussed in the analysis.

Code

```
. xi:logistic cracking i.distr
i.distr           _Idistr_1-6         (naturally coded; _Idistr_1 omitted)

Logistic regression                               Number of obs   =        770
                                                  LR chi2(5)      =      45.81
                                                  Prob > chi2     =     0.0000
Log likelihood = -490.05948                       Pseudo R2       =     0.0446

------------------------------------------------------------------------------
    cracking | Odds Ratio   Std. Err.      z    P>|z|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
   _Idistr_2 |   4.901278   1.538234     5.06   0.000     2.649508    9.066788
   _Idistr_3 |   4.849026   1.606355     4.77   0.000     2.533225    9.281866
   _Idistr_4 |   1.957484   .4316376     3.05   0.002     1.270583    3.015738
   _Idistr_5 |   2.080808   .5283579     2.89   0.004     1.265014    3.422699
   _Idistr_6 |   1.696311   .3620944     2.48   0.013      1.11637    2.577525
       _cons |   .8543689    .124023    -1.08   0.278     .6428094    1.135556
------------------------------------------------------------------------------
```

Odds Ratio

Figure 3-7: Reporting Logistic Regression using the Logistic Command

It may be noted that the base category (i.e. the reference DM, structure type, age group or span length/width) is selected by the programme, STATA 2011, depending on the ordering of the data inputted into the programme. In other words, STATA 2011 will automatically selected the first district or structure type etc. in the table imported for analysis as the base category. Additionally, because the odds ratios could be converted into fractions if need be, it was decided that there was no need to necessarily pre-select a base category.

Lastly, the STATA 2011 logit model used for the analysis was represented as:

$$\mathrm{Prob}(\mathrm{y} = 1|x) = A(x\beta) = \frac{\exp(x\beta)}{1 + \exp(x\beta)} \qquad \dots (4.5)$$

Where A represents the cumulative standard logistic distribution link function (Park, 2009).

O'Halloran (2008) states that when the dependant variable is dichotomous, the STATA 2011 LRA model makes the following assumptions:

1. The independent variables need not be interval or unbound and they are not linear combinations of each other.

2. The errors are not normally distributed and their variance is not constant.

3. The variance of the dependant variables need not be homoscedastic for each level of the independent variables, i.e. variance need not be the same within categories.

4. Linear relationships between dependant and independent variables are not assumed by the logit function.

In summary, the data mining activities outlined in the research methodology were conducted to verify the data and cluster it into relevant and appropriate categories so as to ease the analysis procedure. The binary matrices were useful analysis tools, not only for data mining but also for the LRA. Although relationships were identified during data mining, LRA was used to determine whether they were statistically significant or not as well and provide a more credible basis for interpreting the findings.

Chapter 4

4 Data Analysis and Discussion

4.1 Introduction

This chapter consists of five main sections, excluding the introduction and the summary. The first section presents a summary of the inventory and inspection data for all the bridges and culverts within the Western Cape Province. The second section discusses the most predominant defects on RC structures including, based on informed speculation, their causes and/or deterioration mechanisms. The other three sections present the relationships identified between the predominant defects and the relevant inventory data (location, structure type and span length/width) from the LRA and data mining activities.

4.2 Summary of Inventory and Inspection Data

4.2.1 Distribution of Structures

There were 2 419 structures recorded on the BMS database distributed as in *Figure 4-1*. These consisted of 857 (35 %) bridges and 1 562 (65 %) culverts.

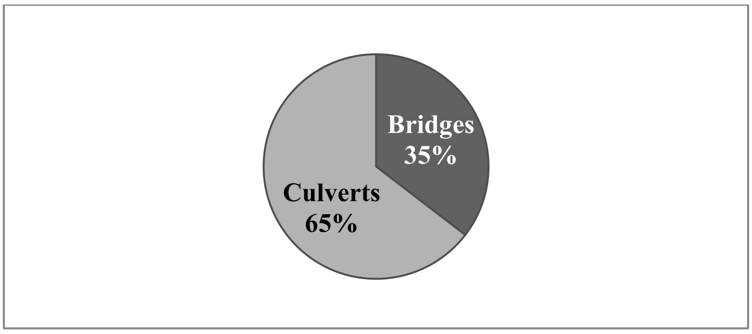

Figure 4-1: Distribution of Bridges and Culverts in the Western Cape Province

The Eden District Municipality (DM) contained the largest number of structures representing 25 % of all the structures. The metropolitan district municipality, the City of Cape Town, had the least quantity of structures (80 bridges and 23 culverts) constituting only 4.3 % of the total. Approximately 61.7 % of the structures were located in the coastal DMs (West Coast; City of Cape Town; Eden and Overberg) and the remaining inland district municipalities (the Cape Winelands DM and the Central Karoo DM). A map showing the district municipalities within the province is provided by *Figure 4-2*.

It may be noted that a small portion of the bridges and culverts (less than 1 % of total structures) locations were not indicated in the database; these were quantified under 'unknown' DMs. *Table 4-1* and *Figure 4-3* illustrate the distribution of the bridges and culverts across the districts.

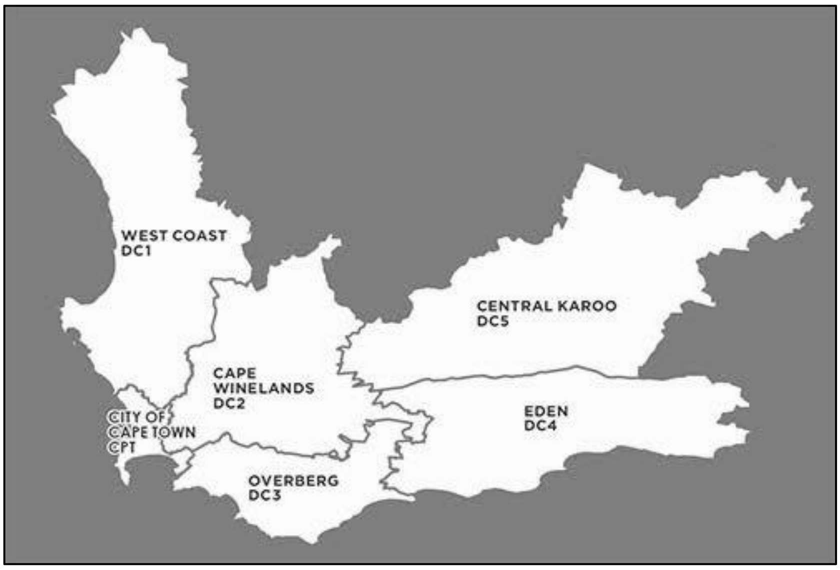

Figure 4-2: Map Showing Western Cape District Municipalities

Adapted from: http://www.localgovernment.co.za/provinces/view/9/western-cape

[Accessed 2014, July 29]

Table 4-1: Distribution of Bridges and Culverts per District Municipality

District Municipality	Bridges	Culverts	Total Structures (%)
Cape Winelands	203	330	22.0
Central Karoo	90	305	16.3
City of Cape Town	80	23	4.3
Eden	179	425	25.0
Overberg	112	212	13.4
West Coast	177	263	18.2
Unknown	16	4	0.8
Total	**857**	**1562**	**100**

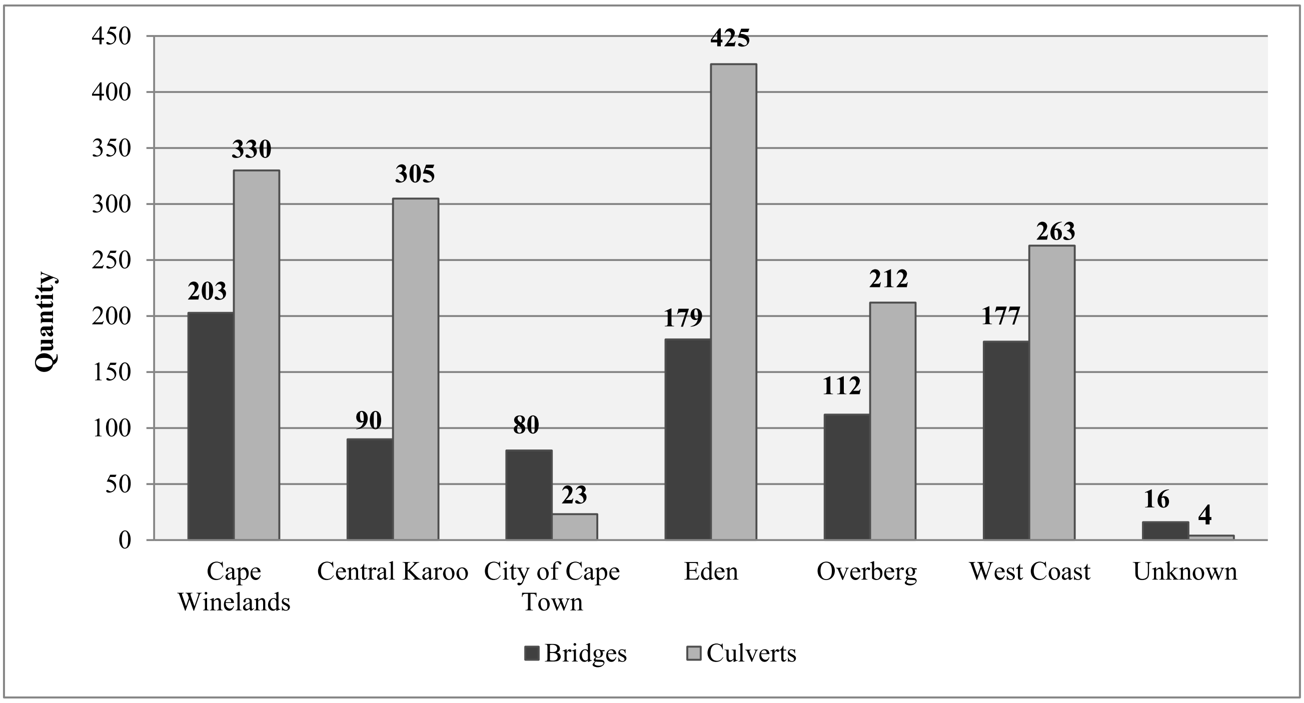

Figure 4-3: Distribution of Bridges and Culverts per District Municipality

4.2.2 Material Classifications

The quantities of structures per material type are summarized in *Table 4-2* and illustrated in *Figure 4-4 and 4-5*. There were approximately 791 (92 %) RC bridges and 1376 (88 %) RC culverts. Together the RC structures represented 90 % of all structures. Only about 3 % of the structures could not be classified due to missing inventory data.

Table 4-2: Distribution of Structures per Material Type

Material Type	Bridges	Culverts	Total Structures
Composite	7	18	25
Masonry	2	1	3
RC	**791**	**1376**	**2163**
Steel and Concrete	29	112	143
Unclassified	28	55	84
Total	857	1562	2419

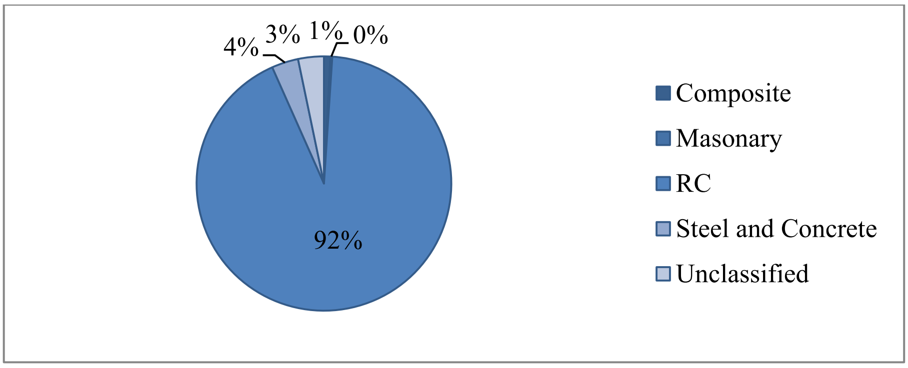

Figure 4-4: Percentage Distribution of Bridge Material Types

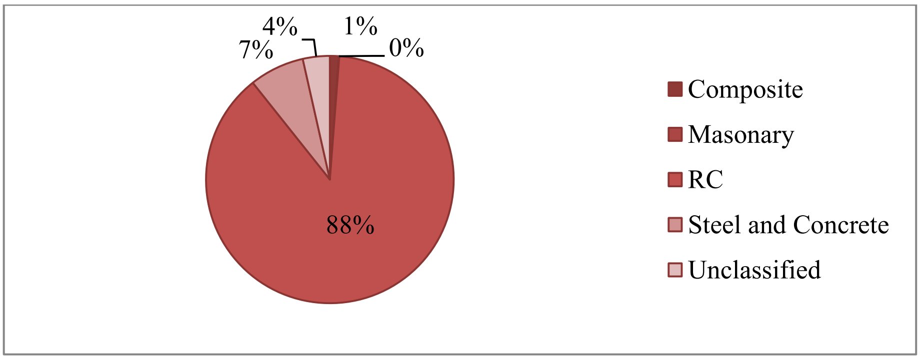

Figure 4-5: Percentage Distribution of Culvert Material Types

4.2.3 Structure Types

The structures were classified according to their respective structure types as shown in *Table 4-3,* below.

Table 4-3: Distribution of Bridges and Culverts per Structure Type

Structure Type	Bridges	Culverts	Total Structures (%)
Arch	29	46	3.1
Armco Pipe	0	50	2.0
Cantilever	7	0	0.3
Cellular	11	21	1.3
Concrete Pipe	0	57	2.4
Continuous	95	60	6.4
Precast Portal Frame	27	253	11.6
Simply Supported (including in-situ portal frame for culverts)	651	1019	69.0
Truss Girder	9	0	0.4
Unclassified	28	56	3.5
Total	**857**	**1562**	**100**

The majority of the structures were simply supported; of the bridges 76 % were simply supported bridges and of the culverts 65 % were simply supported in-situ portal frame culverts. *Figures 4-6* and *4-7* show the percentage of simply supported structures and the percentages of the other structure types. From the illustrations on the right hand side of these two figures it may be seen that continuous bridges constitute 46 % of the other bridge structure types and precast portal frame culverts constitute 47 % of the other culvert structure types.

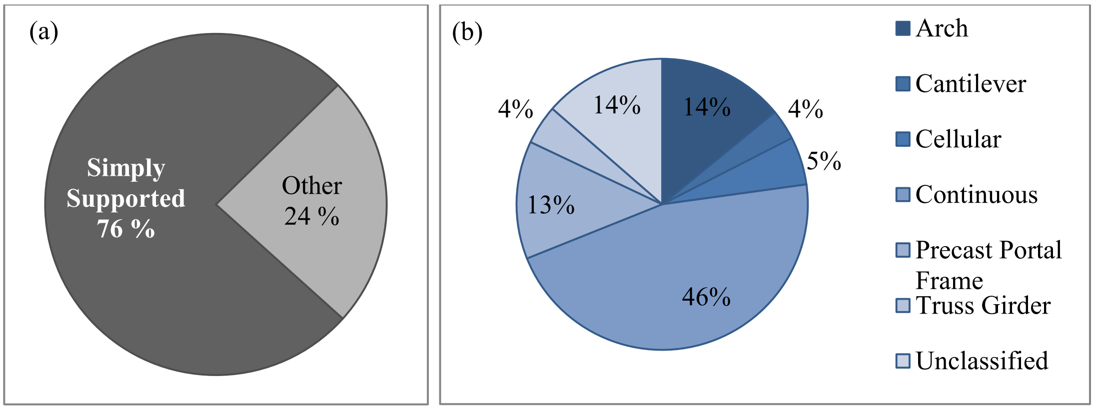

Figure 4-6: Percentage Distribution of Bridges by Structure Type: (a) Simply Supported Bridges vs Other Bridge Types; (b) Distribution of Other Bridge Types

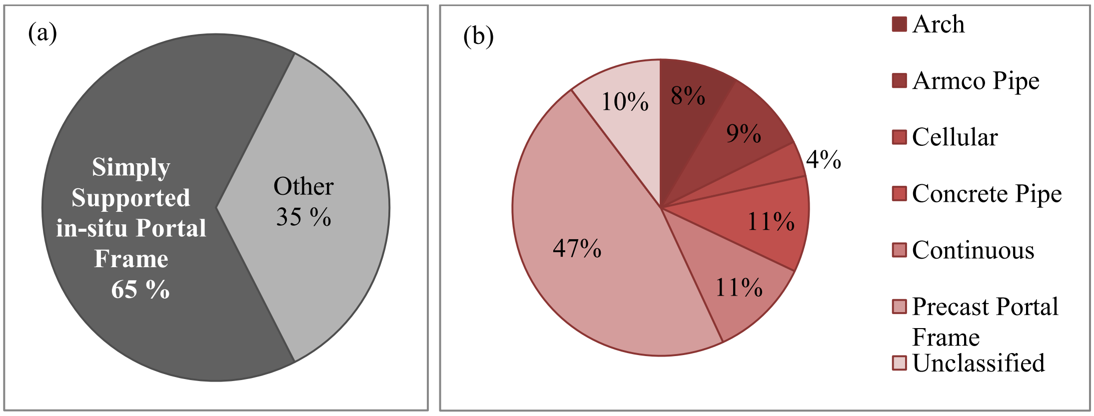

Figure 4-7: Percentage Distribution of Culverts by Structure Type: (a) Simply Supported in-situ Portal Frame Culverts vs Other Bridge Types; (b) Distribution of Other Bridge Types

4.2.4 Span and Overall Lengths

Figures 4-8 and *4-9* represent 786 bridges (92 %) and 1293 culverts (83 %) that had span lengths provided in the database. Over half of the bridges (52 %) had span lengths between 5 m and 10 m and over half of the culverts (53 %) had span lengths between 3 m and 4 m.

Figures 4-10 and *4-11* represent the distribution of bridge and culvert overall lengths, respectively. The bridge category of 50 m and over contained the largest quantity of bridges, which constituted 25 % of the total sum of the bridges. About 75 % of the culverts were found to be less than 10 m in length. The average length of the bridges was calculated to be 43.6 m and the average length of the culverts was calculated to be 9.5 m. It may be noted that approximately 2 % of the total bridge's maintenance regions and lengths as well as 1 % of the total culverts maintenance regions and lengths could not be obtained.

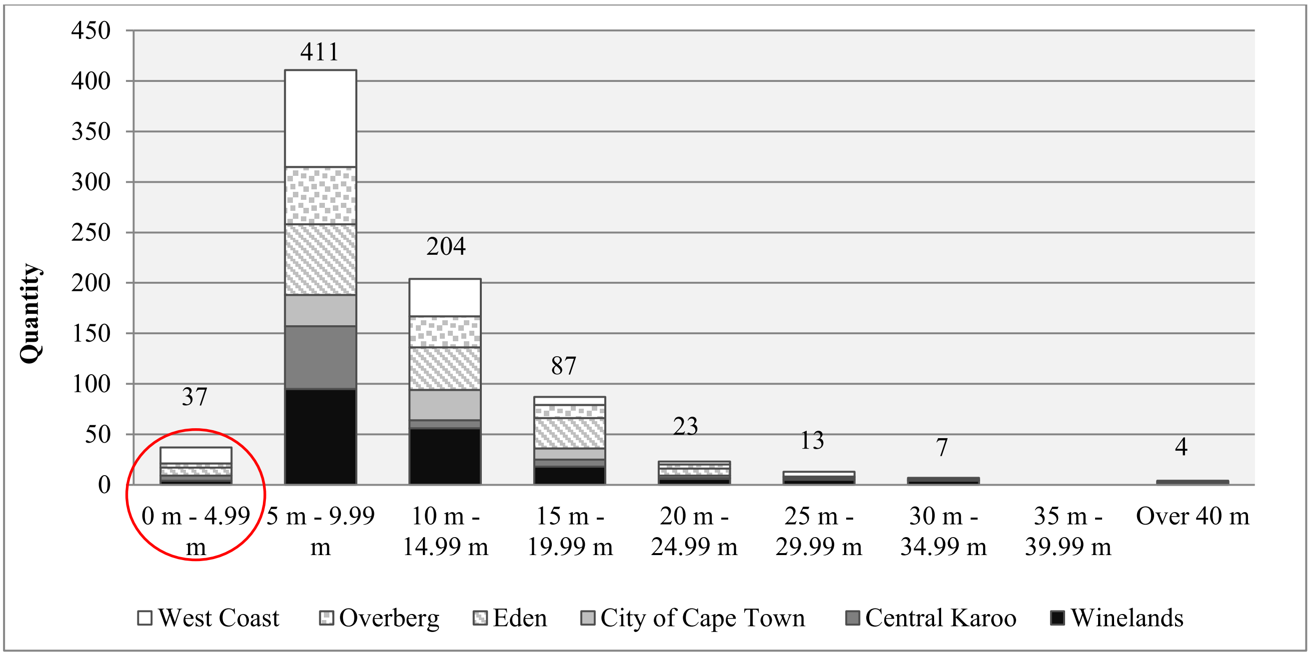

Figure 4-8: Distribution of Bridges by Span Length per District Municipality

It may be noted that 37 of the bridges had span lengths between 0 m and 5 m. However, these bridges satisfied the bridge criterion discussed in *section 2.2.1* as they had at least one span greater than 1.5 m and they were longer than 20 m or were road-over-rail/ rail-over-road bridges.

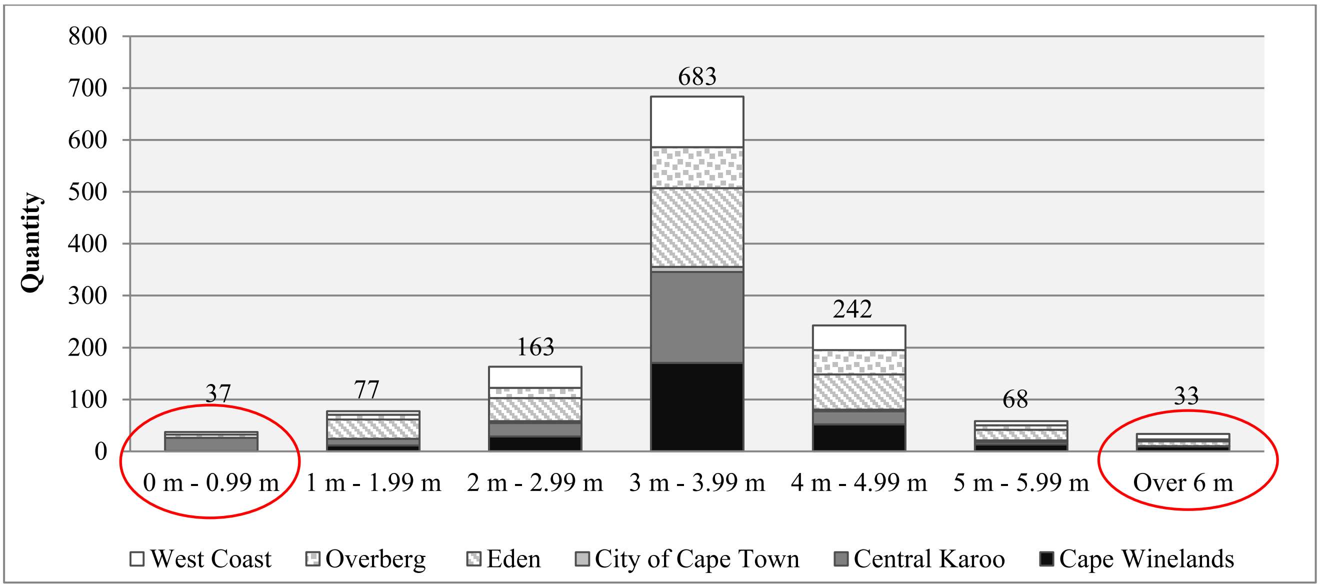

Figure 4-9: Distribution of Culverts by Span Length per District Municipality

There were 37 culverts with widths less than 1 m. After inspecting these using the inventory photos, they were assumed incorrectly recorded in the database. Also, 33 culverts had widths just over 6 m. The greatest culvert width was 6.25 m.

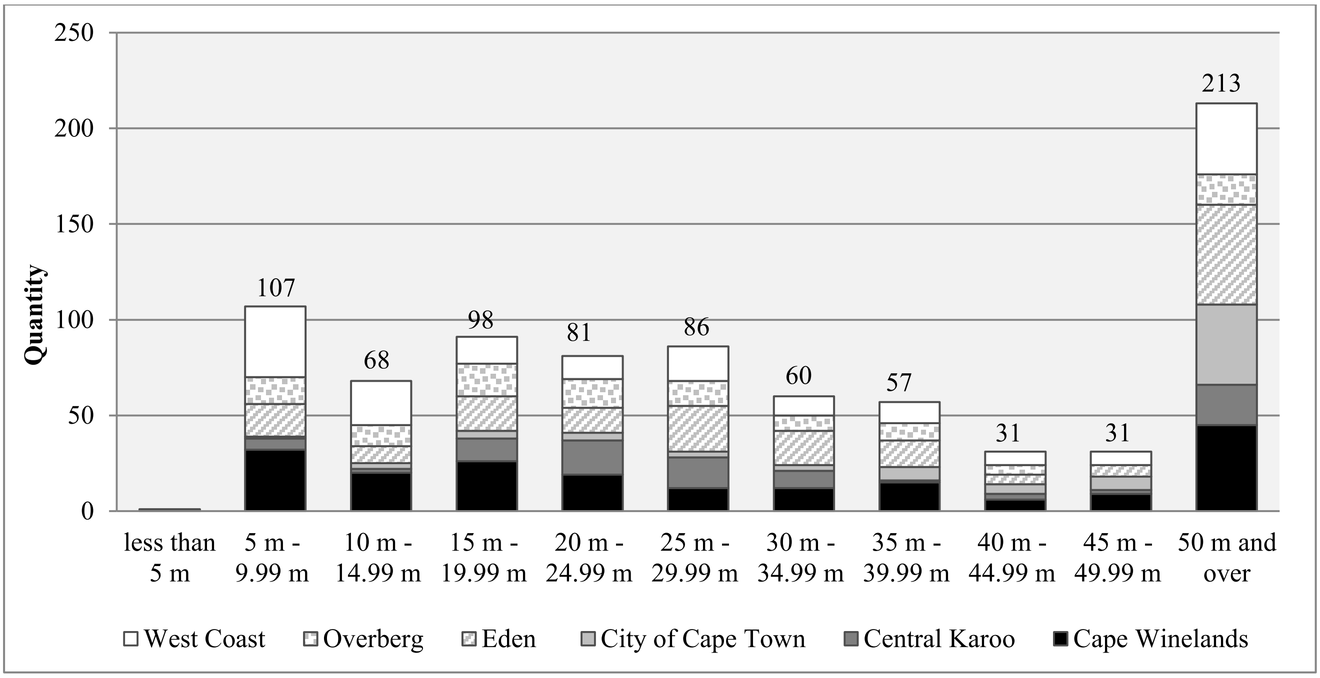

Figure 4-10: Distribution of Bridges by Overall Length per District Municipality

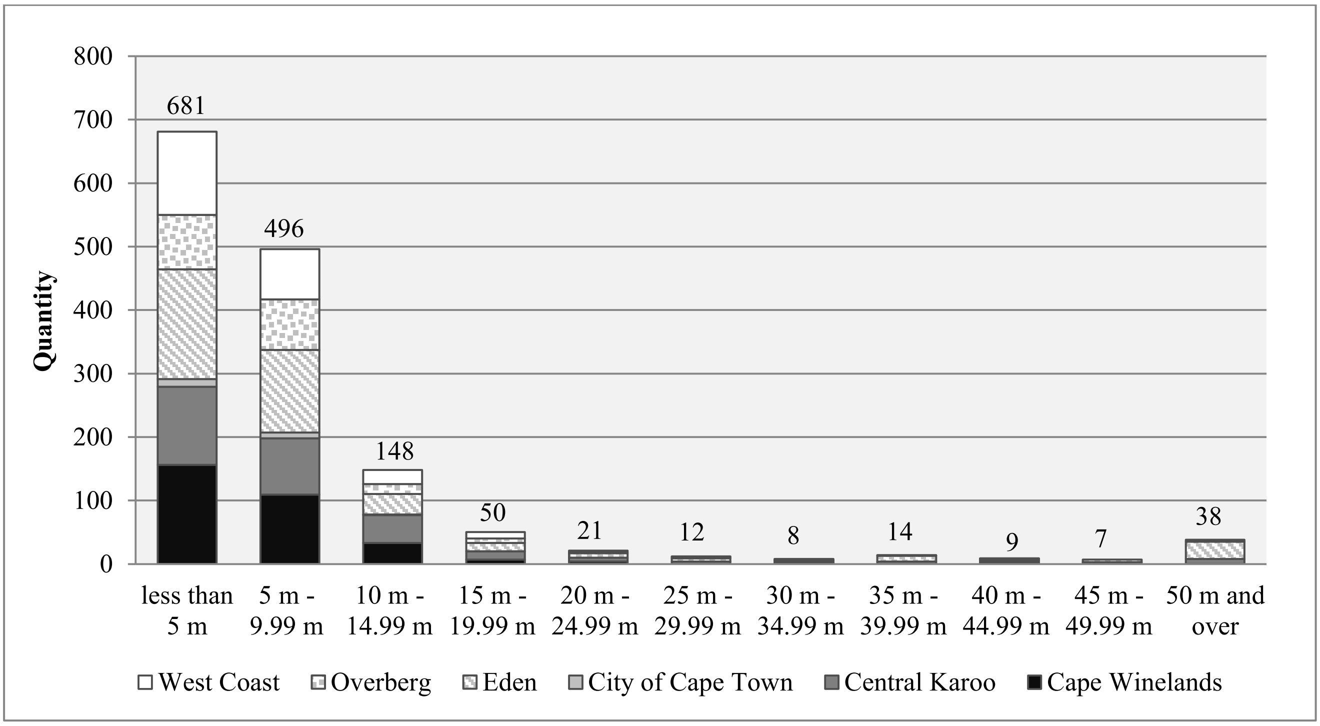

Figure 4-11: Distribution of Culverts by Overall Length per District Municipality

4.2.5 Age and Average Condition

It may be noted that 37 % (321) of the bridge's construction dates were not available on the database; hence their ages could not be determined. Those with known construction dates are tabulated according to their locations in *Table 4-4* and represented in *Figure 4-12*.

Table 4-4: Distribution of Bridges by Age per District Municipality

Age Category	District Municipality						
	Cape Winelands	Central Karoo	CPT	Eden	Overberg	West Coast	Western Cape
<10 years	0	0	1	5	0	0	**6**
10 – 19 years	6	2	6	6	0	1	**31**
20 – 29 years	1	0	7	10	11	5	**44**
30 – 39 years	3	3	6	21	17	22	**92**
40 – 49 years	30	26	2	23	5	17	**103**
50 – 59 years	25	20	4	36	21	34	**140**
60 – 69 years	30	3	0	9	12	17	**71**
70 – 79 years	6	6	0	5	8	10	**35**
80 – 89 years	5	0	0	0	4	3	**12**
90 – 99 years	1	0	0	0	1	0	**2**
Unclassified Ages	56	30	54	64	33	68	**321**
Unknown Districts							**16**
TOTAL	203	90	80	179	112	177	**857**

Approximately 78.1 % of the bridge structures with known construction dates were constructed between 1945 and 1984 and therefore range from ages 30 to 69 years, of which 59.9 % of these were between 40 and 60 years old. The average age of the bridges, computed from available data, was estimated to be 47 years.

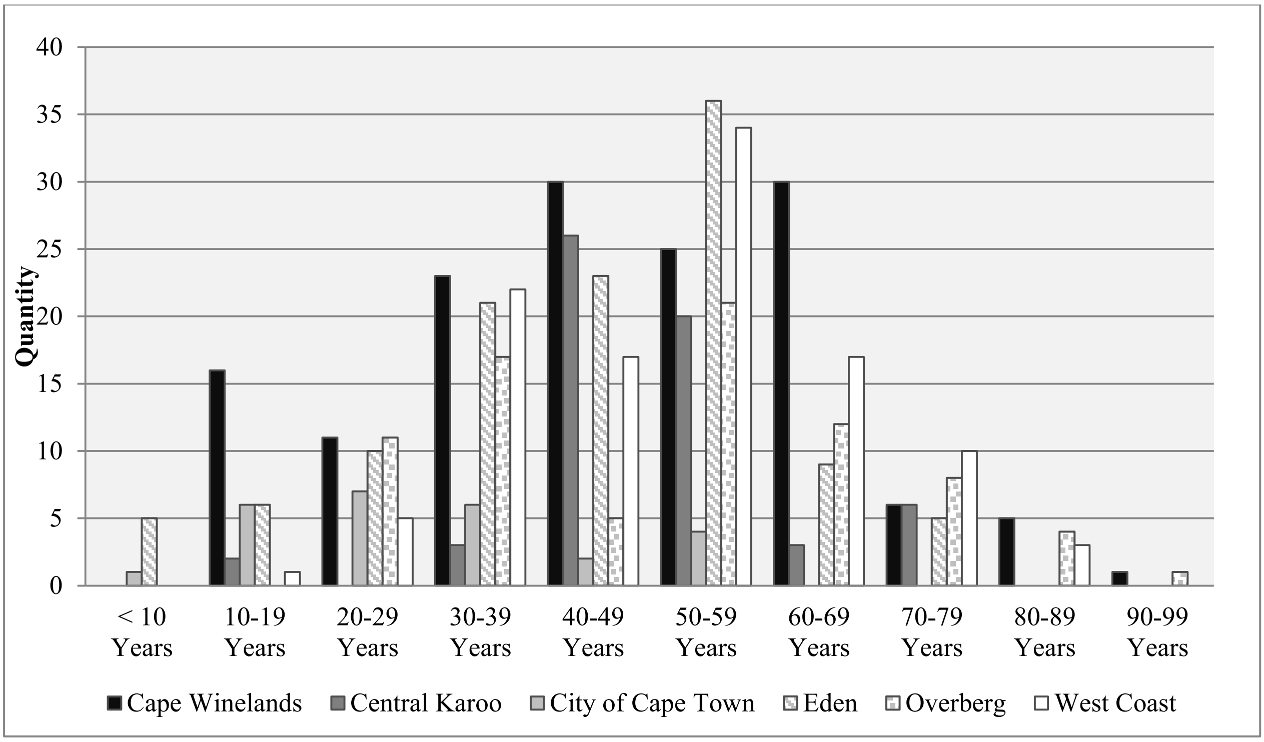

Figure 4-12: Distribution of Bridges by Age per District

The average condition of the bridges per age category was investigated and a graphic representation provided by *Figure 4-13*. The average condition indices were all greater than 70 thus, from *Table 2-11* in *Chapter 2*, it may be concluded that the bridges were generally in good condition.

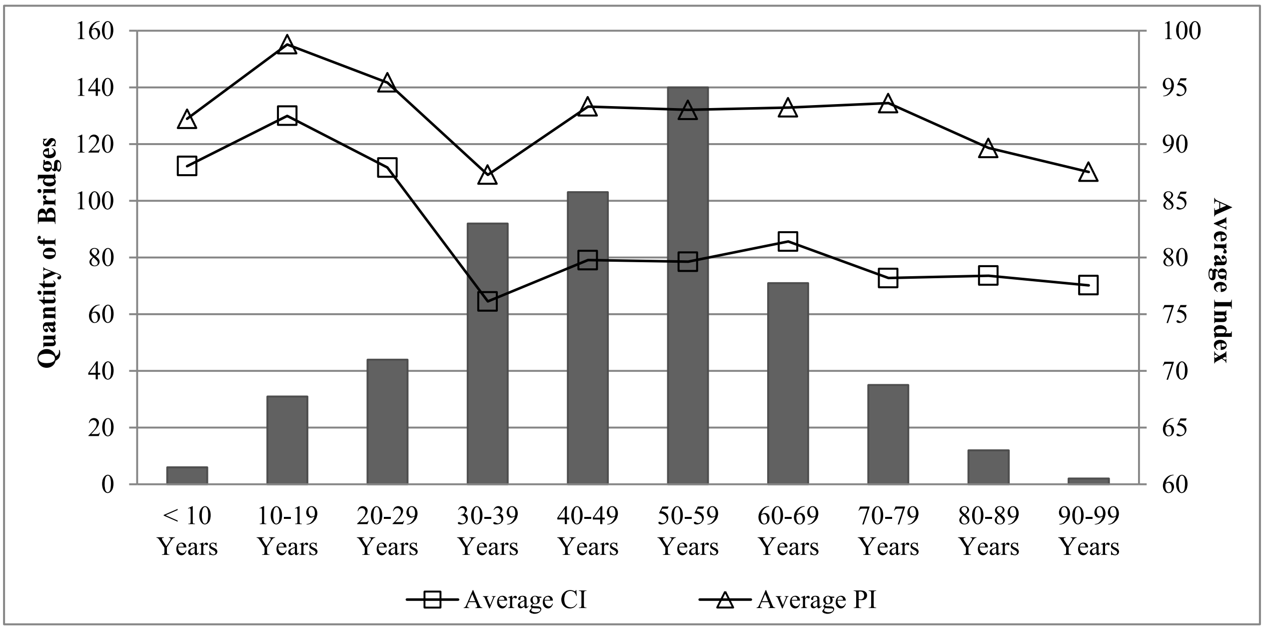

Figure 4-13: Average CI and PI for Bridges per Age Group

It can be seen that the average PI graph follows that of the average CI graph and that there is a general decrease with increasing age group categories. The average CI was calculated to be 81.9 and the average PI calculated to be 92.6. *Figure 4-14,* derived from *Figure 4-13,* highlights observations made with respect to the average CIs and age group categories.

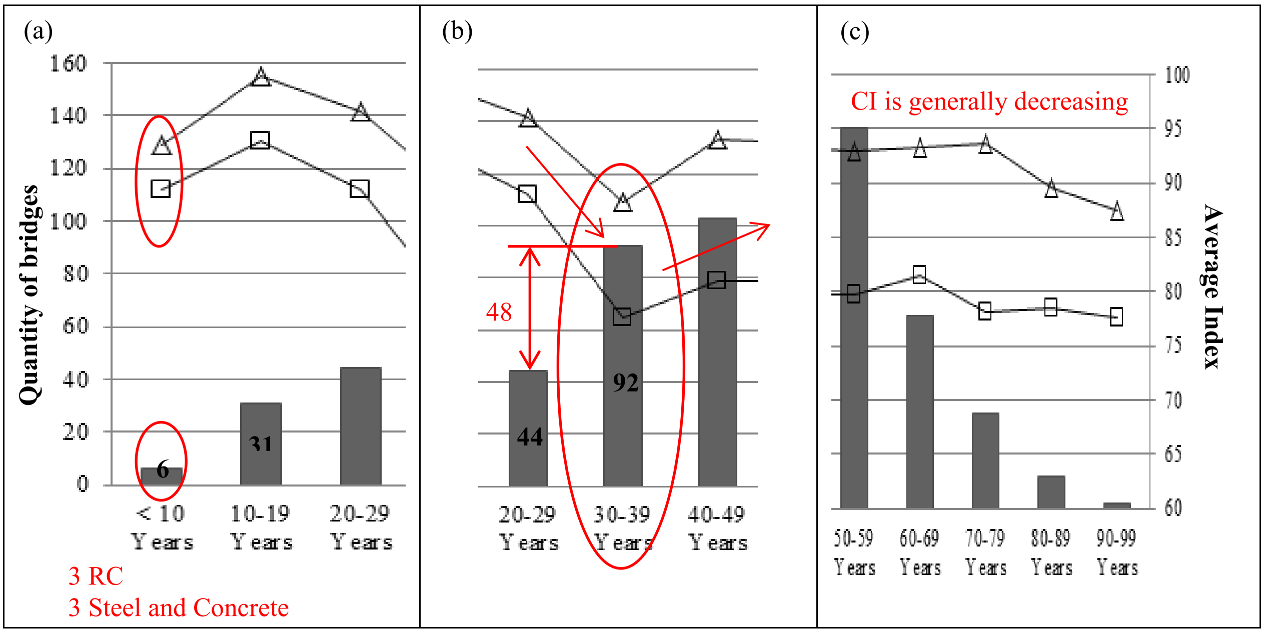

Figure 4-14: (a) – (c) Analysis of CI Relative to Age Categories

The CI is usually expected to decrease with increasing bridge age as deterioration is expected to increase. However, bridges less than 10 years of age had an average CI of 88.1 whereas bridges from 10 to 19 years had an average CI of 92.5 (see *Figure 4-14(a)*). The bridges less than 10 years constituted of three RC bridges and three steel and concrete bridges. Five of these were located in the Eden DM and the remainder in the City of Cape Town. Furthermore, it was noted that the CIs associated with the steel and concrete bridges were lower than those associated with RC bridges. On the other hand, bridges between 10 and 20 years were mostly RC bridges. Approximately 52 % of these were from the Cape Winelands DM and the remaining distributed mainly between the Eden DM and the City of Cape Town.

The lower CI observed for bridges less than 10 years was assumed to be associated with the increase in steel and concrete bridges in this category as these showed more sever deterioration. Also, these bridges were closer a the marine environment as they were located in the Eden DM, increasing their exposure to chloride ions and making them more susceptible to the mechanism of chloride ingress, which requires less time to cause deterioration than carbonation. The bridges in the category of 10 to 19 years were generally located inland (i.e. the Cape Winelands DM) and as a result were less prone to chloride ingress.

The age category ranging from 30 to 39 years, shown in *Figure 4-14(b)*, had the lowest average CI (76.1) and the largest increase (109 %) in quantity of bridges constructed (48 bridges) when comparing consecutive years. These bridges were mostly simply supported with the exception of 7 continuous bridges and 2 cantilever bridges. The bridges were distributed as follows; 29 in the Cape Winelands DM; 21 in the Eden DM; 16 in the Overberg DM; 15 in the West Coast DM; 8 in the City of Cape Town and 3 in the Central Karoo DM. The low average CI was therefore attributed to several causes which included the following:

1. The large decrease (52 %) in the construction of bridges during the period from 1975 to 1994 may have resulted in higher quality bridges. There were less bridges constructed between 1985 and 1994 compared to the period between 1975 and 1984, thus less care could have been taken during the construction of the bridges of age category 30 to 39 years.

2. The construction techniques employed during this period.

3. Approximately 65 % of the structures were located along the coastal district municipalities and could have therefore suffered the consequences of marine exposure.

4. Enough time had passed to allow for the older structures to have experienced carbonation.

The average CIs for bridges of age category ranging from 50 to 99 years shows a slight decrease. This was to be expected and though the average CI of 81.4 for age category 60 to 69 years is higher than the average CI of 79.8 for the age category of 50 to 59 years, the difference is CIs is considered reasonably minor.

Approximately 81 % (1 266) of the culvert's construction dates were not available in the database; hence their ages could not be determined. Those with known construction dates are tabulated, according to their location, in *Table 4-5* and represented in *Figure 4-15*.

Table 4-5: Distribution of Culverts by Age per District Municipality

Age Category	**District Municipality**						
	Cape Winelands	**Central Karoo**	**CPT**	**Eden**	**Overberg**	**West Coast**	**Western Cape**
<10 years	1	0	4	22	5	3	**35**
10 – 19 years	10	1	3	9	11	6	**40**
20 – 29 years	5	1	0	9	9	3	**27**
30 – 39 years	9	3	1	2	1	2	**18**
40 – 49 years	21	30	0	3	5	9	**68**
50 – 59 years	11	23	0	6	7	8	**55**
60 – 69 years	11	4	1	4	4	5	**29**
70 – 79 years	4	2	0	0	0	7	**14**
80 – 89 years	2	0	0	3	3	3	**10**
90 – 99 years	0	0	0	0	0	0	**0**
Unclassified Ages	256	241	14	167	167	217	**1266**
Unknown Districts							**4**
Total	**330**	**305**	**23**	**425**	**212**	**263**	**1562**

Approximately 52.1 % of the total culverts with known construction dates were constructed between 1945 and 1974 and therefore range in age from 40 to 69 years. The average age of the culverts, computed from available data, was estimated to be 39.5 years.

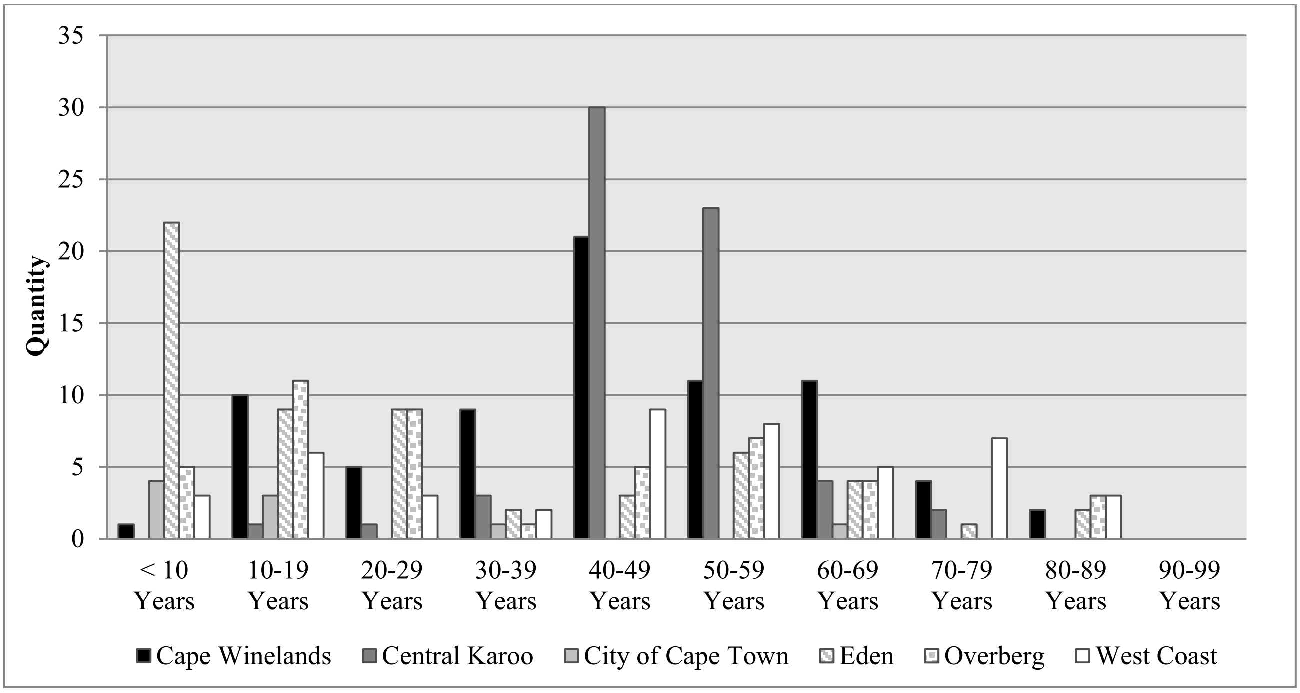

Figure 4-15: Distribution of Culverts by Age per District Municipality

The average condition of the culverts per age category was investigated and a graphic representation provided by *Figure 4-16*. The average condition indices were above 70 and using the same reasoning as that used with the bridges, the culverts were also concluded to generally be in good condition.

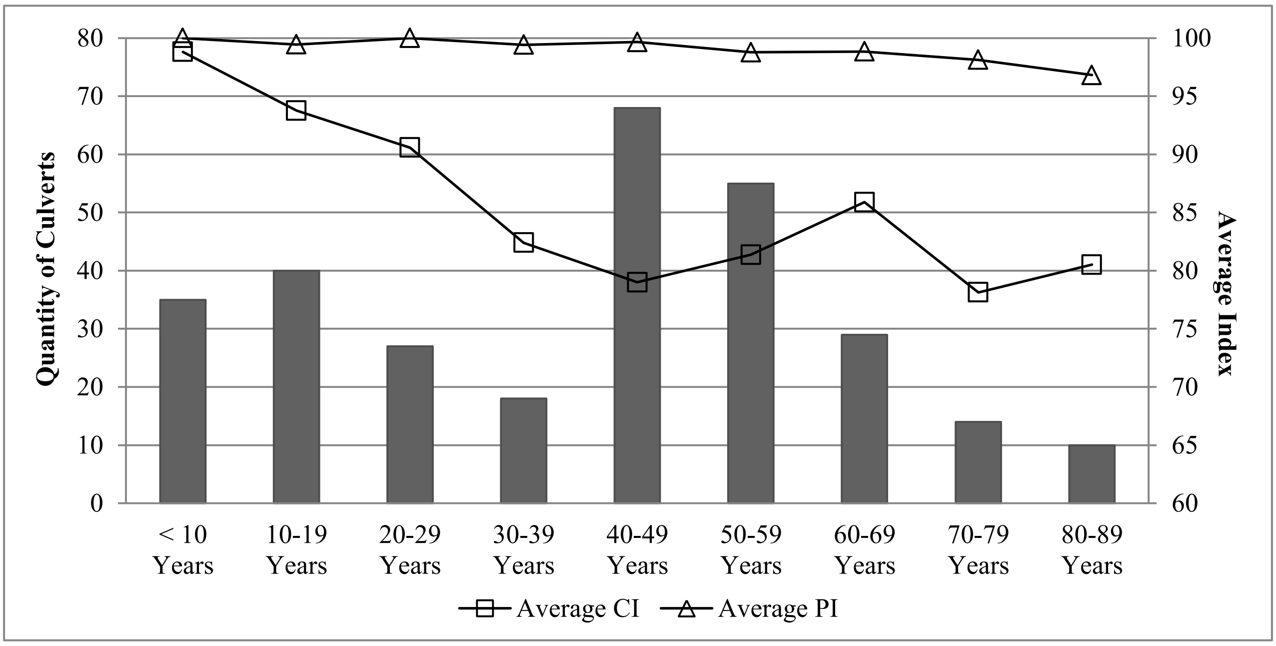

Figure 4-16: Average CI and PI for Culverts per Age Group

Contrary to bridges, the shapes of the culvert PI and CI graphs do not follow each other. Also, the general decrease in average CI with age is more pronounced than that of the decrease in average PI with age. The average CI was calculated to be 84 % and the average PI was calculated to be 99.5 %. *Figure 4-17,* derived from *Figure 4-16,* highlights observations made with respect to the average CIs and age categories.

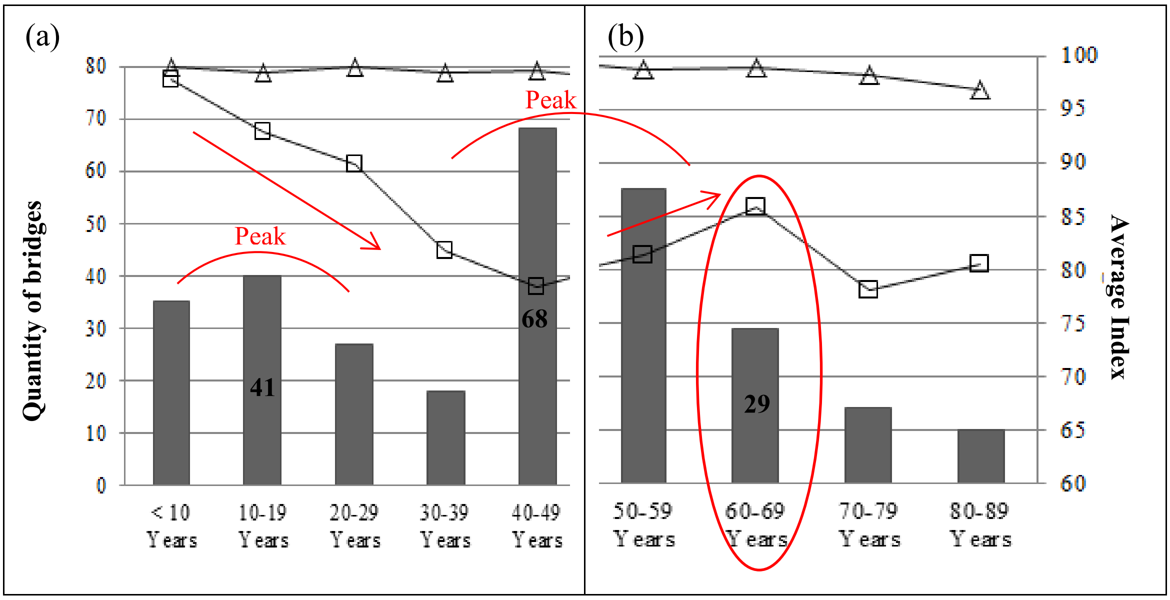

Figure 4-17: (a) – (b) Analysis of CI Relative to Age Categories

There are two age category peaks in *Figure 4-17*; one at the age category of 10 to 19 years and the other at the age category of 40 to 49 years. The peaks suggest that there were two main construction periods; that from 1924 to 1974 and the other from 1975 to the current year.

The average CIs show a decrease with increasing age categories from the age category less than 10 years to that of 40 to 49 years, as seen in *Figure 4-17(a).* This suggests a decrease in average condition with increasing age which was to be expected. The largest quantity of culverts was constructed between the period between 1965 and 1974. Thus the low average CI of the culverts of age category 40 to 49 years is assumed also characteristic of this.

Referring to *Figure 4-17(b),* the average CIs for age categories 50 to 59 years is 81.4 and the average CI for age categories 60 to 69 years is 86.3, showing an increases in average CIs when compared to the previous years. The bridges in age category 50 to 59 years were also mostly reinforced concrete with the exception of one composite. Almost 60 % of these were located inland (21 in the Central Karoo DM and 11 in the Cape Winelands DM) and the others distributed across the coastal DMs (8 in the West Coast; 7 in Eden and 7 in Overberg).

The culverts in the age category 60 to 69 years were mostly reinforced concrete simply supported in-situ portal frames, with the exception of 3 continuous and 2 concrete pipe culverts. Over half the bridges are located inland (11 in the Cape Winelands DM and 4 in Central Karoo DM). The remainder are located along the coastal district municipalities (5 in the West Coast DM; 4 in the Eden DM; 4 in the Overberg DM and 1 in the City of Cape Town). The average CIs for age groups 50 to 59 years and 60 to 69 years, which increased with increasing age, were therefore attributed to several reasons including the following:

1. The design and use of a more durable concrete mix resulting in improved long-term performance (high CI).

2. The construction techniques employed during this period.

3. Approximately 60 % of the culverts from the age category 50 to 59 years as well as 52 % of the culverts from age category 60 to 69 years were located in inland municipalities. It was therefore assumed that they had been less susceptible to the consequences of the marine environment hence their higher average condition.

In addition, the average CI of 78.8 observed in bridges of age category ranging from 80 years to 89 years is higher than the average CI of 78.1 for the age category of 70 to 79 years. However, the difference was considered reasonably minor.

4.2.6 Condition of Structures

Figure 4-18 and *4-19* represent the condition and priority indices for the bridges and culverts. There were 112 bridges (13 %) and 156 culverts (9.9 %) with CIs lower than 70, suggesting they had visible deterioration and that would require preventative maintenance or renewal of isolated parts.

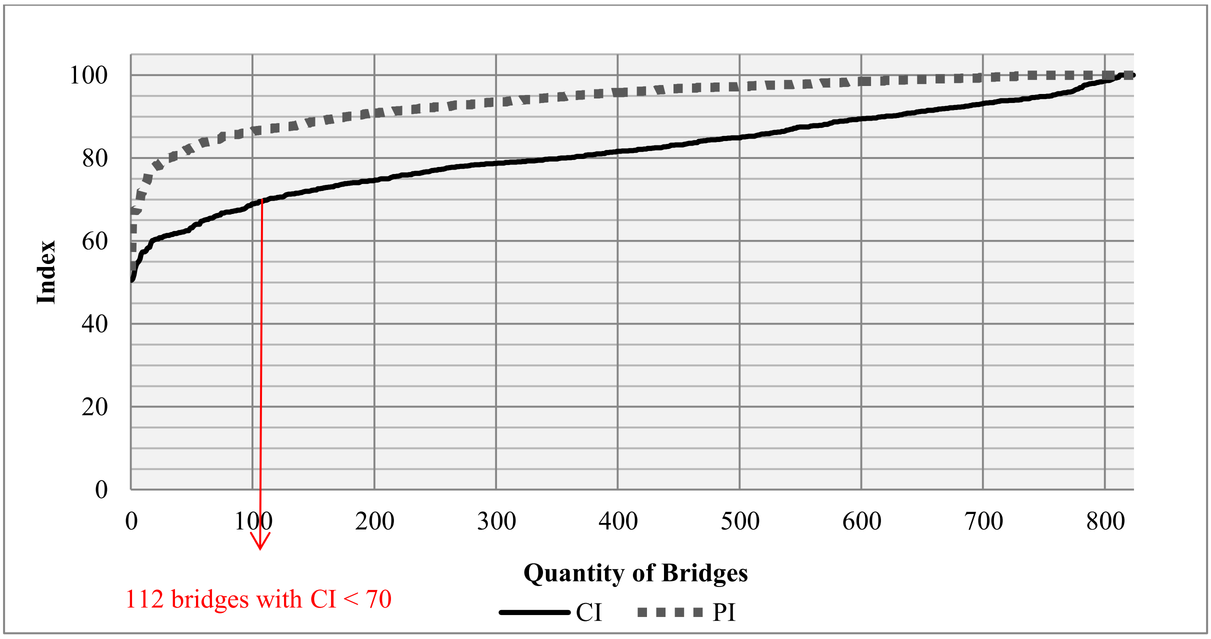

Figure 4-18: CI and PI of Bridge Structures

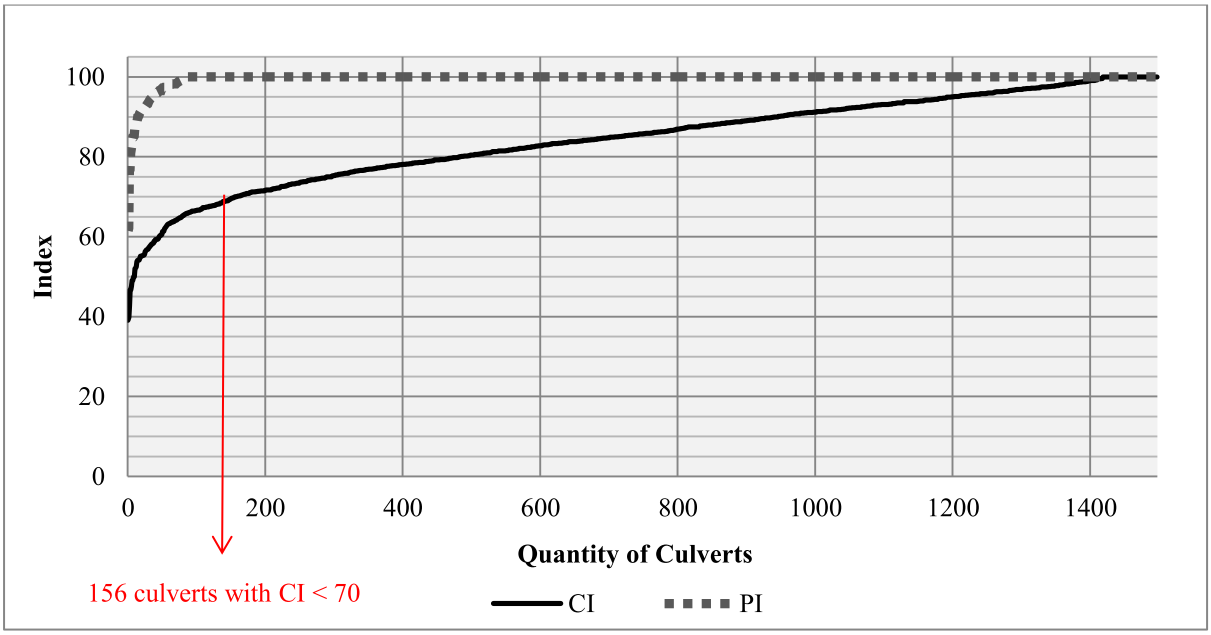

Figure 4-19: CI and PI for Culvert Structures

In summary, the data suggested that there were almost twice as many culverts as there were bridges. The Eden DM contained the largest amount of structures, which constituted about 25 % of their total sum. On the other hand, the City of Cape Town contained the least quantity of structures which, constituted just below 5 % of the total. The remaining structures were distributed between the Cape Winelands DM (22 %); the Central Karoo DM (16.3 %); the Overberg DM (13.4 %) and the West Coast DM (18.2 %). Also, it may be noted that 61 % of the structures, the majority, were located along the costal district municipalities.

Reinforced concrete was the most dominant material type as it constituted 92 % of the bridges and 88 % of the culverts. Furthermore, the majority of the (76 %) bridges were simply supported bridges and the majority of the culverts were simply supported in-situ (65 %) and precast portal frame culverts (16.4 %). Thus portal frame culverts (81.4 %) constituted the majority of culvert structure types.

Approximately 53 % of the bridges had span lengths less than 10 m and 61 % of the culverts had widths less than 4 m. The average bridge length was 43.6 m and the average culvert length was 9.5 m. The combined lengths of the bridges and culverts were approximately 36 km and 14 km, respectively. The average bridge length and span as well as the average culvert length and width suggested that the bridges and culverts in the Western Cape Province were of medium length and span/width.

The average bridge age was 47 years and the average culvert age was 37 years. When comparing increasing age categories with average CIs, for both bridges and culverts, the findings may be summarised as follows:

(a) The average CIs of the bridges exhibited a general decrease with increasing age. However, the bridges that were less than 10 years had a lower average CI than those from 10 to 19 years. The low CI was assumed to be a consequence of their location as most of them (83 %) were located in the costal DMs and were therefore more susceptible to the mechanism of chloride ingress.

    Bridges in the age category of 30 and 39 years had the lowest average CI which was associated with the possibility of compromised quality due to the rapid construction over this period as well as the construction techniques used. Also, a large quantity of these bridges (65 %) was located in the coastal DMs and may have experienced chloride ingress. Additionally, they may have also experienced carbonation.

(b) The average CIs of the culverts showed a more pronounced decrease with increasing age. However the age categories of 50 to 59 years and 60 to 69 years showed an increase in average CIs. The increase in average CIs were attributed to the design and use of a more durable concrete mix and the construction techniques used. Furthermore, 60 % of the culverts from the age category of 50 to 59 years and 52 % of those from the age category of 60 to 69 years were located in the inland district municipalities (the Cape Winelands DM and the Central Karoo DM) thus decreasing their exposure to the marine environment.

The average CI and PI computed for bridges was 81.9 and 92.6, respectively. The average CI and PI computed for culverts was 84 and 99.5, respectively, suggesting that the culverts were generally in better condition than the bridges. The average PIs of the culverts were higher than those of bridges even when their corresponding CIs were low. This implied that the BMS regarded the condition of bridges generally higher than that of the culverts. There were 112 (13 %) bridges and 156 (9.9 %) culverts with CIs lower than 70 indicating they would require remedial work activities. Nonetheless, the majority of the structures were regarded in good condition as their CIs were greater than 70.

4.3 Predominant Defects on RC Bridges

The inventory data provided in the previous section indicates that there are approximately 788 RC bridges. The distribution of defects on these structures is illustrated in *Figure 4-20*. Refer to *Appendix A* and *Appendix B* for tabulated DM specific data pertaining to inventory and inspection data relevant to this section.

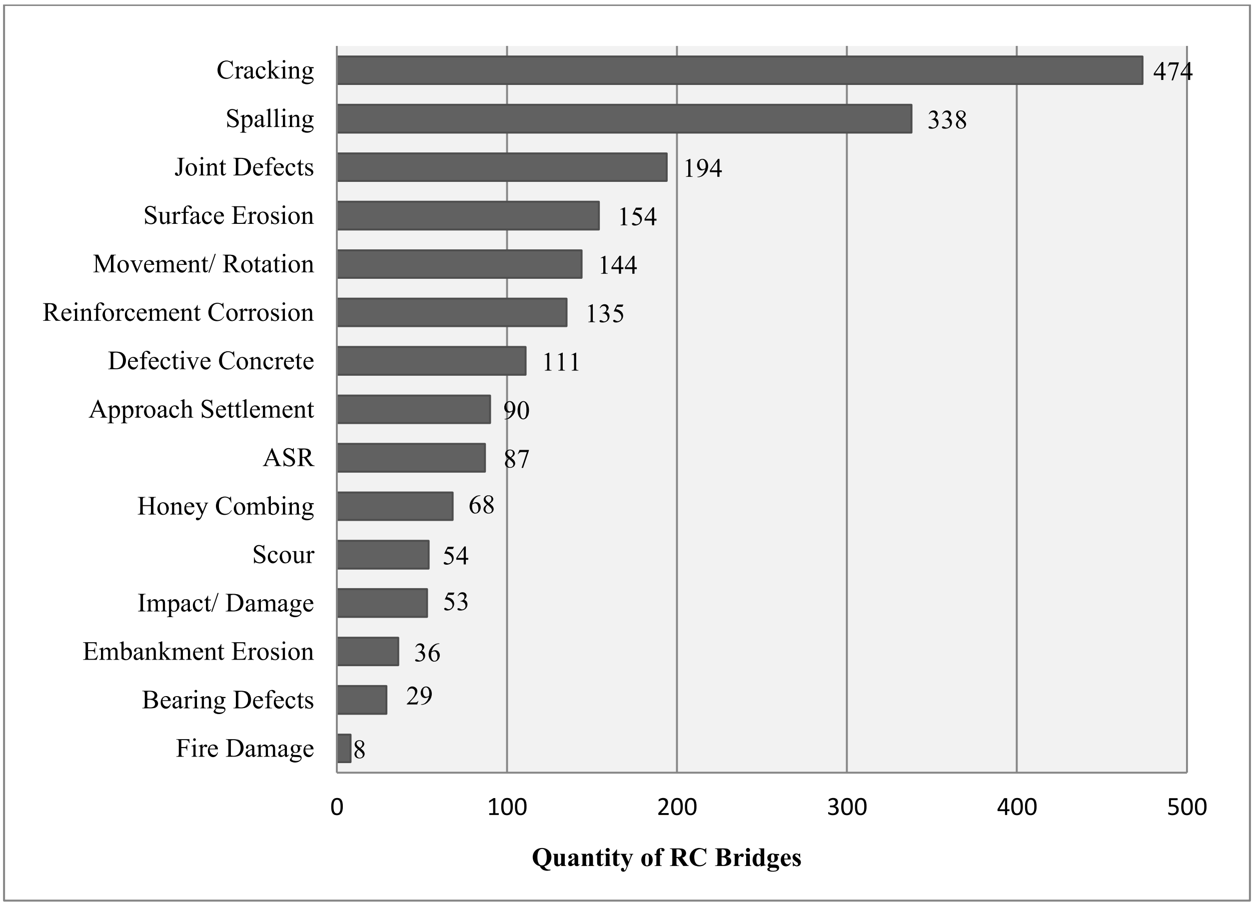

Figure 4-20: Distribution of Defects across RC Bridges in the Western Cape

Figure 4-20 shows that cracking and spalling were the most common defects among RC bridges. A total of 60 % of RC bridges had cracking and about 50 % of RC bridges had spalling. Over 15 % of the bridges also had joint defects, surface erosion, movements/ rotation as well as reinforcement corrosion. Hence these defects were identified as the most predominant RC bridge defects in the Western Cape.

The distribution of defects on the RC bridges was investigated per district municipality and the findings graphically summarized in *Figure 4-21.* A summary of their predominance was given in *Table 4-6*.

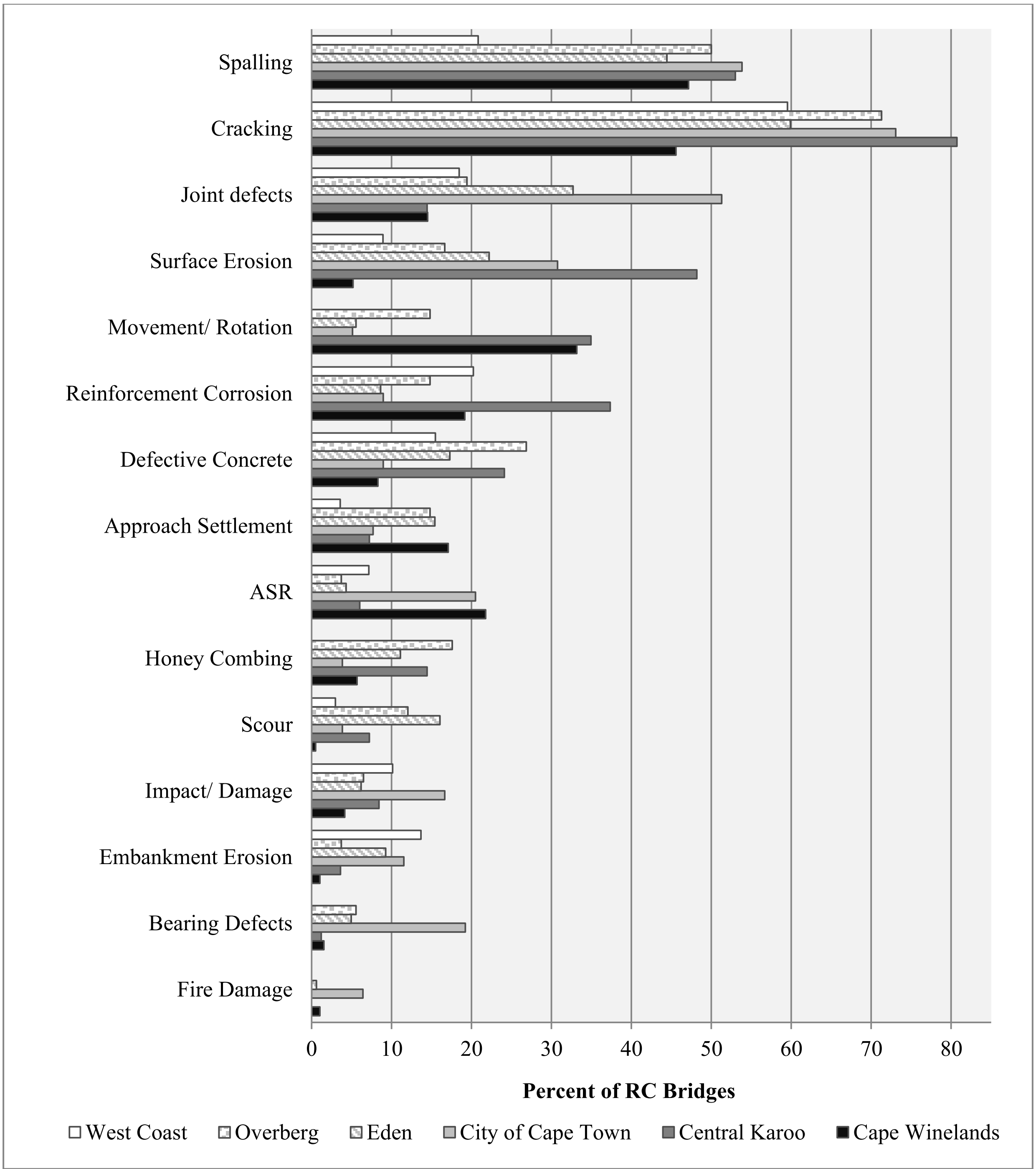

Figure 4-21: Distribution of Defects on RC Bridges within the Various District Municipalities

Table 4-6: Predominant Defects on RC Bridges in order of Frequency per District Municipality

	District Municipality					
	Cape Winelands	**Central Karoo**	**CPT**	**Eden**	**Overberg**	**West Coast**
1.	Spalling	Cracking	Cracking	Cracking	Cracking	Cracking
2.	Cracking	Spalling	Spalling	Spalling	Spalling	Spalling
3.	Movement/ Rotation	Surface Erosion	Joint Defects	Joint Defect	Joint Defect	Reinforcement Corrosion
4.	ASR	Reinforcement Corrosion	Surface Erosion	Surface Erosion	ASR	Defective Concrete
5.	Reinforcement Corrosion	Movement/ Rotation	ASR	Defective Concrete	Defective Concrete	Impact/ Damage

The highest percentage of RC bridges with cracking was in the Central Karoo DM (80.7 %) followed by the City of Cape Town (73.1 %) and the Overberg DM (71.3 %). The percentage of bridges with cracking in the Eden DM (59.9 %) and the West Coast DM (59.5 %) were relatively similar and the Cape Winelands DM had the least percentage of RC bridges with cracking (45.6 %). The percentage of RC bridges with spalling was generally higher for district municipalities with higher percentages of RC bridges with cracking and lower for those with lower percentages of RC bridges with cracking. Thus it followed that a larger percentage of the RC bridges in the Central Karoo DM had spalling compared to the percentage of RC bridges with spalling in the Cape Winelands DM. A summary of the percentage of RC bridges with cracking and spalling per district municipality is given in *Table 4-7.*

Table 4-7: A Summary of the Percentages of Total RC Bridges with Cracking and Spalling in the District Municipalities

District Municipality	**Cracking (%)**	**Spalling (%)**
Cape Winelands	45.6	47.2
Central Karoo	80.7	53.0
City of Cape Town	73.1	53.0
Eden	59.9	44.4
Overberg	71.3	50.0
West Coast	59.5	20.8

The predominant defects of cracking; spalling and reinforcement corrosion were grouped as they were regarded related and typical of the mechanism of chloride ingress (which was very common in the Western Cape Province) and possibly carbonation. District municipalities with higher percentages of RC bridges with cracking and spalling did not necessarily have higher percentages of reinforcement corrosion. Similarly, the district municipalities with lower percentages of RC bridges with cracking and spalling did not necessarily have lower percentages of RC bridges with reinforcement corrosion (see *Figure 4-22*).

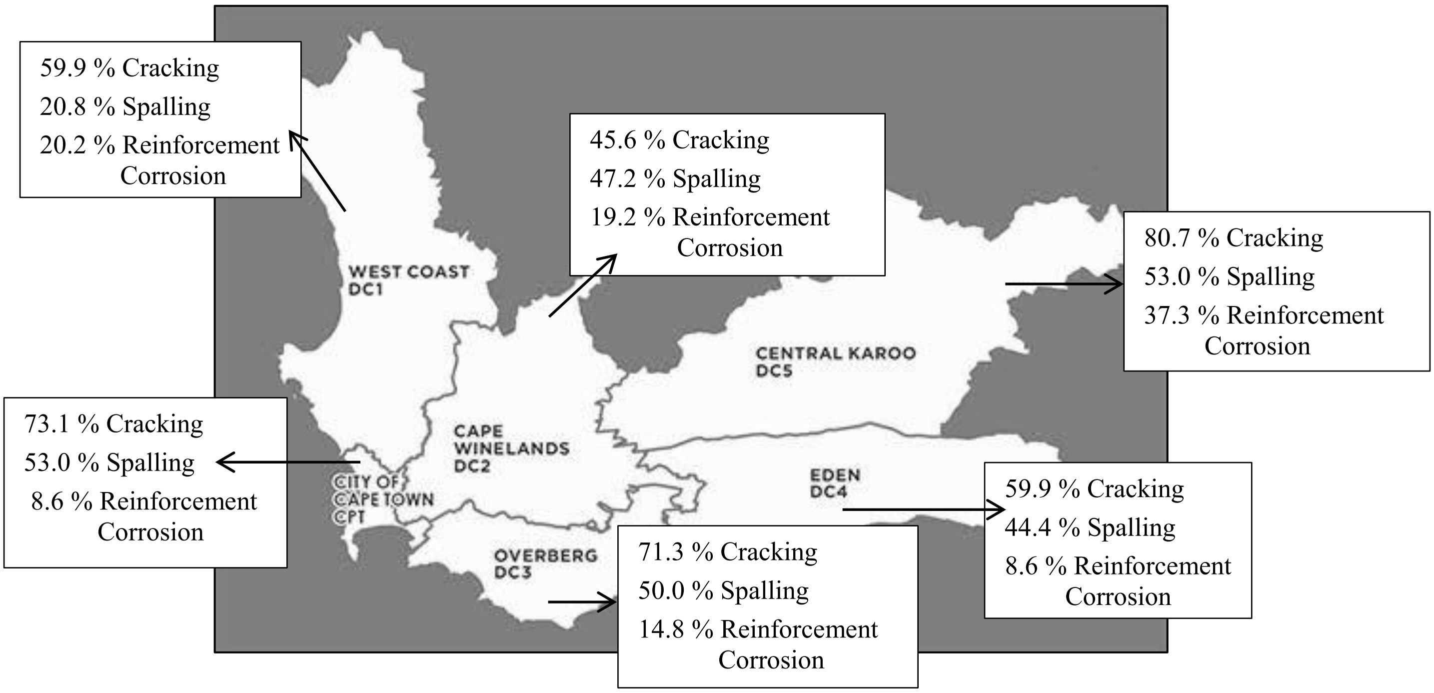

Figure 4-22: Cracking, Spalling and Reinforcement Corrosion of RC Bridges in the District Municipalities

Reinforcement corrosion is a time dependant defect which is expected to propagate with time. Thus the mean age of RC bridges in each district municipality was investigated as well as the standard deviation of the ages and summarized in *Table 4-8*.

Table 4-8: Mean Age and Standard Deviation for RC Bridges of each District Municipality

	Cape Winelands	Central Karoo	CPT	Eden	Overberg	West Coast
Mean	47.2 Years	50.1 Years	29.9 Years	44.0 Years	50.4 Years	51.5 Years
Standard Deviation	18.15	11.75	14.74	16.00	18.50	14.58

The findings showed that the Central Karoo DM not only had the highest percentage of RC bridges with cracking and spalling as well as a relatively higher percentage of RC bridges with reinforcement corrosion, these RC bridges also had a relatively high mean age (50.1 years). Therefore the higher percentage of RC bridges with cracking, spalling and reinforcement corrosion could be explained by the mean age of the RC bridges being higher when compared to those of other district municipalities. Furthermore, the cracking spalling and reinforcement corrosion could also be attributed to the mechanism of carbonation which would have had enough time to propagate into the concrete and corrode the reinforcements.

The RC bridges in the Overberg DM had one of the highest mean age (50.4 years) but also had the highest standard deviation (18.5). Thus the high percentage of RC bridges with cracking, spalling and reinforcement corrosion could also be linked to the age of the RC bridges. However, the spread of/higher variation in the age may explain why the percentage of RC bridges with cracking spalling and reinforcement corrosion in this district were not necessarily the highest. It was interesting to note that the bridges in the City of Cape Town had the lowest mean age (29.2 years) and a lower variation in age (14.7) however the percentage of RC bridges with cracking and spalling was almost equivalent to those in the Overberg DM. Nonetheless, the lower percentage of RC bridges with reinforcement corrosion may be explained by that the RC bridges were built more recently relative to those of other district municipalities.

A higher percentage of the RC bridges in the West Coast DM (20.2 %), boarded by the Atlantic Ocean, exhibited reinforcement corrosion than those located in the Eden DM (8.6 %), which was boarded by the Indian Ocean. However, the RC bridges in the West Coast DM had a mean age of 51.5 years which was higher than that of the RC bridges in the Eden DM (44.0 years). Thus the higher percentage of reinforcement corrosion may not only be attributed to the generally higher age but also the climatic conditions of the location as the coastal area produces winds which aid the transportation and deposition of chloride ions onto RC structures facilitating and accelerating their deterioration.

Lastly a relatively lower percentage of RC bridges in the Cape Winelands have cracking and spalling and reinforcement corrosion. The lower percentage of RC bridges with these defects may be explained by the lower mean age (47.2 years) and assumed characteristic of the inland environment where the rates of chloride penetration are expected to be lower.

The distribution of the cracking and spalling on the critical structural elements of RC bridges was investigated and represented in *Figure 4-23.* The pie charts suggest that most of the cracking (40 %) was found located on the abutments and most of the spalling (50 %) on the deck/ beams.

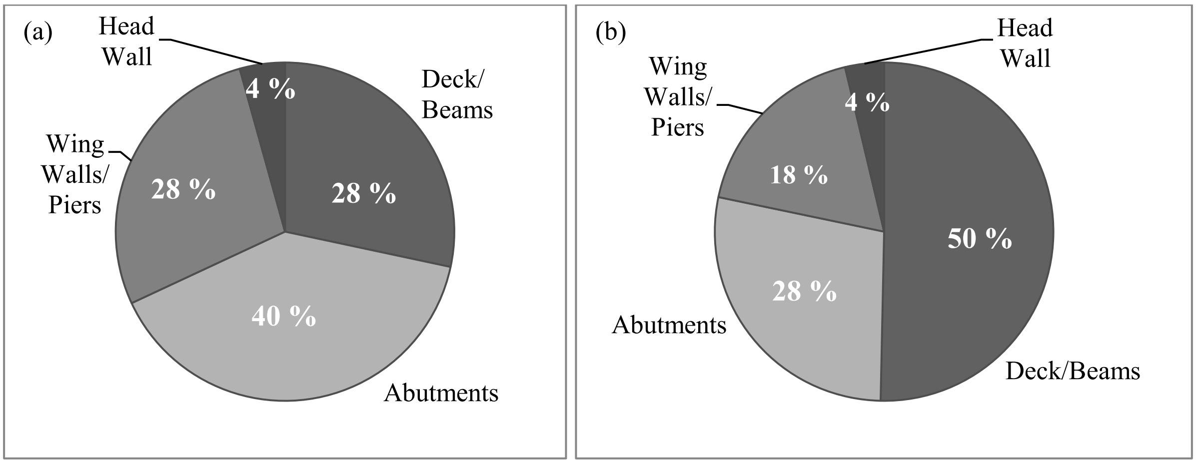

Figure 4-23: Distribution of (a) Cracking and (b) Spalling on Critical Structural Elements of RC Bridges

The other predominate defects identified which were joint defects, surface erosion and movements/rotations were also investigated. The joint defects, prevalent in approximately 25 % of RC bridges, were more dominant in RC bridges located in the Eden DM (32.7 %), the City of Cape Town (51.3 %) and the Overberg DM (19.4 %). Multiple factors including; small movements such as daily thermal cycles, traffic induced movements from loading, faulty installation of materials, extended service life, lack of routine maintenance and debris consequently collecting in the seals and poor surface drainage were associated with these defects. Furthermore, these defects were assumed to be linked to those of impact damage and visible movements/rotations. Lastly, the predominant defect of surface erosion was assumed to be related to cracking, spalling and reinforcement corrosion as well as typical of the climatic conditions as the Western Cape experienced long winters which had a high number of intense rainfall days.

A logistic regression analysis was conducted to examine the significance of the various locations of the predominant defects on RC bridges in relation to the base category; a RC bridge in the Cape Winelands DM. The results are tabulated in *Table 4-9*.

Table 4-9: Results from Logistic Regression Analysis between Defects and District Municipalities for RC Bridges

Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
			Cracking			
Central Karoo	4.901278	1.538234	5.06	0.000	2.649508	0.066788
City of Cape Town	4.849026	1.606355	4.77	0.000	2.533225	9.281866
Eden	1.957484	.4316376	3.05	0.002	1.270583	3.015738
Overberg	2.080808	.5283579	2.89	0.004	1.265014	3.422699
West Coast	1.696311	.3620944	2.48	0.013	.6428094	2.577525
			Spalling			
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
Central Karoo	1.239786	.3265072	0.82	0.414	.7399053	2.077386
City of Cape Town	1.629405	.4571856	1.74	0.082	.9401458	2.823988
Eden	.9532636	.2065001	-0.22	0.825	.6234793	1.457485
Overberg	1.239186	.3063913	0.87	0.386	.1823847	2.011861
West Coast	.2913753	.069648	-5.16	0.000	.6850509	.4654971
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
			Joint Defects			
Central Karoo	.9839034	.3671781	-0.04	0.965	.4734747	2.044599
City of Cape Town	7.699309	2.416376	6.50	0.000	1.797471	14.24281
Eden	3.02486	.803276	4.17	0.000	1.932374	5.090363
Overberg	3.418934	-995311	4.22	0.000	.5057948	6.049093
West Coast	.9170744	.2784296	-0.29	0.776	.1150375	1.66278
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
			Surface Erosion			
Central Karoo	16.83718	6.602677	7.20	0.000	7.806779	36.3134
City of Cape Town	9.627643	3.932056	5.54	0.000	4.323924	21.4369
Eden	5.475621	2.061234	4.52	0.000	2.618234	11.45139
Overberg	7.038877	2.772352	4.95	0.180	3.252721	15.23211
West Coast	1.762984	.7452334	1.34	0.000	.7699027	4.03702
Independent	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z|**	**[95 % Conf. Interval]**	

Variable				(p -value)		
Movement/ Rotation						
Central Karoo	1.065683	.2947539	0.23	0.818	.6197219	1.832564
City of Cape Town	.116728	.0626651	-4.00	0.000	.0407577	.3343028
Eden	.1223245	.0460076	-5.59	0.000	.0585285	.2556582
Overberg	.1493616	.0628578	-4.52	0.000	.0654661	.3407698
West Coast	.4457654	.1118893	-3.22	0.001	.2725528	.729058
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
Reinforcement Corrosion						
Central Karoo	2.481289	.7234537	3.12	0.002	1.401193	4.393967
City of Cape Town	.4482328	.1962842	-1.83	0.067	.1900014	1.057427
Eden	.4132643	.1383302	-2.64	0.008	.2144415	.7964288
Overberg	.5675676	.2032316	-1.58	0.114	.281335	1.145015
West Coast	1.048248	.2780126	0.18	0.859	.6233208	1.762855

The Central Karoo DM's odds ratio for cracking was approximately 4.90, which is higher than that of any other district. This indicated that a RC bridge in the Central Karoo DM was 4.90 times more likely to exhibit cracking as opposed to the base category. In contrast, a RC bridge in the West Coast DM, which had an odds ratio of about 1.70, was found less likely to have cracking relative to the base category when compared to the other district municipalities.

The West Coast DM was the only statistically significant predictor variable for spalling and surface erosion. Hence, results obtained regarding other district municipalities could be disregarded. The odds ratio indicated that a RC bridge in the West Coast DM was 0.29 times more likely to have spalling and 16.8 times more likely to have surface erosion in comparison to the base category.

The investigation involving joint defects and district municipalities indicated that the statistically significant locations were the City of Cape Town, the Eden DM and the Overberg DM. A bridge in the City of Cape Town was found to have a higher likelihood of joint defects relative to the base category, when compared to a RC bridge in the Eden or Overberg DM. The likelihood was estimated at 7.7 times more than that of the base category.

The statistically significant predictor variables for movement/rotations were the City of Cape Town, the Eden DM, the Overberg DM and the West Coast DM. The odds ratios indicating the likelihoods of RC bridges in these district municipalities to have movement and/or rotation associated defects to be about 0.12, 0.12, 0.15 and 0.44 correspondingly.

Lastly, only the RC bridges in the Central Karoo and Eden DMs were statistically significant predictor variables for reinforcement corrosion. These locations had corresponding odds ratios of 2.48 and 0.41. Thus the RC bridges in the Central Karoo DM were more likely to have reinforcement corrosion, relative to the base category, as opposed to those in the Eden DM. It may be noted that the interpretation for odds ratios with corresponding p-value < 0.05 followed similar reasoning throughout the analysis.

The RC bridges in the Central Karoo DM generally had higher odds ratios compared to the other district municipalities. For this reason the average CIs and PIs of the RC bridges were investigated per district municipality and the findings graphically illustrated in *Figure 4.24* and summary statistics for the CIs provided in *Table 4-10*. The following observations and deductions were made:

- RC bridges in the Eden DM had the highest average CI which was 83.5 whereas those in the Central Karoo DM had the lowest average CI which was 77.1. The low average CI of the RC bridges in the Central Karoo DM may be assumed related to several factors related to improper construction techniques and practices employed during casting in-situ which adversely affect the durability of RC bridges. These include less care being taken with regards to curing, compaction, the use of clean as opposed to dirty mixing water, inaccurate batching and ensuring even cover depths. Also, the Central Karoo DM is a semi-arid environment, improper practices such as inadequate curing of concrete during construction in such a location may lead to premature cracking and not only compromise the concretes strength thereafter but also allow easy formation of other defects and consequently further deterioration.

- It was interesting to note that the West Coast DM had the highest average CI of 82.4. However, it also had a high percentage of RC bridges with cracking, spalling and reinforcement corrosion and the highest mean age relative to other districts. Thus, it may be assumed that the defects were either not generally located on the structures critical elements or that they were common but not severe or both.

- The properties of each district municipality's average CI are tabulated in *4-10.* It can be seen that that the CIs of the various district municipalities have had similar deviations from their means and that the data was left skewed and fat-tailed.

- The averages CIs were all greater than 70, indicating the structures were generally in a good condition.

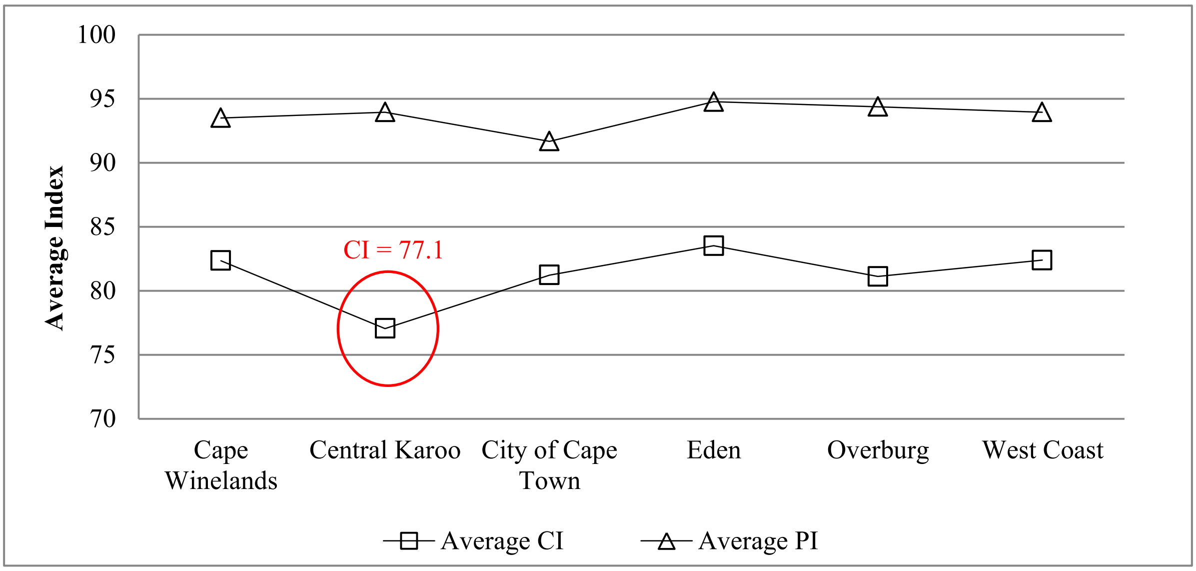

Figure 4-24: Average CI and PI for RC Bridges per District Municipality

Table 4-10: Summary of Properties for RC Bridge CIs per District Municipality

	District Municipality					
	Cape Winelands	**Central Karoo**	**CPT**	**Eden**	**Overberg**	**West Coast**
Observations	191	83	76	159	99	169
Mean	82.4	77.1	81.2	83.5	81.1	82.4
Min	50.5	53.2	51.8	57.6	57.3	58.5
Maximum	100	95.6	100	100	100	98.7
Standard Deviation	10.9735	9.489434	10.94565	9.894881	10.3292	9.418514
Variance	120.4177	90.04936	119.8072	97.90868	106.6925	88.70841
Skewness	-0.5361323	-0.3881352	-0.6925752	-0.228067	-0.3150188	-0.2911582
Kurtosis	2.688596	3.07323	2.833797	2.567355	2.285653	2.27703

4.4 Predominant Defects on RC Culverts

The inventory data provided in the previous section indicated that there are a total of about 1370 RC culverts. The distribution of defects on these structures is graphically illustrated in *Figure 4.25*.

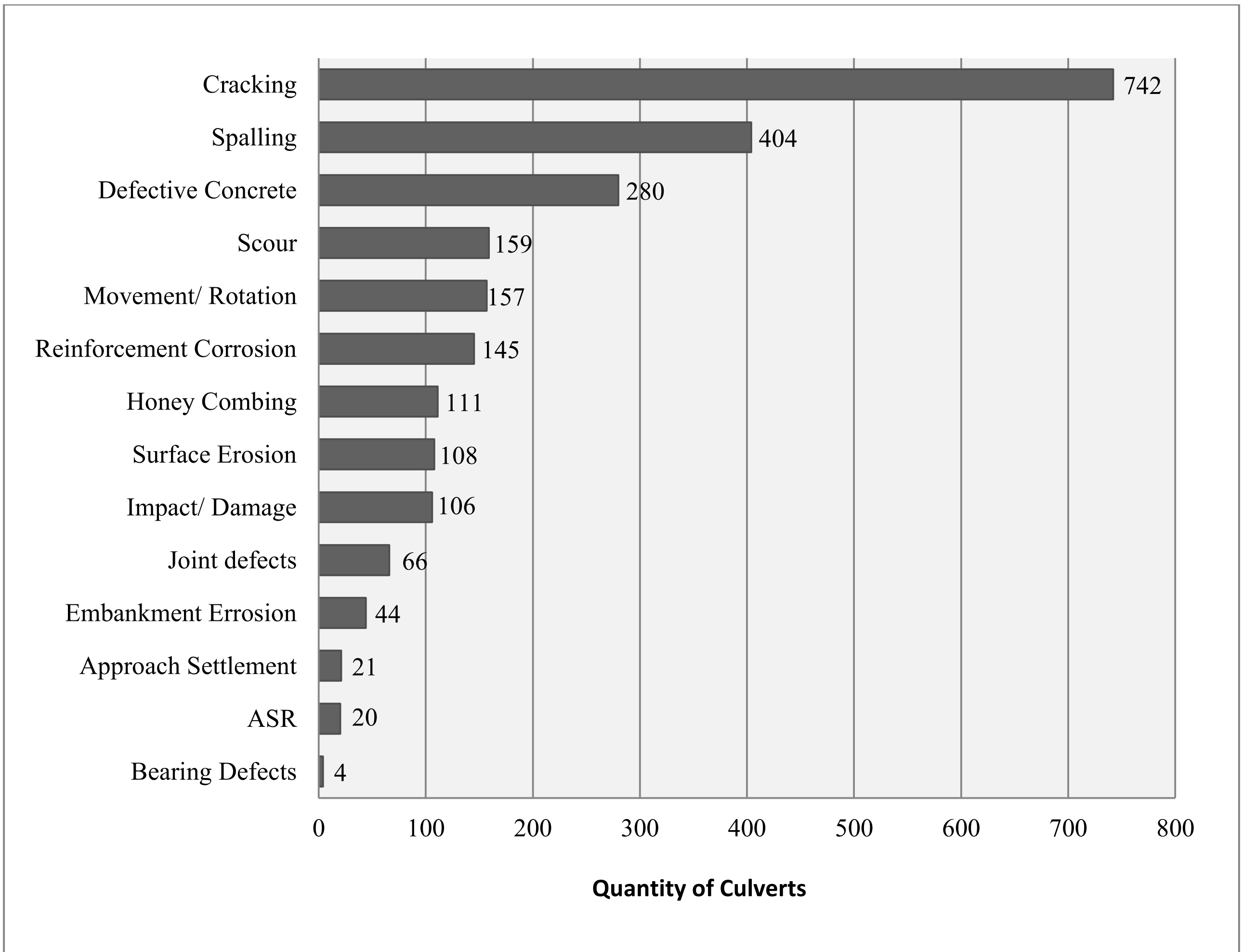

Figure 4-25: Distribution of Defects across RC Culverts in the Western Cape

Figure 4-25 shows that cracking and spalling were also the most common defects among culverts as they have the highest observed frequency when compared to the other defects. Approximately 54 % of the total RC culvert structures had cracking and 30 % had spalling. Almost 10 % of the culverts also had defective concrete, scour, movements/ rotation and reinforcement corrosion. Hence these defects were identified as the most predominant RC bridge defects in the Western Cape.

The distribution of defects on the RC culverts was investigated per district municipality and the findings graphically summarized in *Figure 4-26*. Also, a summary of their predominance is provided in *Table 4-11*.

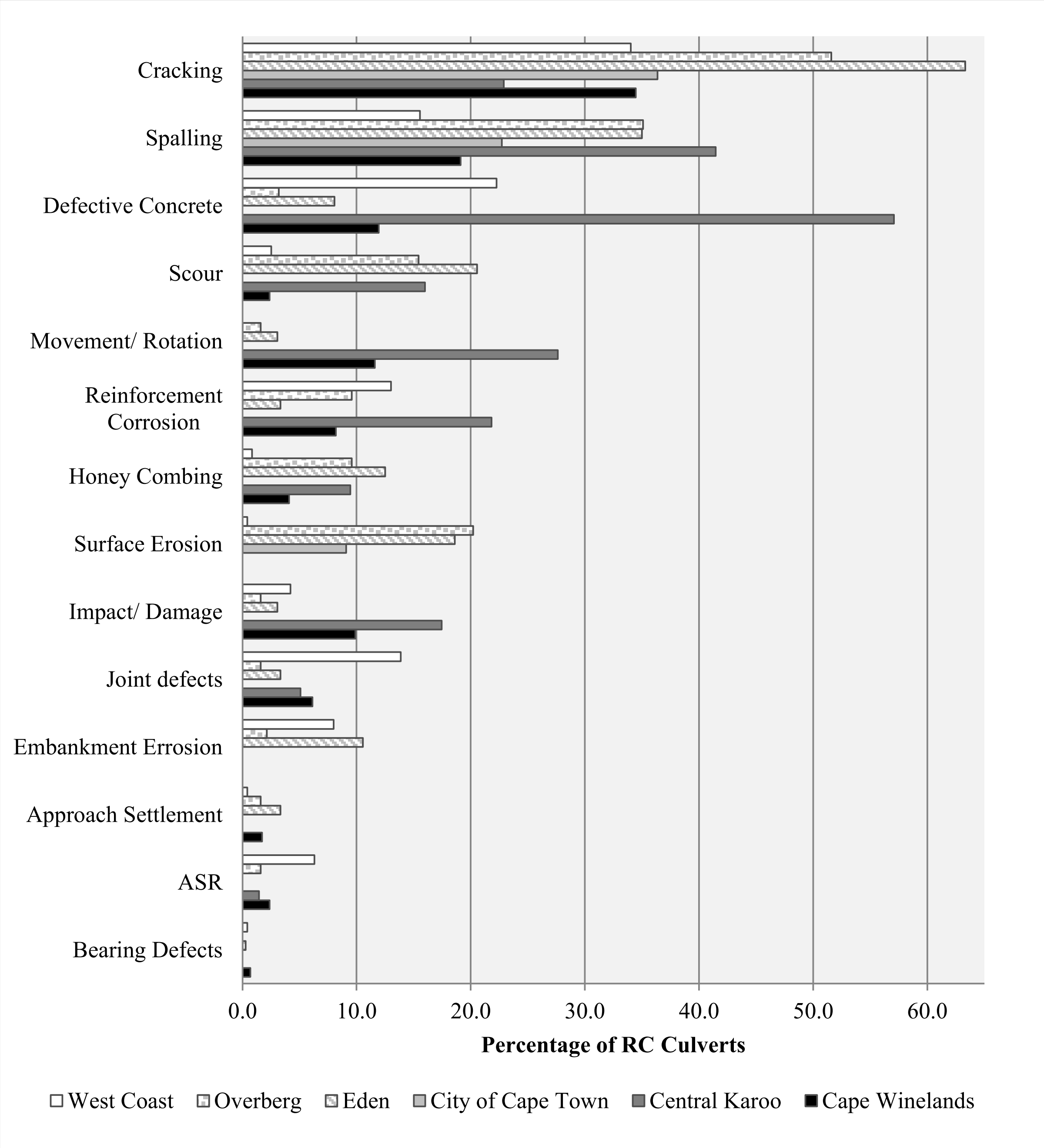

Figure 4-26: Distribution of Defects on RC Culverts within the Various District Municipalities

Table 4-11: Predominant Defects on RC Culverts in order of Frequency

	District Municipality					
	Cape Winelands	**Central Karoo**	**CPT**	**Eden**	**Overberg**	**West Coast**
1.	Cracking	Defective Concrete	Cracking	Cracking	Cracking	Cracking
2.	Spalling	Spalling	Spalling	Spalling	Spalling	Defective Concrete
3.	Defective Concrete	Cracking		Scour	Surface Erosion	Spalling
4.	Movement/ Rotation	Movement/ Rotation		Surface Erosion	Scour	Reinforcement Corrosion
5.	Impact Damage	Impact Damage		Honeycombing	Honeycombing Reinforcement Corrosion	Joint Defects Embankment Erosion

The largest percentage of RC culverts with cracking was in the Eden DM (63.3 %) followed by the Overberg DM (51.6 %). The percentage of RC culverts with cracking in the City of Cape Town (36.4 %), Cape Winelands DM (34.5 %) and the West coast DM (34.0 %) were relatively similar. The Central Karoo DM had the least percentage of RC culverts with cracking (22.9 %). The percentage of RC culverts that had spalling was generally higher for district municipalities which had higher percentages of RC bridges with cracking and lower for those with lower percentages of RC bridges with cracking with the exception of the RC culverts in the Central Karoo DM. A larger percentage of these exhibited spalling as opposed to cracking. A summary of the percentage of RC culverts with cracking and spalling per district municipality is provided in *Table 4-12*.

Table 4-12: A Summary of the Percentages of Total RC Culverts with Cracking and Spalling in the Districts

District Municipality	Cracking (%)	Spalling (%)
Cape Winelands	34.5	19.1
Central Karoo	22.9	41.5
City of Cape Town	36.4	22.7
Eden	63.3	35.0
Overberg	51.6	15.5

West Coast	34.0	15.5

As with the RC bridges, the predominant defects of cracking; spalling and reinforcement corrosion were grouped and associated with the mechanism of chloride ingress and possibly carbonation. Once again, district municipalities with relatively high percentages of RC culverts with cracking and spalling did not necessarily have relatively higher percentages of reinforcement corrosion. Similarly, the district municipalities with relatively lower percentages of RC culverts with cracking and spalling did not necessarily have relatively lower percentages of RC culverts with reinforcement corrosion (see *Figure 4-27*).

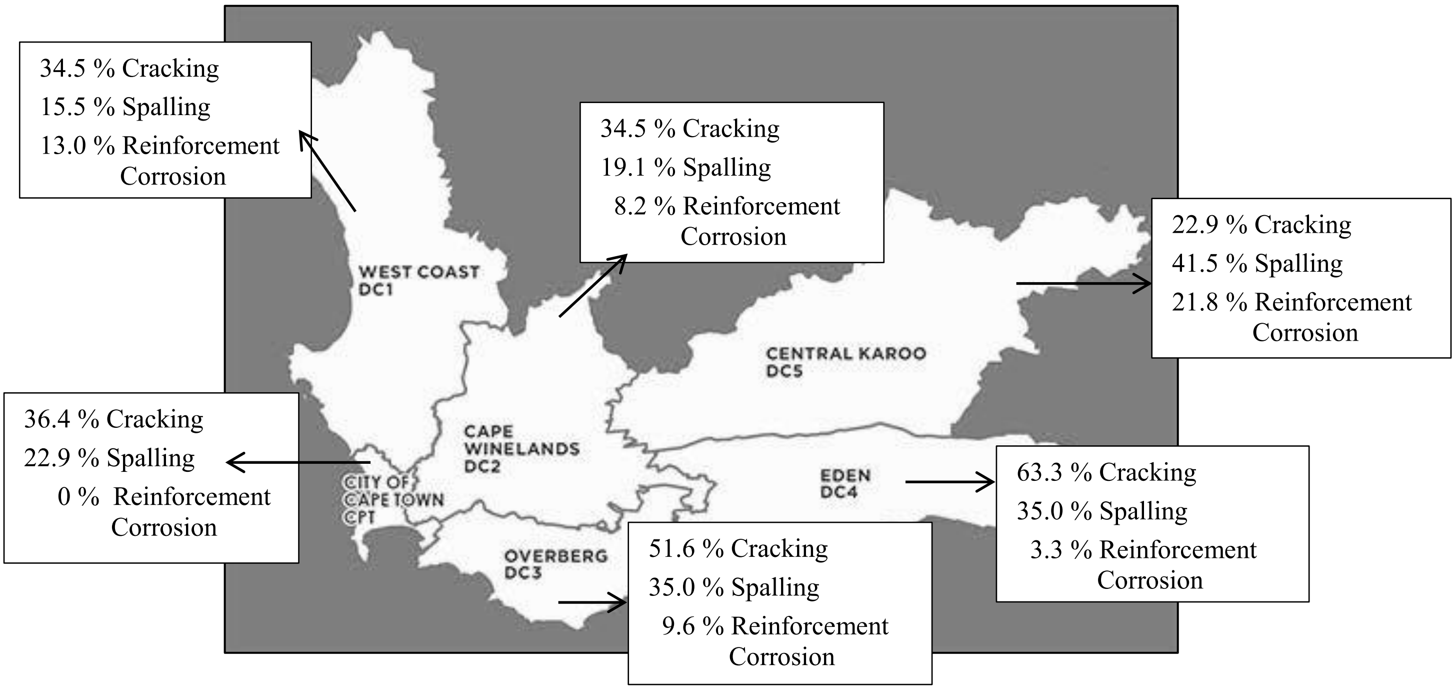

Figure 4-27: Cracking, Spalling and Reinforcement Corrosion of RC Culverts in the District Municipalities

The mean age of the RC culverts in each DM was also investigated as well as the standard deviation of the ages and summarized in *Table 4-13*.

Table 4-13: Mean Age and Standard Deviation for RC Culverts of each District Municipality

	Cape Winelands	Central Karoo	CPT	Eden	Overberg	West Coast
Mean	44.3Years	49.0 Years	18.7 Years	25.2 Years	34.1 Years	46.3 Years
Standard Deviation	18.33	10.04	20.06	23.6	23.9	24.35

The findings show that though the percentage of RC culverts with cracking and spalling was the highest in the Eden DM, the percentage of RC culverts with reinforcement corrosion (3.3 %) in Eden DM was low. The mean age of these RC culverts (25.2 years) was relatively lower than that of the other district municipalities. Thus the low percentage of RC culverts with reinforcement corrosion could be assumed characteristic of the lower mean age. The Overberg DM, also located along the coast, also had a high percentage of RC culverts with cracking and spalling and in addition to this a higher percentage of RC culverts with reinforcement corrosion (9.6 %). The mean culvert age in the Overberg DM was 34.1 years, which is higher than that of the Eden DM and was assumed to be the reason for the higher quantity of RC culverts with reinforcement corrosion.

When comparing the West Coast DM and the City of Cape Town, both bordered by the Atlantic Ocean, it may be seen that both these municipalities had similar percentages of RC culverts with cracking. Though the percentage of RC culverts with spalling is higher in the City of Cape Town, the percentage of RC culverts with reinforcement corrosion was higher in the West Coast DM. The mean ages of the RC culverts in the West Coast DM and the City of Cape Town are 46.3 years and 18.7 years, respectively. It may be noted that the spread/ variance of the age in the West coast (24.35) was significantly more than that of the City of Cape Town (10.04). Nonetheless the higher percentages of RC culverts with reinforcement corrosion in the West Coast DM may be explained by the much higher mean age.

A higher percentage of the RC culverts in the Cape Winelands DM had cracking compared to those in the Central Karoo DM. However, a higher percentage of RC culverts had spalling in the Central Karoo DM. As both district municipalities are located more inland, the structures were therefore less susceptible to the effects of a marine environment. It was interesting to note that the percentage of RC culverts in the Central Karoo DM (21.8 %) with reinforcement corrosion was much higher than that of the Cape Winelands DM (8.2 %). Moreover, it was also much higher than that of the other district municipalities. The mean age of the RC culverts in the Cape Winelands DM and Central Karoo DM were 44.3 years and 49.0 years, respectively. The RC culverts in the Central Karoo DM not only had the highest mean age, they also had the least spread in age as the standard deviation of the age was lower than that of any other district municipality (10.0). Thus the higher percentage of RC culverts with reinforcement corrosion was not only attributed to the high mean age and low spread in age but also the impacts of poor construction practices and techniques (discussed earlier) in a semi-arid environment.

The distribution of cracking and spalling on the critical structural elements of RC culverts was investigated and is represented in *Figure 4-28.* The graphic suggests that most of the cracking (35 %) and the most of the spalling (40 %) was located on the wing walls.

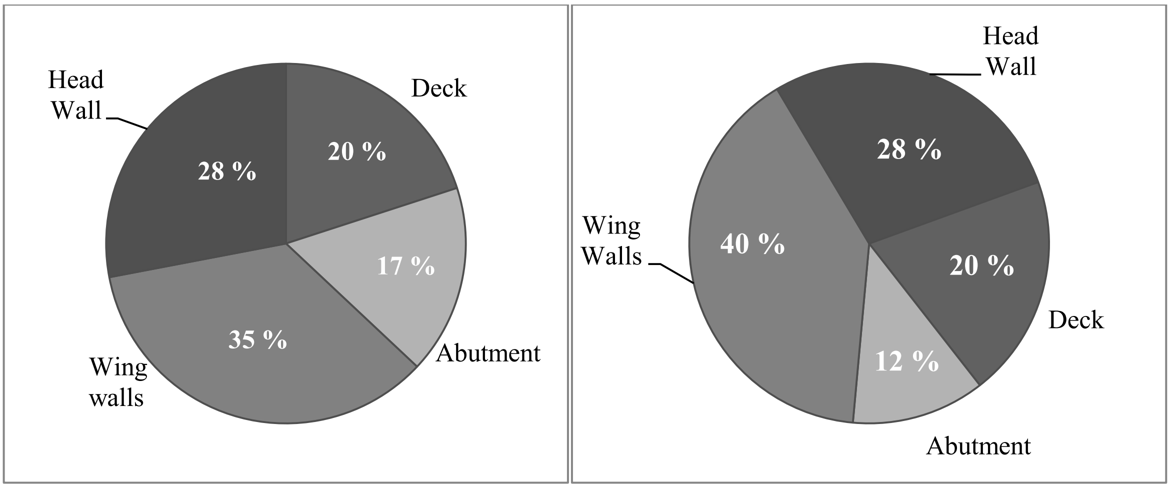

Figure 4-28: Distribution of a) Cracking and (b) Spalling on Critical Structural Elements of RC Culverts

Other predominant defects identified, i.e. were that of defective concrete, scour and movements/rotations, were also investigated. The defective concrete, prevalent in approximately 20 % of RC culverts was more dominant in RC culverts located in the Central Karoo DM (57.1 %) and the West Coast DM (22.3 %). The defective concrete was assumed as a result of the employment of the poor construction practices and techniques discussed earlier which included but are not limited to; less care being taken with regards to curing and compaction, the use of dirty as opposed to clean mixing water, inaccurate batching and insufficient concrete cover depths. The consequences of these techniques would include the concrete not reaching its desired strength and compromised quality and subsequently deterioration. The ultimate penalty being that the structure would have a reduced service life should measures not be taken to repair or reconstruct the structures. However, these measures would imply additional costs. Scour was attributed to the increased outlet velocities experienced downstream during seasons of heavy rainfall. The Western Cape experiences moderately high rainfall during winter and the culverts are therefore expected to be saturated as well as experience large variations in vortices and eddy currents over the intensely heavy rainfall days resulting in an increased likelihood of undermining and scour. Lastly, as with the RC bridges, the the movement/rotation of the RC culverts were associated with small movements such as daily thermal cycles; traffic induced movements from loading; faulty installation of materials; extended service life; lack of routine maintenance and debris consequently collecting in the seals and poor surface drainage.

A logistic regression analysis was conducted to examine the significance of the various locations on the predominant defects on RC culverts in relation to the base category; a RC culvert in the Cape Winelands DM. The results were tabulated in *Table 4-14*.

Table 4-14: Results from Logistic Regression Analysis between Defects and District Municipalities for RC Culverts

Independent Variable	Odds Ratio	Std. Err.	Z	P>|z| (p -value)	[95 % Conf. Interval]	
			Cracking			
Central Karoo	9.085738	1.836264	10.92	0.000	6.11405	13.50179
City of Cape Town	1.074965	.4944752	0.16	0.875	.4363633	2.648135
Eden	3.249325	.5351617	7.16	0.000	2.352886	4.487304
Overberg	2.050284	.392828	3.75	0.000	1.408401	2.984706
West Coast	.9952092	.1829964	-0.03	0.979	.6940611	1.427023
			Spalling			
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
Central Karoo	3.002679	.5784842	5.71	0.000	2.058352	4.380241
City of Cape Town	1.239496	.6569538	0.41	0.685	.4386254	3.502646
Eden	2.269231	.4202946	4.42	0.000	1.578424	3.262373
Overberg	2.317857	.4948464	3.94	0.000	1.52532	3.522186
West Coast	.7796429	.1813763	-1.07	0.285	.494165	1.23004
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
			Defective Concrete			
Central Karoo	9.853236	2.144724	10.51	0.000	6.431324	15.09584
City of Cape Town	1	(omitted)				
Eden	.6433319	.1701687	-1.67	0.095	.3830738	1.080408
Overberg	.2447619	.1107354	-3.11	0.002	.1008422	.5940808
West Coast	2.115062	.503923	3.14	0.002	1.32593	3.373849
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	

Scour						
Central Karoo	7.78882	3.243751	4.93	0.000	3.443326	17.61835
City of Cape Town	1	(omitted)				
Eden	10.53447	4.25801	5.83	0.000	4.770444	23.26304
Overberg	7.520473	3.254011	4.66	0.000	3.220633	17.56099
West Coast	.877464	.5196158	-0.22	0.825	.2748942	2.800871
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
Movement/ Rotation						
Central Karoo	2.912656	.660962	4.71	0.000	1.86693	4.544125
City of Cape Town	1	(omitted)				
Eden	.2391707	.0852543	-4.01	0.000	.1189298	.4809783
Overberg	.1243973	.0758809	-3.42	0.001	.0376347	.4111817
West Coast	1.227509	.3212527	0.78	0.433	.7349463	2.050187
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
Reinforcement Corrosion						
Central Karoo	3.130841	.8088095	4.42	0.000	1.88697	5.194661
City of Cape Town	1	(omitted)				
Eden	.3850576	.1396888	-2.63	0.009	.1891176	.784006
Overberg	1.196429	.39119	0.55	0.583	.6303386	2.270909
West Coast	1.680421	.4826947	1.81	0.071	.9570035	2.950683

The statistically significant district municipalities for cracking spalling, scour and movement/or rotation defects were the Central Karoo DM, the Eden DM and the Overberg DM. The Central Karoo DM had the highest odds ratios for all four defects, with the highest being 9.09 for cracking. Thus a RC culvert in the Central Karoo was more likely to have the specified defects, with reference to the base category, as opposed to a RC culvert in the Eden DM or the Overberg DM. In addition to this RC culverts in the Central Karoo DM and the Eden DM were also estimated to have corresponding odds ratios of 3.13 and 0.38 for reinforcement corrosion, in relation to the base category.

Statistically significant relationships between defective concrete and the Central Karoo DM, the Overberg DM and the West Coast DM were identified. Again the RC culverts in the

Central Karoo DM were found to be more likely to have defective concrete with reference to the base category, when compared to the other two districts. A RC culvert in the West Coast DM was the least likely to have defective concrete in relation to a RC culvert in the Overberg DM or the West Coast DM with reference to the base category as it had the lowest odds ratio estimated to be 0.24.

The City of Cape Town had an odds ratio outcome of one for defective concrete, scour, movement/rotations and reinforcement corrosion. This indicated that there was no difference found when comparing a RC bridge in the City of Cape Town to the base category.

Again the Central Karoo DM was noted to have the highest odds ratios compared to the other district municipalities. Thus the average CIs and PIs of the RC culverts were investigated per district municipality and the findings graphically illustrated in *Figure 4.29* and summary statistics for the CIs provided in *Table 4-15*. The following observations were made:

- RC culverts in the City of Cape Town had the highest average CI which was 90.1 whereas those in the Central Karoo DM had the lowest average CI which was 76.8. The low average CI of the bridges in The Central Karoo DM may be attributed to their high mean age as well as the same reasons given for the low CI of the RC bridges which included poor construction practices and techniques. On the other hand, the high CI of the RC culverts in the City of Cape Town may be linked to their lower mean age. It may be noted that there were far less RC culverts in the City of Cape Town than in any other district municipality. For this reason they could have been far easier to manage and maintain. The average CIs for the bridges in the Eden DM (84.9), the Overberg DM (86.3) and the West Coast DM (86.5) were relatively similar suggesting that though there were variations in the percentages of RC culverts with various defects, the severity of the defects may have been somewhat similar.

- As with the RC bridges, the properties of the district specific condition indices for the RC culverts in *Table 4-15* show they had similar deviations from their means and that the data was left skewed and fat-tailed.

- The average PIs were all close to 100 and others 100, indicating the structures were regarded in very good condition.

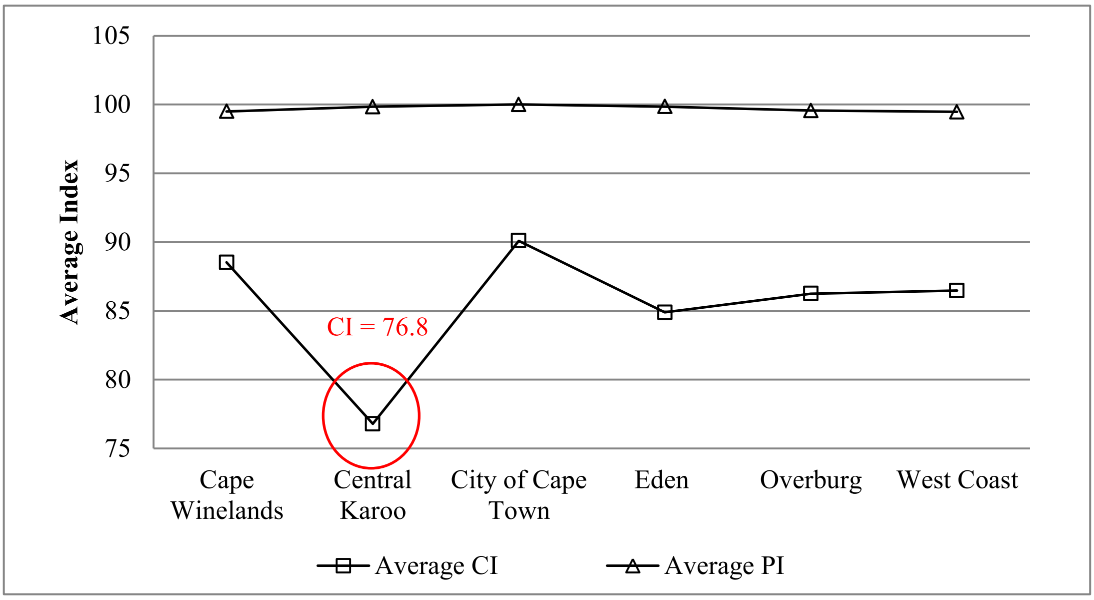

Figure 4-29: Average PI and CI for RC Culverts per District Municipality

Table 4-15: Summary of Properties for RC Culverts CIs per District Municipality

	District Municipality					
	Cape Winelands	**Central Karoo**	**CPT**	**Eden**	**Overberg**	**West Coast**
Observations	287	274	22	358	184	234
Mean	88.5	76.8	90.1	84.9	86.3	86.5
Min	50	39.8	58.9	42.7	55.1	46.7
Maximum	100	100	100	100	100	100
Standard Deviation	8.912985	10.14914	11.80916	11.27219	9.260186	9.645065
Variance	79.4413	103.005	139.4562	127.0623	85.75105	93.02728
Skewness	-1.064783	-0.3732308	-1.374165	-0.595970	-0.9938956	-0.9165625
Kurtosis	4.066225	3.580893	3.773563	2.927445	4.00885	3.791288

4.5 Investigation of Relationships between Defects and Structure Type

4.5.1 RC Bridges

An indication of the distribution of RC bridge structure types has been provided by *Figure 4.30*. It may be seen that 81 % of the RC bridges were simply supported. The other 19 % consists of the various other bridge types of which continuous RC bridges accounted for 60 % of.

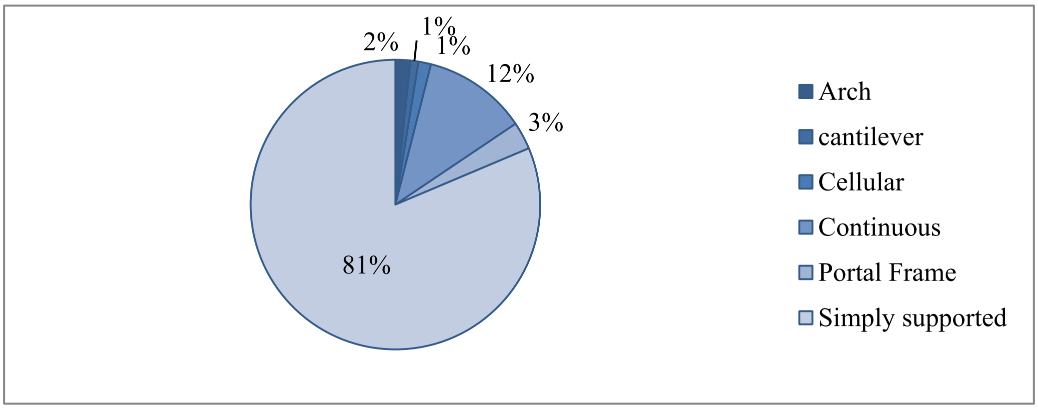

Figure 4-30: Distribution of RC Bridge Structure Types

Table 4-16 contains results obtained from the binary logistic regression analysis to measure the significance of the various structure types on the predominant defects in relation to the base category; cellular RC bridges.

Table 4-16: Results from Logistic Regression Analysis between Defects and Various Structure Types for RC Bridges

Independent Variable	Odds Ratio	Std. Err.	Z	P>|z| (p -value)	[95 % Conf. Interval]	
Cracking						
Continuous	2.810606	1.868637	1.55	0.120	.7636195	10.34482
Portal Frame	1.65	1.309023	0.75	0.454	.4039504	7.581377
Simply Supported	2.860294	1.808171	1.66	0.096	.828539	9.874348
Arch	3.9375	3.418984	1.58	0.114	.7179775	21.59386
Cantilever	1	(omitted)				

Spalling						
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
Continuous	1.830065	1.302452	0.85	0.396	.4535958	7.383532
Portal Frame	.7017544	.5917104	-0.42	0.674	.1344197	3.663596
Simply Supported	2.249754	1.533764	1.19	0.234	.5913308	8.559327
Arch	1.666667	1.475102	0.58	0.564	.2940921	9.445266
Cantilever	2.666667	2.828427	0.92	0.355	.3335287	21.32084
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
Joint Defects						
Continuous	1.844262	1.50678	0.75	0.454	.371863	9.146658
Portal Frame	.4090909	.4399517	-0.83	0.406	.0497057	3.366928
Simply Supported	1.528846	1.203365	0.54	0.590	.3268716	7.150729
Arch	2	1.972027	0.70	0.482	.2895569	22.14374
Cantilever	2,25	2.625	0.70	0.487	.2286199	1.028495
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
Surface Erosion						
Continuous	1322619	7.16e+08	0.03	0.979	0	
Portal Frame	1	(omitted)			0	
Simply Supported	779567.3	4.22e+08	0.03	0.980	0	
Arch	1434357	7.77+08	0.03	0.979	0	
Cantilever	3229589	1.75+08	0.03	0.978	0	
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
Movement/ Rotation						
Continuous	.66	.5582186	-0.49	0.623	.1257795	3.463204
Portal Frame	.6428571	.640301	-0.44	0.657	.0912636	4.528262
Simply Supported	1.131737	.8918799	0.16	0.875	.2415153	5.303297
Arch	.375	.4881406	-0.75	0.451	.0292425	4.808918
Cantilever	1	(omitted)				
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	

Reinforcement Corrosion						
Continuous	1.315789	1.449256	0.25	0.803	.1519301	11.39539
Portal Frame	1.428571	1.738491	0.29	0.769	.1315338	15.51552
Simply Supported	2.270059	2.392258	0.78	0.437	.2877447	17.90882
Arch	3	3.714835	0.89	0.375	.264909	33.97392
Cantilever	2	3.03315	0.46	0.648	.1023573	39.07878

The Walt test values (z-values) for the analysis between surface erosion and the various structure types were not statistically significant as the corresponding confidence intervals included zeroes. Hence there were no relationships found between the two. Furthermore, none of the RC bridge structure types were found statistically significant with regards to any of the defects as the p-values obtained were all greater than 0.05. Nonetheless, the relationships between cracking as well as the dominant structure types (namely, continuous RC bridges and simply supported RC bridges) were investigated per district municipality and the findings tabulated in the *Tables 4-17* and *4-18.* It may be noted that the base category was that of the structure type being investigated located in the Cape Winelands DM.

Table 4-17: Results from Logistic Regression Analysis between Cracking and Continuous RC Bridges per District Municipality

Independent Variable	Odds Ratio	Std. Err.	Z	P>|z| (p -value)	[95 % Conf. Interval]	
Central Karoo	5.5	5.474943	1.71	0.087	.7816981	38.6978
City of Cape Town	15.4	14.30077	2.94	0.003	2.495036	95.05275
Eden	3.08	2.094259	1.65	0.098	.8124052	11.67693
Overberg	2.933333	2.238916	1.41	0.159	.657155	13.09348
West Coast	7.7	7.440632	2.11	0.035	1.158674	51.170657

A continuous RC bridge located in the City of Cape Town and the West Coast DM were statistically significant with regards to predicting the likelihood of cracking. Thus when compared to a continuous RC bridge in the Cape Winelands DM (the base category), a RC bridge in the City of Cape Town was 15.4 times more likely to have cracking and an RC bridge in the West Coast was 7.7 times more likely to have cracking.

Table 4-18: Results from Logistic Regression Analysis between Cracking and Simply Supported RC Bridges per District Municipality

Independent Variable	Odds Ratio	Std. Err.	Z	P>|z| (p -value)	[95 % Conf. Interval]	
Central Karoo	4.982143	1.661145	4.82	0.000	2.591844	9.576869
City of Cape Town	3.65625	1.331324	3.56	0.000	1.790993	7.464107
Eden	1.96875	.4795246	2.78	0.005	1.22142	3.173336
Overberg	2.296875	.6723449	2.84	0.005	1.294113	4.076641
West Coast	1.708333	.3981727	2.30	0.022	1.081876	2.69754

A simply supported RC bridge in the either of the district municipalities was statistically significant when attempting to predict the likelihood of cracking. A simply supported RC bridge in the Cape Winelands DM had the highest odds ratio (4.98) compared to that of an RC bridge in the other district municipalities. Thus it was more likely to have cracking, relative to the base category, in comparison to the RC bridges in the other district municipalities. Furthermore, the percentages of the various RC bridges with the predominant defects were graphically illustrated in *Figure 4-31.* However, because the quantities of RC bridge structure types were exceptionally varied *Table 4-19* provides the quantity and percentage of RC bridges with defects per RC bridge structure type

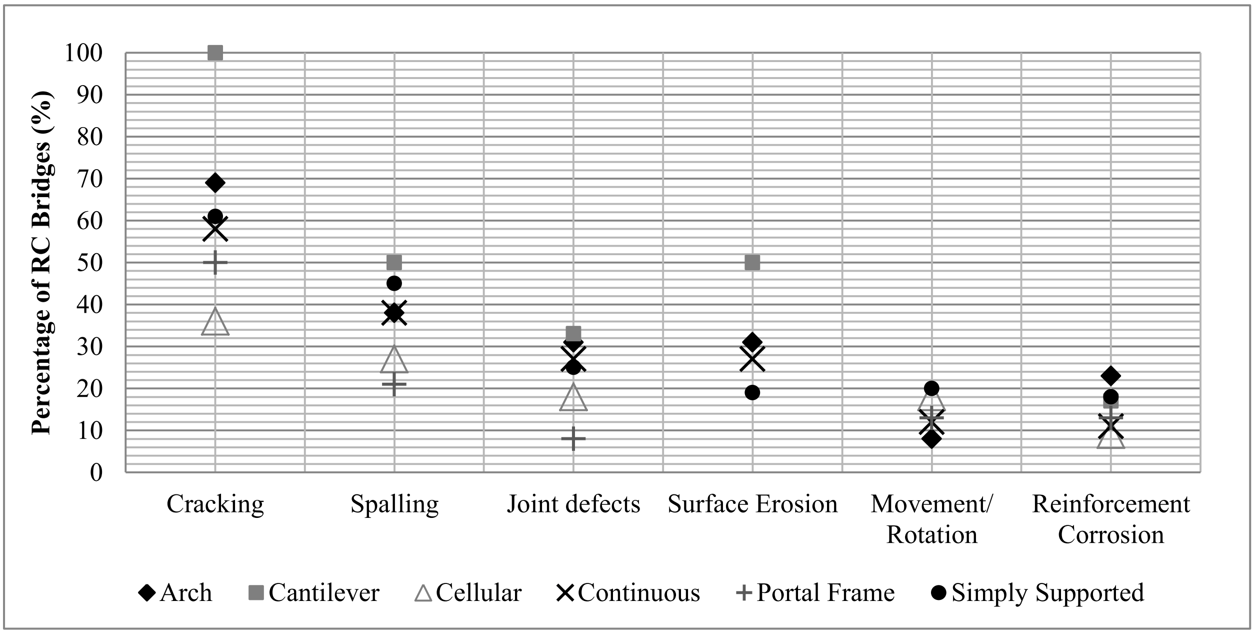

Figure 4-31: Percentage of RC Bridges with Predominant Defects per Structure Type

Table 4-19: Quantity and Percentage of RC Bridges with Defects per Structure Type

Defects	Arch	Cantilever	Cellular	Continuous	Portal Frame	Simply supported
Reinforcement Corrosion	**3 (23 %)**	**1 (17 %)**	**1 (9 %)**	**10 (11 %)**	**3 (13 %)**	**116 (18 %)**
Cracking	**9 (69 %)**	**6 (100 %)**	**4 (36 %)**	**53 (58 %)**	**12 (50 %)**	**389 (61 %)**
Spalling	**5 (38 %)**	**3 (50 %)**	**3 (27 %)**	**35 (38 %)**	**5 (21 %)**	**286 (45 %)**
Defective Concrete	3 (23 %)	0 (0 %)	1 (9 %)	6 (7 %)	2 (8 %)	97 (15 %)
Approach Settlement	1 (8 %)	1 (17 %)	0 (0 %)	6 (7 %)	4 (17 %)	78 (12 %)
Surface Erosion	**4 (31 %)**	**3 (50 %)**	**0 (0 %)**	**25 (27 %)**	**0 (0 %)**	**122 (19 %)**
Bearing Defects	0 (0 %)	1 (17 %)	0 (0 %)	9 (10 %)	0 (0 %)	19 (3 %)
Scour	2 (15 %)	0(0 %)	0 (0 %)	9 (10 %)	1 (4 %)	42 (7 %)
ASR	1 (8 %)	1 (17 %)	0 (0 %)	14 (15 %)	1 (4 %)	70 (11 %)
Impact/ Damage	0(0 %)	0 (0 %)	0 (0 %)	9 (10 %)	2 (8 %)	42 (7 %)
Honey Combing	2 (15 %)	0 (0 %)	1 (9 %)	4 (4 %)	2 (8 %)	59 (9 %)
Embankment Erosion	1(8 %)	0 (0 %)	0 (0 %)	7 (8 %)	0 (0 %)	28 (4 %)
Joint defects	**4 (31 %)**	**2 (33 %)**	**22 (18%)**	**25 (27 %)**	**2 (8 %)**	**159 (25 %)**
Movement/ Rotation	**1 (8 %)**	**0 (0 %)**	**22 (18 %)**	**11 (12 %)**	**3 (13 %)**	**126 (20 %)**
Fire Damage	0 (0 %)	0 (0 %)	0 (0 %)	2 (2 %)	(0 %)	6 (1 %)

About 61 % (389) of the simply supported RC bridges had cracking and 45 % (286) had spalling. Joint defects were common in 20 % of these bridges and the other defects affected below 20 %. Cracking was prevalent in 58 % (53) of the continuous bridges and spalling in 38 % (35). Furthermore, surface erosion and joint defects affected just over a quarter (27 %) of these bridges. Lastly, the other bridge types, constituting only 7 %, were mostly affected by cracking and spalling.

The mean age and the standard deviation of the age for the various structure types were investigated and have been reported in *Table 4-20*.

Table 4-20: Mean Age and Standard Deviation of the Age for the Various RC Bridge Structure Types

	Arch	Cantilever	Cellular	Continuous	Portal Frame	Simply Supported
Mean	61.5 Years	46.2 Years	40.0 Years	42.7 Years	20.2 Years	48.7 Years
Standard Deviation	18.33	10.04	23.60	20.06	23.90	24.35

The average CI's and PI's for the structures were investigated per structure type and the findings represented in *Figure 4-32.* The following observations were made:

- Cantilever RC bridges, which were the least, had the highest average CI which was 90.1, while the arch RC bridges had the lowest average CI which was 76.2. There were only seven cantilever RC bridges and these had a mean age of 46.1 year. Whereas there were 13 arch RC bridges and these had a mean age of 61.5 years. It may be noted that most of the years of the construction of the arch bridges were not known therefore the mean age was determined from data available and may therefore not be a true representation of the mean age of arch RC bridges. However, assuming that it was, this would explain the lower average CI of the arch RC bridges as they were much older.

- The slightly lower CI (81.3) of the simply supported RC bridges compared to the CI of the continuous bridges (82.9) may be attributed to their higher mean age (48.7 years). It may be noted that the simply supported bridges were not only the most dominant RC bridge structure type, they also had the highest mean age compared to the other RC bridge structure types. In addition to this about 60 % of them were located along the costal district municipality's thus a large quantity of them were susceptible to the consequences of the marine environment. *Table 4-21* below, provides a summary of the distribution of the dominant RC bridge structure types; continuous and simply supported RC bridges.

Table 4-21: Distribution of Continuous and Simply supported RC Bridges in the Various District Municipalities

Independent Variable	Continuous	Simply Supported
Cape Winelands	16	172
Central Karoo	7	76
City of Cape Town	18	56
Eden	27	124
Overberg	14	73
West Coast	9	137
Total	91	638

- The properties of the district specific condition indices in *Table 4-22* showed that the CIs had similar deviations from their means and that the data was left skewed and fat-tailed.

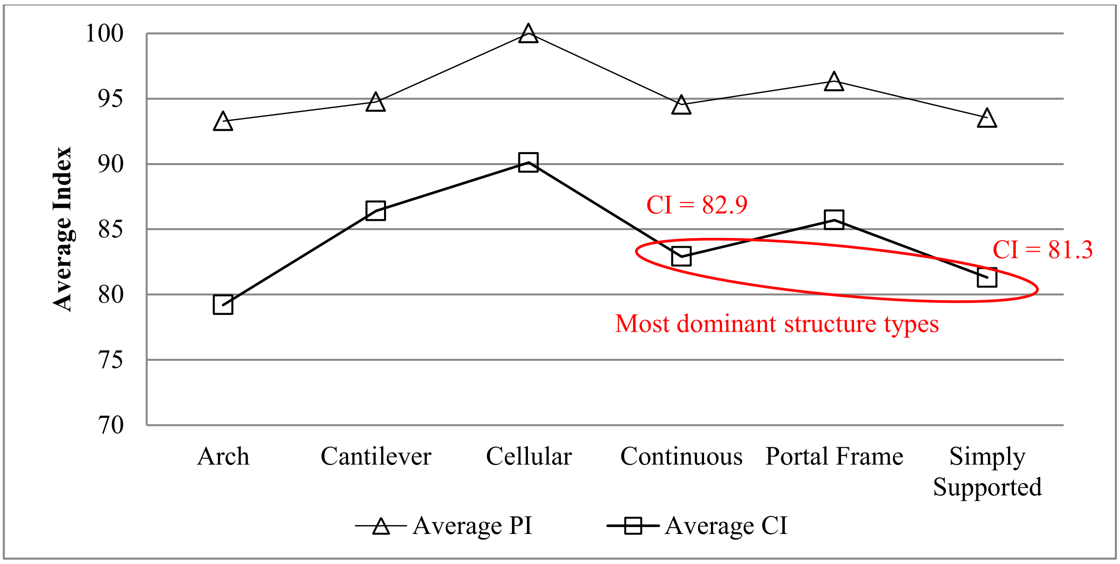

Figure 4-32: Average PI and CI for RC Bridges of Various Structure Types

Table 4-22: Summary of Properties for RC Bridge CIs per Structure Type

	Structure Type					
	Arch	**Cantilever**	**Cellular**	**Continuous**	**Portal Frame**	**Simply Supported**
Observations	12	7	11	90	24	631
Mean	79.2	86.4	90.1	82.9	85.7	81.3
Min	51.8	72.6	76.7	54.2	60.5	50.5
Maximum	98.6	100	100	100	98	100
Standard Deviation	13.39241	10.21199	8.07741	10.55313	10.21546	10.18452
Variance	179.3566	104.2848	65.24455	111.3685	104.3556	103.7245
Skewness	-0.4940339	-0.2066309	-0.538888	-0.4701893	-0.694203	-0.370646
Kurtosis	2.576986	1.629048	2.033921	2.578223	2.63129	2.657202

4.5.2 RC Culverts

The distribution of RC culvert structure types has been provided by *Figure 4.33.* Simply supported cast in-situ portal frame RC culverts constituted 72 % and the precast portal frame RC culverts constituted 18 %. Thus portal frame RC culverts constituted 90 % of the total quantity of RC culverts. The remaining 10 % was that of cellular, concrete pipe and continuous RC culverts.

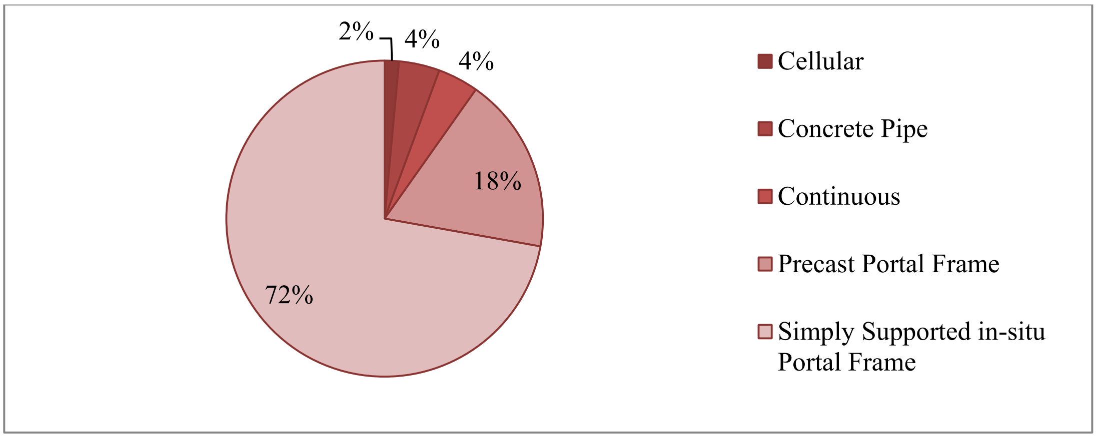

Figure 4-33: Distribution of RC Culvert Structure Types

Table 4-23 contains results obtained from the binary logistic regression analysis to measure the significance of the various structure types on the predominant defects in relation to the base category; cellular RC culverts.

Table 4-23: Results from Logistic Regression Analysis between Defects and Various Structure Types for RC Culverts

Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
Cracking						
Concrete Pipe	.875	.4660804	-0.25	0.802	.308038	2.485489
Continuous	1.788462	.9447631	1.10	0.271	.63508	5.036522
Portal Frame	1.003378	.4761462	0.01	0.994	.3958521	2.543294
Simply Supported	2.162531	.9969617	1.67	0.094	.8760819	5.338018
Spalling						
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
Concrete Pipe	.26	.16083	-2.18	0.029	0.0773463	.873991

Continuous	.5487013	.310116	-1.06	0.288	.1812393	1.661191
Portal Frame	.8571429	.4186139	-0.32	0.752	.3261099	2.232367
Simply Supported	.8094544	.3836015	-0.45	0.656	.3197473	2.04917
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
Defective Concrete						
Concrete Pipe	2.615385	1.799403	1.40	0.162	.679052	10.07321
Continuous	3.063063	2.098106	1.63	0.102	.800034	11.72745
Portal Frame	1.127832	.732137	0.19	0.853	.3159971	4.025368
Simply Supported	1.1425667	.899954	0.56	0.574	.4137101	4.912924
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
Scour						
Concrete Pipe	5.066667	5.452701	1.51	0.132	.6147052	41.76166
Continuous	2.66	2.932629	0.89	0.375	.3065058	23.08472
Portal Frame	2.13964	2.24115	0.73	0.468	.274635	16.66961
Simply Supported	2.462156	2.538085	0.87	0.382	.3264858	18.56807
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
Movement/ Rotation						
Concrete Pipe	.339291	.4882105	-0.75	0.453	.0202193	5.693484
Continuous	1.826923	2.060347	0.53	0.593	.2003373	16.6614
Portal Frame	1.857778	1.950694	0.59	0.555	.2372574	14.54681
Simply Supported	2.837806	2.92392	1.01	0.311	.3766617	21.38031
Independent Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
Reinforcement Corrosion						
Concrete Pipe	112823.1	4.44e+07	0.03	0.976	0	
Continuous	581838.9	2.29e+08	0.03	0.973	0	
Portal Frame	599624.8	2.36e+08	0.03	0.973	0	
Simply Supported	327449.7	1.29e+08	0.03	0.974	0	

The corresponding z-values for the investigation of relationships between RC culvert districts and reinforcement corrosion indicated that there were no relationships between the data. As with the RC bridges, the findings in *Table 4-23* show that there were no statistically significant relationships between defects and RC culvert structure types. The relationships between cracking as well as the dominant structure types (namely, precast portal frame RC culverts and simply supported in-situ portal frame RC culverts) were investigated per district municipality and the findings tabulated in the *Tables 4-24* and *4-25.* It may be noted that the base category was that of the structure type being investigated located in the Cape Winelands DM.

Table 4-24: Results from Logistic Regression Analysis between Cracking and Precast Portal Frame RC Culverts per District Municipality

Independent Variable	Odds Ratio	Std. Err.	Z	P>|z| (p -value)	[95 % Conf. Interval]	
Central Karoo	5.297659	2.190096	4.03	0.000	2.356081	11.91181
City of Cape Town	2.030760	1.615686	0.89	0.373	.4270142	9.657814
Eden	2.378378	.9665085	2.13	0.033	1.072447	5.274559
Overberg	3.008547	1.403757	2.36	0.018	1.205577	7.507903
West Coast	.8058608	.4748903	-0.37	0.714	.253893	2.557817

Table 4-25: Results from Logistic Regression Analysis between Cracking and Simply Supported in-situ Portal Frame RC Culverts per District Municipality

Independent Variable	Odds Ratio	Std. Err.	Z	P>|z| (p -value)	[95 % Conf. Interval]	
Central Karoo	18.95469	5.913089	9.43	0.000	10.28434	34.93468
City of Cape Town	.5203252	.4311717	-0.79	0.430	.1025463	2.640158
Eden	4.147735	.8261147	7.14	0.000	2.807211	6.128398
Overberg	1.878462	.4243881	2.79	0.005	1.206414	2.924884
West Coast	.8672987	.1782105	-0.69	0.488	.579699	1.297313

Precast portal frame and simply supported in-situ portal frame RC culverts located in the Central Karoo DM, the Eden DM and the Overberg DM were statistically significant when investigating cracking. The precast and simply supported in-situ portal frame RC culverts in the Central Karoo DM had the highest odds ratios which were 5.29 and 18.95, respectively. Simply supported in-situ portal frame RC culverts in the Overberg DM were the least likely to have cracking, relative to the base category, when compared to the other significant predictor variables. Subsequently, the distributions of the predominant defects on the various structure types were investigated and the results graphically illustrated in *Figure 4-34.* The quantities of RC culverts of the various structure types were exceptionally varied thus the quantity and percentage of RC culverts with defects per RC culvert structure type were provided in *Table 4-26.*

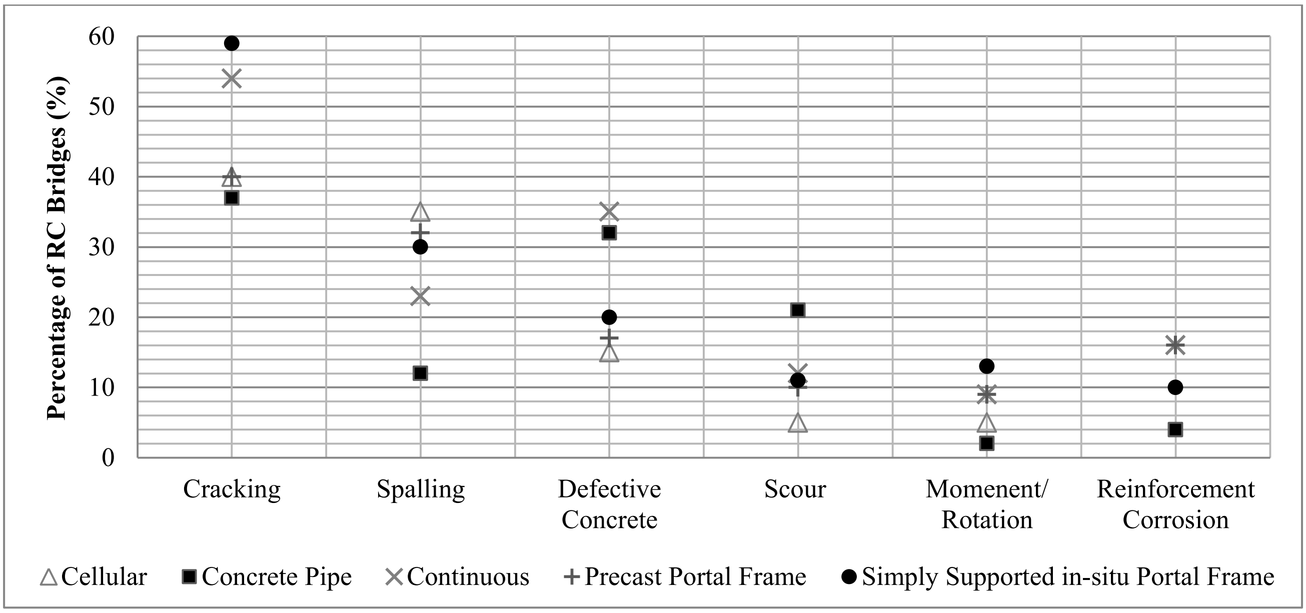

Figure 4-34: Percentage of RC Bridges with Indicated Defects per Structure Type

Table 4-26: Percentage of RC Culverts with Specified Defect per Structure Type

Defects	Cellular	Concrete Pipe	Continuous	Precast Portal Frame	Simply Supported in-situ Portal Frame
Reinforcement Corrosion	**0 (0 %)**	**2 (4 %)**	**9 (16 %)**	**40 (16 %)**	**94 (10 %)**
Cracking	**8 (40 %)**	**21 (37 %)**	**31 (54 %)**	**99 (40 %)**	**580 (59 %)**
Spalling	**7(35 %)**	**7 (12 %)**	**13 (23 %)**	**78 (32 %)**	**299 (30 %)**
Defective Concrete	**3(15 %)**	**18 (32 %)**	**20 (35 %)**	**41 (17 %)**	**198 (20 %)**
Approach Settlement	0 (0 %)	0 (0 %)	3 (5 %)	1 (0 %)	17 (2 %)
Surface Erosion	1 (5 %)	1(2 %)	12 (21 %)	11 (4 %)	83 (8 %)
Bearing Defects	0 (0 %)	0 (0 %)	0 (0 %)	0 (0 %)	4 (0 %)
Scour	**1 (5 %)**	**12 (21 %)**	**7 (12 %)**	**25 (10 %)**	**113 (11 %)**
ASR	2 (10 %)	0 (0 %)	0 (0 %)	4 (2 %)	14 (1 %)
Impact/ Damage	0 (0 %)	6 (11 %)	8 (14 %)	21 (9 %)	71 (7 %)
Honey Combing	0 (0 %)	0 (0 %)	5 (9 %)	11 (4 %)	95 (10 %)
Embankment Erosion	0 (0 %)	2 (4 %)	3 (5 %)	7 (3 %)	32 (3 %)
Joint defects	1 (5 %)	1 (2 %)	5 (9 %)	7 (3 %)	52 (5 %)
Movement/ Rotation	**1 (5 %)**	**1 (2 %)**	**5 (9 %)**	**22 (9 %)**	**128 (13 %)**

Approximately 59 % of the simply supported RC culverts had cracking and 30 % had spalling. Defective concrete was common in 20 % of these bridges and the other defects affected below 15 %. Cracking was prevalent in 40 % of the portal frame RC culverts and spalling in 32 %. The other culvert types, constituting only 10 %, were mostly affected by cracking and spalling and had defective concrete.

The mean age and the standard deviation of the age for the various structure types were investigated and have been reported in *Table 4-27*.

Table 4-27: Mean Age and Standard Deviation of the Age for the Various RC Culvert Structure Types

	Cellular	Concrete Pipe	Continuous	Precast Portal Frame	Simply Supported in-situ Portal Frame
Mean	18.8 Years	18.2 Years	60.0 Years	20.8 Years	50.5 Years
Standard Deviation	26.01	18.19	18.11	13.73	15.48

Furthermore, the average CI's and PI's for the structures were investigated per structure type and the findings represented in *Figure 4-35.* The following observations were made:

- Cellular RC culverts had the highest average CI which was 89.2. The high CI may be associated with their low mean age of 18.8 years; these RC culverts structure types were younger than most of the other RC culvert structure types.

- The precast portal frame RC culverts also had a relatively high average CI of 87.7 and a low mean age (20.8 years) compared to the other structure types. On the other hand, the simply supported cast in-situ portal frame RC culverts had the lowest average CI (83.7). These bridges also had the highest mean age (50.5 years) with a relatively low spread. There is generally more care and quality control taken with the construction of precast structures as when compared to structures cast in-situ. Therefore the lower CI of the simply supported cast in-situ portal frame culverts may not only be associated with their high mean age but also the inadequate construction practices and techniques associated with casting in-situ (discussed earlier). Also, over half of both the precast (52.8 %) and the simply supported in-situ portal frames (60.0 %) RC culverts where located inland thus the adverse consequences of the marine environment were expected to be less of a contributor to their deterioration. *Table 4-28* provides an indication of the distribution of both precast and simply supported cast in-situ RC culverts across the various district municipalities.

Table 4-28: Distribution of Precast and Simply supported in-situ Portal Frame RC Culverts in the Various District Municipalities

Independent Variable	Precast Portal Frame	Simply Supported in-situ Portal Frame
Cape Winelands	57	211
Central Karoo	59	185
City of Cape Town	8	8
Eden	62	256
Overberg	34	132
West Coast	26	197
Total	246	989

- The properties of the average CIs of the various structure types tabulated in *Table 4-29* indicated that the district specific condition indices had similar deviations from their means and the data was left skewed and fat-tailed.

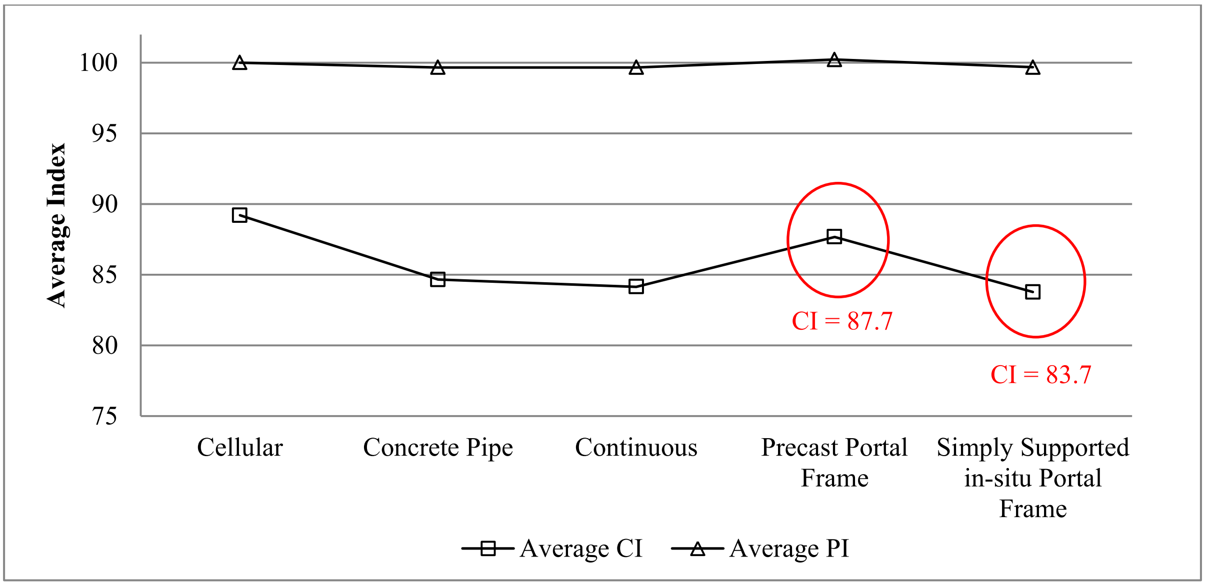

Figure 4-35: Average PI and CI for RC Culverts of Various Structure Types

Table 4-29: Summary of Properties for RC Culvert CIs per Structure Type

	Structure Type				
	Cellular	**Concrete Pipe**	**Continuous**	**Precast Portal Frame**	**Simply Supported in-situ Portal Frame**
Observations	20	55	57	243	979
Mean	89.2	84.6	84.2	87.7	83.7
Min	63.5	39.8	58.4	46.7	42.7
Maximum	100	100	100	100	100
Standard Deviation	11.11959	12.47817	9.310365	11.08355	10.64727
Variance	123.6453	155.7047	86.6829	122.845	113.3644
Skewness	-0.8503185	-1.119874	-0.6460737	-0.9546923	-0.629661
Kurtosis	2.660569	4.77997	3.141485	3.1351134	3.084511

4.6 Investigation of Relationships between Defects and Age

4.6.1 RC Bridges

A total of 514 (65 %) RC bridges had known construction dates hence their ages could be determined. The distribution of RC bridge ages is illustrated in the *Figure 4-36.* It was found that 51.5 % of these bridges were less than 50 years of age, of which almost one fifth (19.6 %) range between 40 – 49 years.

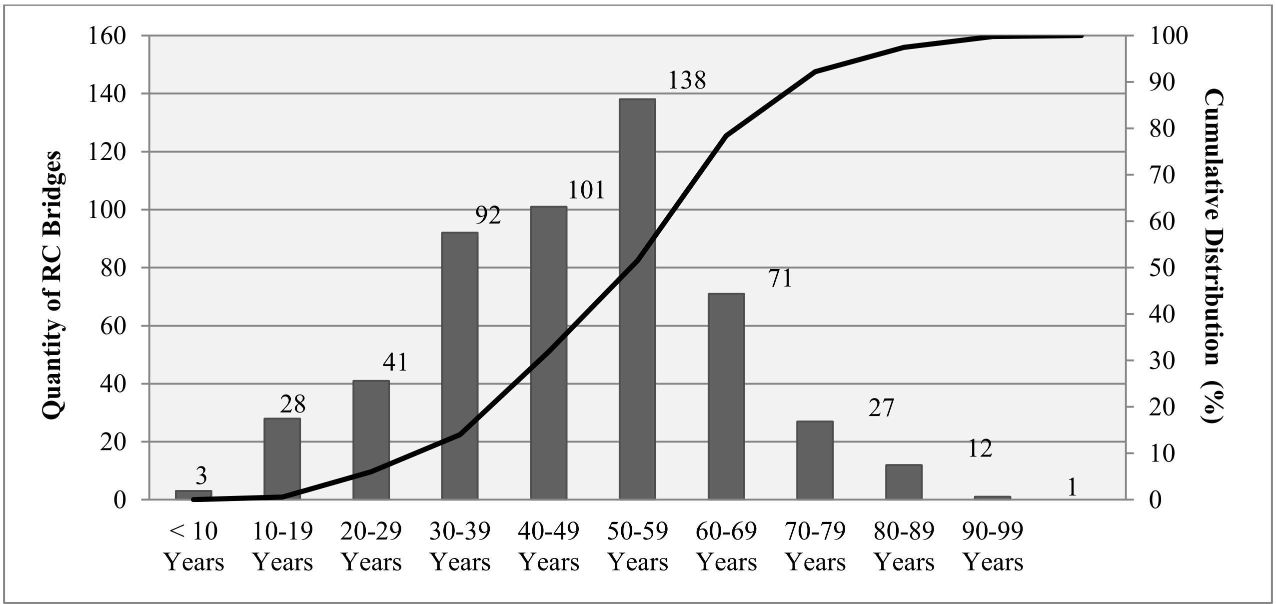

Figure 4-36: Distribution of RC Bridges by Age

The average RC bridge age was calculated to be 42.8 years. *Table 4-30* provides summary statistics for the RC bridge ages.

Table 4-30: Summary Statistics for RC Bridge Ages

Summary Statistics	
Mean	42.8 years
Min	5 years
Max	99 years
Standard deviation	16.506
Variance	272.4479
Skewness	-0.980537
Kurtosis	2.771726

Table 4-31 presents results obtained from the binary logistic regression analysis to measure the significance of age on the predominant defects on RC bridges.

Table 4-31: Results from Logistic Regression Analysis between Defects and Age for RC Bridges

Dependant Variable	Odds Ratio	Std. Err.	Z	P>|z| (p -value)	[95 % Conf. Interval]	
Cracking	1.006504	.0021886	2.98	0.003	1.002224	1.01.803
Spalling	1.000638	.0021009	0.30	0.761	.9965291	1.004764
Joint Defects	.9989302	.0023973	-0.45	0.656	.9942455	.5251843
Surface Erosion	1.0029	.0025935	1.12	0.263	.9978301	1.007996
Movement/ Rotation	.9980825	.0026834	-0.71	0.475	.992837	1.003356
Reinforcement Corrosion	.9965077	.0027598	-1.26	0.207	.1799083	.4095258

The continuous predictor variable (age) was statistically significant only when investigating cracking as the corresponding p-value < 0.05. Thus, a unit increase in age was likely to increase the odds of a RC bridge having cracking or rather crack initiation by 1.006. The continuous variable, age, was categorised as seen in *Table 4-32* and the categorical variables investigated for cracking in relation to the base category 0 to 9 years.

Table 4-32: Results from Logistic Regression Analysis between Cracking and Age Categories for RC Bridges

Categorical Variable	Odds Ratio	Std. Err.	Z	P>|z| (p -value)	[95 % Conf. Interval]	
10 – 19 years	1.5	1.924351	0.32	0.752	.1213625	18.5395
20 – 29 years	1.153846	1.46185	0.11	0.910	.0963229	13.82185
30 – 39 years	1.5	1.864393	0.33	0.744	.1312528	17.1425
40 – 49 years	6.608691	8.245348	1.51	0.130	.5729436	76.22877
50 – 59 years	4.923077	6.101553	1.29	0.193	.4337867	55.87236
60 – 69 years	2.375	2.964059	0.69	0.488	.2057515	27.41474
70 – 79 years	2.9099091	3.740653	0.83	0.406	.2340131	36.16383
80 – 89 years	1.428571	1.939312	0.26	0.793	.0998573	20.43732
90 -100 years	1	(omitted)				

Although age was found to be statistically significant with regards to cracking. The results in *Table 4-32* indicate that the categorical variables were not statistically significant relative to the base category (p-value > 0.05).

Figure 4-37 shows that quantity of RC bridges with the predominant defects of cracking, spalling, reinforcement corrosion and movements/rotations generally increased with increasing age categories. Thus implying a general increase the percentage of RC bridges with deterioration with increasing age; this was to be expected.

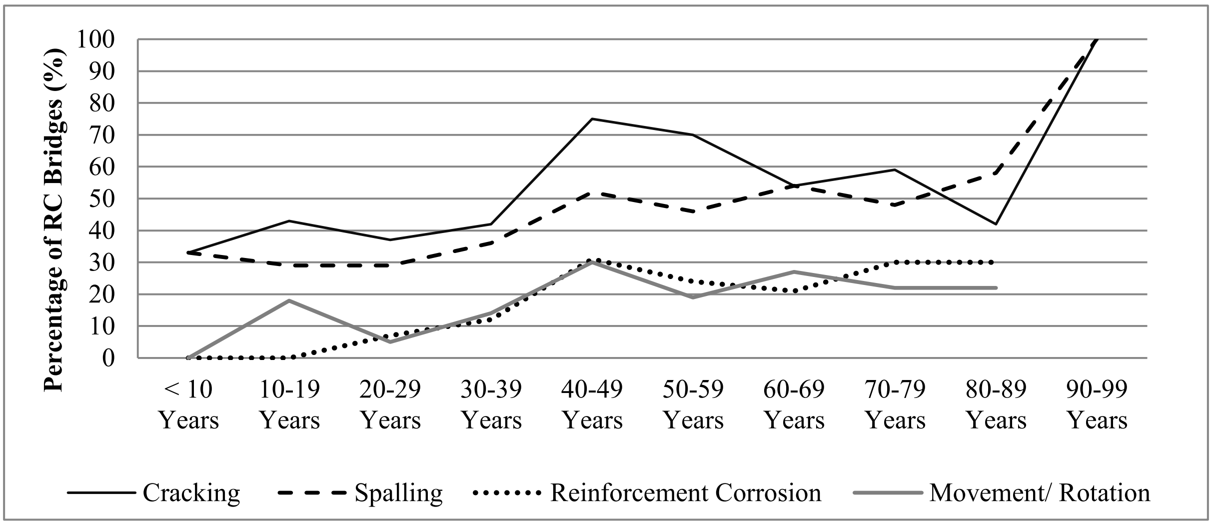

Figure 4-37: Relationship between Age and Cracking; Spalling, Reinforcement Corrosion and Movement/ Rotation for RC Bridges

However, the quantity of RC bridges with the predominant defects of joint defects and surface erosion generally decreased with increasing age categories (see *Figure 4-38*).

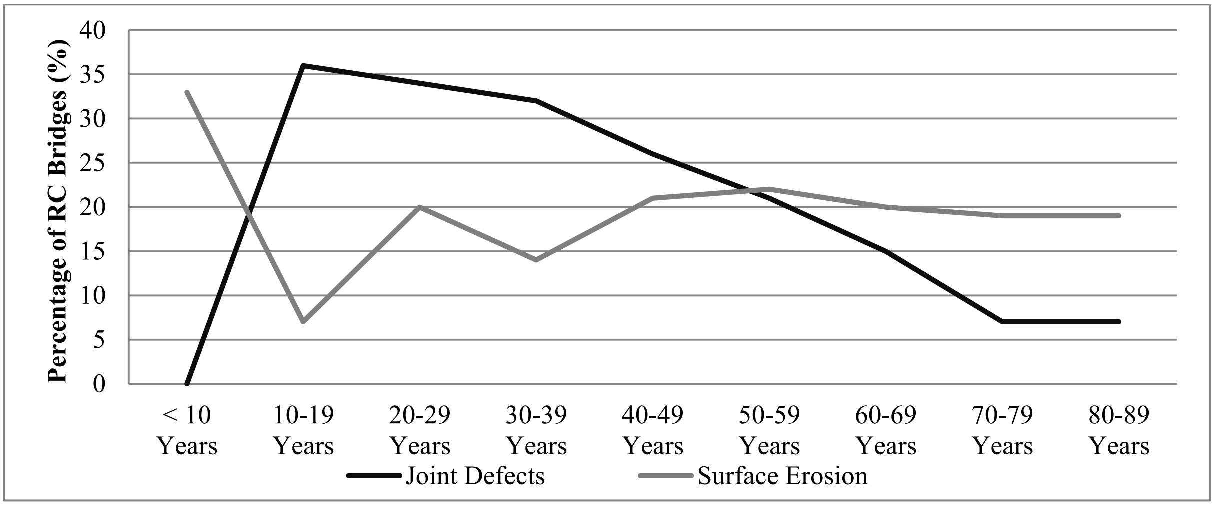

Figure 4-38: Relationship between Age and Joint Defects and Surface Erosion for RC Bridges

The increase in RC bridges with joint defects (with decreasing age categories) was assumed a consequence of the evolution of expansion joint assembly's. These were presumed to have become less durable with modification over time hence their decreasing performance levels. Though there was a decrease in the percentage of RC bridges with surface erosion (with increasing age categories), there were an increase in RC bridges (with age categories) with cracking, spalling etc. suggesting that most of the surface erosion could have possibly formed part of the spalling. Thus it would be inaccurate to assume that the quality of concrete had improved or worsened over the years. In addition to this the average condition of the RC bridges in each age category was investigated and the average CIs and their corresponding PIs provided in *Figure 4-39*.

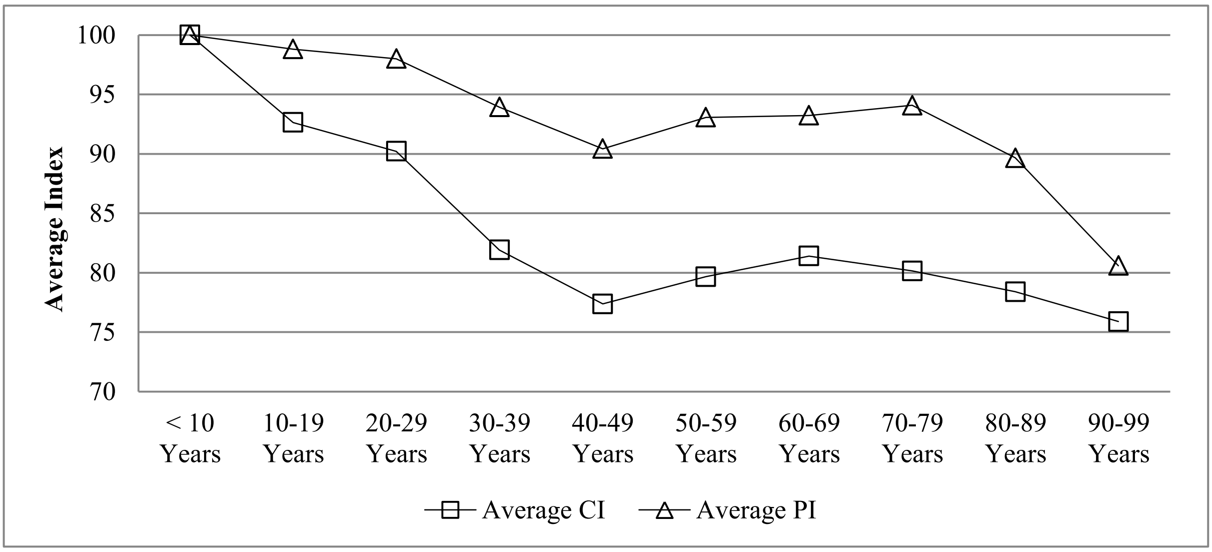

Figure 4-39: Average PI and CI for RC Bridges of Various Age Groups

It may be seen that the average PI graph mimicked the shape of average CI graph. Also, the average CIs and corresponding PIs for the increasing age groups showed a general decrease with increasing age. The overall average CI and PI were, 83.8 and 93.2, respectively. The average CI for the age category of 40 to 49 years, which contained the highest quantity, of the RC bridges, was 77.4. Furthermore, all the RC bridge CIs were greater than 70 implying that they were generally in good condition.

Figure 4-40 derived from *Figure 4-36* and *4-39,* highlights observations made with respect to the average CIs for the increasing age categories and is further discussed on the next page.

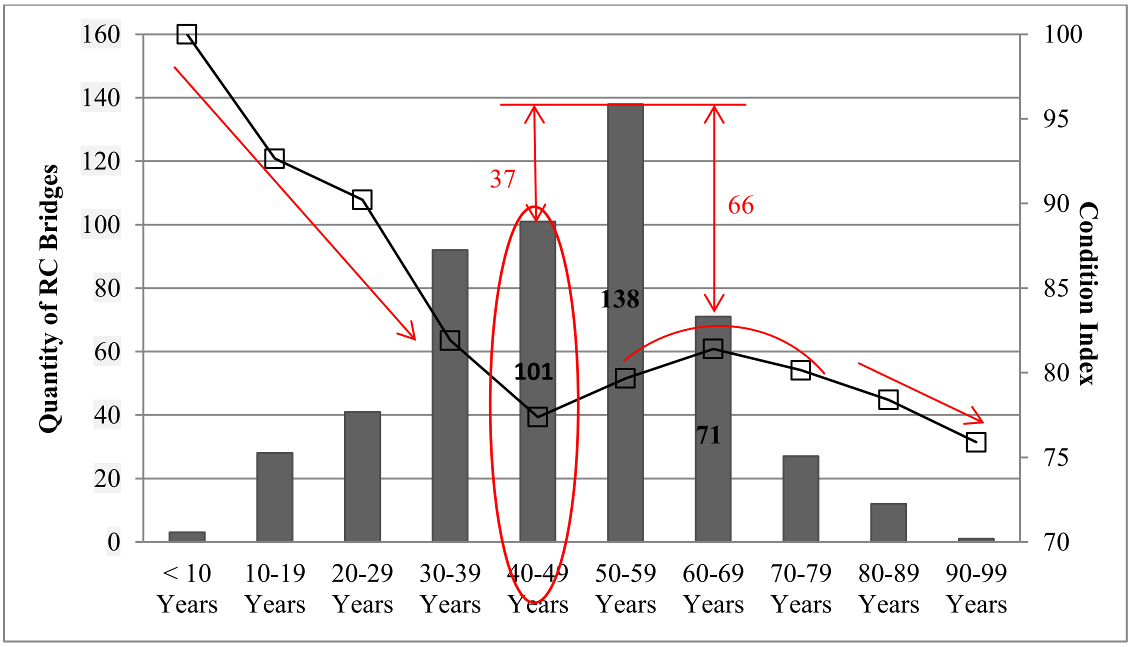

Figure 4-40: Quantity and Average Condition Indices of RC Bridges as a Function of Age

A clear decrease in average CI with increasing age categories was observed for RC bridges less than 50 years of age. This was to be expected as deterioration was generally expected to increase with increasing age. The RC bridges between ages 40 to 49 years had an average CI of 77.4, one of the lowest average CIs relative to the other age categories. These RC bridges constituted mostly of simply supported RC bridges (77 %) and were distributed mostly inland (see *Table 4-33*).

Table 4-33: Distribution of RC Bridges of Age Category 40 – 49 years

District Municipality	Quantity
Cape Winelands	30
Central Karoo	24
City of Cape Town	2
Eden	23
Overberg	5
West Coast	17
Total	**101**

The average CI values for RC bridges of age groups 50 to 59 years and 60 to 69 years were 79.7 and 81.4 correspondingly. These showed an increase with increasing age categories as opposed to a decrease. Most of these were also simply supported RC bridges and were distributed mostly along the coast (see *Table 4-34*). The peaking of the average CI graph at age category 60 to 69 may be associated with several factors such as:

1. The design and use of a more durable concrete mix as well as adequate construction practices and techniques during that period.

2. Almost half (46.4 %) of the RC bridges being distributed inland as opposed to the costal district municipalities.

3. The lesser quantity of RC bridges constructed (71) compared to the consecutive age categories of 50 to 59 years and 40 to 49 years. Construction of RC bridges increased by approximately 48 % when comparing the period of 1945 to 1954 and the period from 1955 to 1964. It is assumed that the increase in construction could have compromised the quality of the RC bridges produced.

Table 4-34: Distribution of RC Bridges of Age Category 40 – 49 years

District Municipality	Quantity (50 – 59 years)	Quantity (60 – 69 years)
Cape Winelands	24	30
Central Karoo	20	3
City of Cape Town	4	0
Eden	36	9
Overberg	20	12
West Coast	34	17
Total	**138**	**71**

The average CI of the bridges constructed during the period from 1915 to 1944 show a gradual decrease with increasing age categories. The age category of 90 to 99 years had the lowest average CI of 75.9.

4.6.2 RC Culverts

A total of 266 (19 %) of RC culverts had known construction dates hence their ages could be determined. The distribution of RC bridge ages is illustrated in the histogram in *Figure 4-41.* It was found that 63 % of these bridges were less than 50 years of age, of which just over a fifth (22%) of these range between 40 – 49 years.

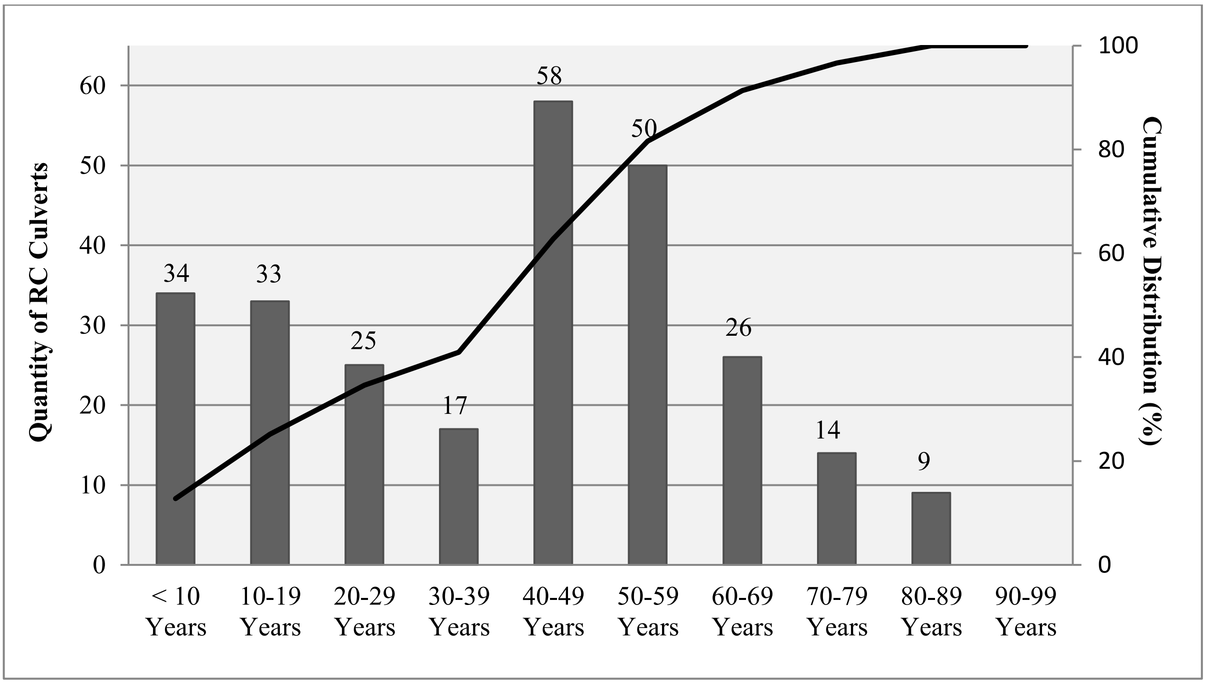

Figure 4-41: Distribution of RC Culverts by Age

The average RC bridge age was also calculated to be 42.8 years. *Table 4-35* provides summary statistics for the RC bridge ages.

Table 4-35: Summary Statistics for RC Culvert Ages

Summary Statistics	
Mean	42.8 years
Min	4 years
Max	89 years
Standard deviation	19.03693
Variance	362.4048
Skewness	-0.299355
Kurtosis	2.313018

Table 4-36 presents results obtained from the binary logistic regression analysis to measure the significance of age on the observed predominant defects on RC culverts.

Table 4-36: Results from Logistic Regression Analysis between Defects and Age for RC Culverts

Dependant Variable	Odds Ratio	Std. Err.	Z	P>|z| (p -value)	[95 % Conf. Interval]	
Cracking	1.007968	0.0017794	4.50	0.000	1.004486	1.011462
Spalling	1.0051010	0.0020516	2.49	0.013	1.001088	1.00913
Defective Concrete	1.002308	.0022386	1.03	0.302	.9979299	1.006705
Scour	1.007791	.0033164	2.36	0.018	1.001312	1.014312
Movement/ Rotation	1.001156	.0027805	0.42	0.677	.9957211	1.006621
Reinforcement Corrosion	-.00393	.0025727	-1.53	0.127	-.0089723	.0011123

The findings show that age was statistically significant for cracking, spalling and scour. Thus a unit increase in RC culvert age correspondingly increased the odds of these defects being present on the RC culvert by about 1.007, 1.005 and 1.008.

Table 4-37 presents results obtained from the binary logistic regression analysis to measure the significance of the categorical age groups on cracking, spalling and scour in relation to the base category; an RC culver between ages 0 to 9 years with the same defect.

Table 4-37: Results from Logistic Regression Analysis between Cracking, Spalling and Scour and Age Categories for RC Culverts

Categorical Variable	Odds Ratio	Std. Err.	Z	P>|z| (p -value)	[95 % Conf. Interval]	
Cracking						
10 – 19 years	2.206863	1.993185	0.88	0.381	.3758298	12.95865
20 – 29 years	8.999865	7.55589	2.62	0.009	1.73622	46.65168
30 – 39 years	22.8568	20.11009	3.56	0.000	4.07477	128.2117
40 – 49 years	30.39954	23.69525	4.38	0.000	6.597646	140.07
50 – 59 years	37.33277	29.5491	4.57	0.000	7.913334	176.125
60 – 69 years	15.99976	13.24302	3.35	0.001	3.15917	81.03151
70 – 79 years	53.33253	52.3794	4.05	0.000	7.780419	365.5792
80 – 89 years	12.6792	12.6792	3.35	0.010	1.836634	89.20397
90 -100 years	20.33924	14.87607	4.05	0.000	4.850295	85.29061

Categorical Variable	Odds Ratio	Std. Err.	Z	P>|z| (p -value)	[95 % Conf. Interval]	
			Spalling			
10 – 19 years	2.129032	2.661342	0.60	0.545	.1837213	24.67203
20 – 29 years	8.25	9.334914	1.86	0.062	.8980807	75.78662
30 – 39 years	18	20.42725	2.55	0.011	1.946653	166.4395
40 – 49 years	23.29412	24.44641	3.00	0.003	2.978092	182.2025
50 – 59 years	15.52941	16.45103	2.59	0.010	1.947322	123.8432
60 – 69 years	9.9	11.05511	2.05	0.040	1.109474	88.33915
70 – 79 years	28.28571	32.74097	2.89	0.004	2.926131	273.4264
80 – 89 years	26.4	32.12027	2.69	0.007	2.432017	286.577
90 -100 years	14.36471	14.6112	2,62	0.009	1.96542	105.464
Categorical Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
			Scour			
10 – 19 years	1080938	7.59e+08	0.02	0.984	0	
20 – 29 years	1456826	1.02e+08	0.02	0.984	0	
30 – 39 years	2234039	1.57e+08	0.02	0.983	0	
40 – 49 years	598481.6	4.20e+08	0.02	0.985	0	
50 – 59 years	3191987	2.24e+08	0.02	0.983	0	
60 – 69 years	3046840	2.14e+08	0.02	0.983	0	
70 – 79 years	1396129	9.80e+08	0.02	0.984	0	
80 – 89 years	2094353	1.47e+08	0.02	0.983	0	
90 -100 years	2388704	1.68e+08	0.02	0.983	0	

All age categories except 10 – 19 years were statistically significant for cracking and spalling. The highest odds ratio was that of age category 70 – 79 years. Thus a bridge of this category was more likely to have cracking or spalling, with reference to the base category, as opposed to that of any other age category. Its odds ratio suggested that it was were 53.33 times more likely to have cracking and 28.28 times more likely to have spalling than the base category.

The confidence intervals of all the age categories for Scour were all zero, indicating that the z –values were not significant and that there were no relationships between the age categories and scour.

The predominant defects investigated in *Figure 4-42* all suggested that there was a general increase in the percentage of RC culverts with cracking, spalling, defective concrete, scour, observed movement and rotations as well as reinforcement corrosion with increasing age categories.

More so, it can be noted that a high percentage of RC culverts between 0 to 59 years had the predominant defects. However, a lower percentage of RC culverts in age category of 60 to 69 years had the predominant defects. The decrease in the percentage of RC culverts with defects may be attributed to previous maintenance activities and/or the quality of the materials that was used for RC culvert construction during the period from 1945 – 1954 as well as several other reasons.

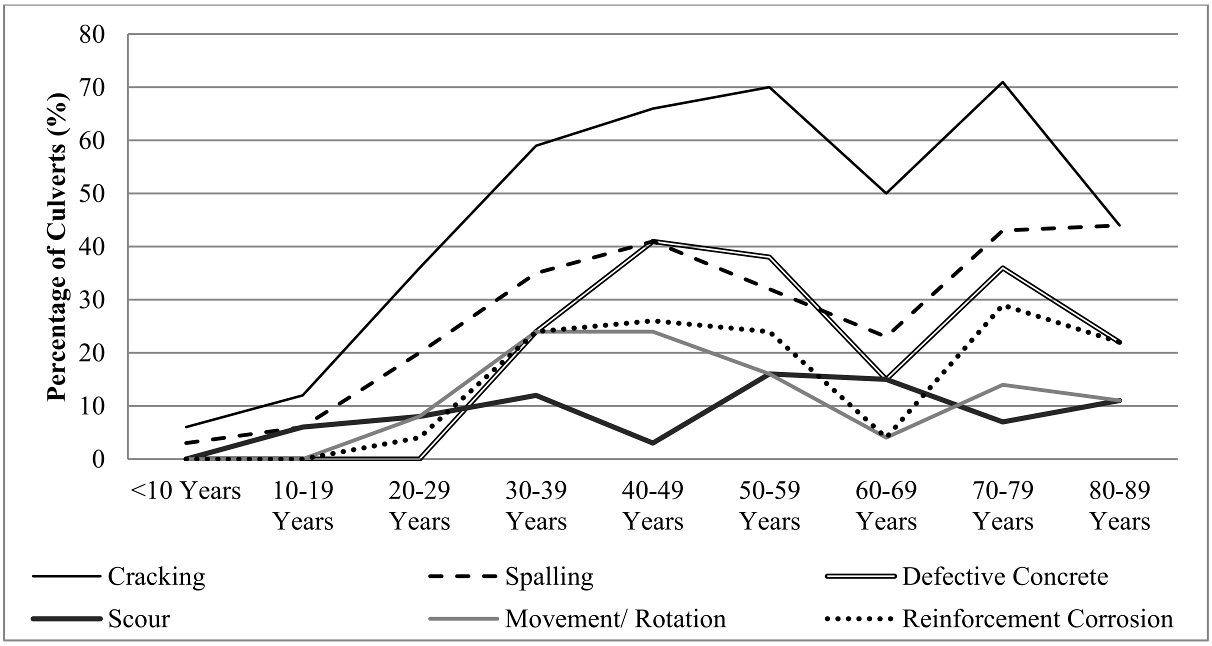

Figure 4-42: Relationship between Age and Cracking; Spallling; Defective Concrete; Scour; Movement/ Rotation and Reinforcement Corrosion for RC Culverts

The average CIs and corresponding PIs for the increasing age categories showed a general decrease with increasing age categories, as seen in *Figure 4-43.*

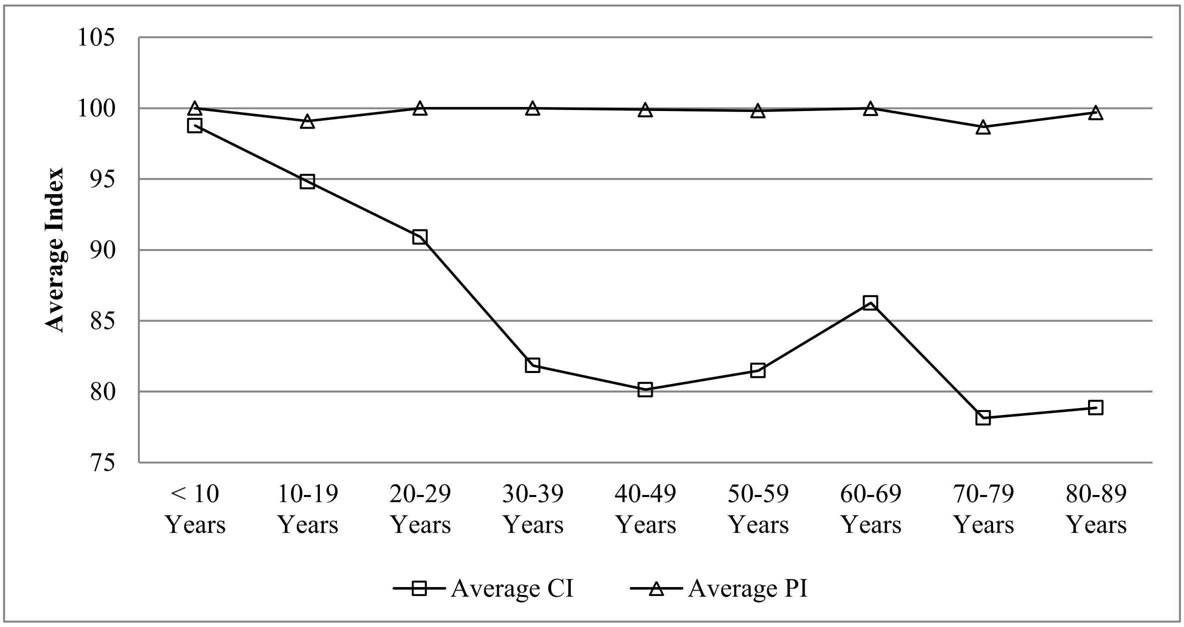

Figure 4-43: Average PI and CI for RC Culverts of Various Age Groups

It may also be noted that as opposed to the PIs of the RC bridges which increased or decreased with increasing or decreasing CIs, respectively, the PIs of the culverts showed no trend as they were generally constant and higher than those of RC culverts. These findings were consistent with the literature which stated that BMSs generally regard the condition of bridges higher than that of culverts. The average CI and PI were 84.0 and 99.7, respectively. All the average CIs were greater than 70 implying that the RC culverts were generally in good condition.

Figure 4-44 derived from *Figure 4-41* and *4-43,* highlights observations made with respect to the average CIs for the increasing age categories and is further discussed on the next page.

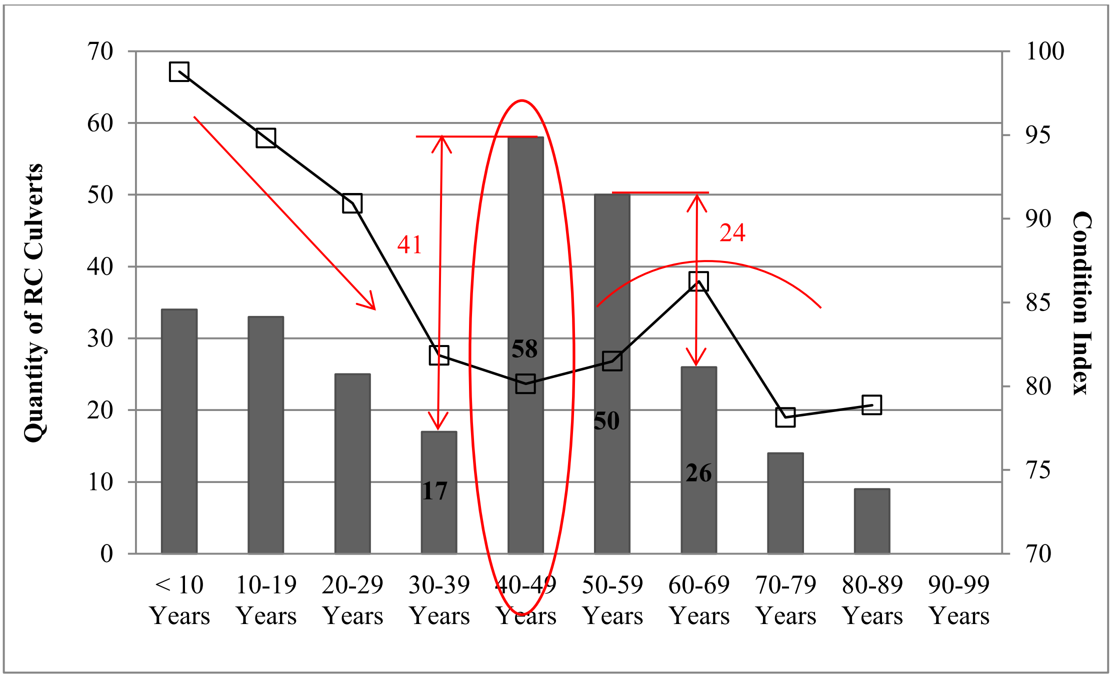

Figure 4-44: Quantity and Average Condition Indices of RC Culverts as a Function of Age

The average CIs for the RC culverts less than 50 years of age show a steady decrease with increasing age categories. This suggests a decrease in average condition with increasing age which was to be expected as the quantity of RC culverts with predominant defects generally increased with increasing age categories (*see Figure 4-42*).

The age category of 40 to 49 years had an average CI of 77.4 and may be seen as the turning point of the average CI graph. There was a 70 % decrease in quantity of RC culverts constructed when comparing the age categories of 40 to 49 years and those of 30 to 39 years. The decrease in construction was considered to have contributed to the increase in the average CI as more attention was given to fewer projects. The distribution of the RC culverts of age category 40 to 49 years is provided in *Table 4.38*.

Table 4-38: Distribution of RC Culverts of Age Category 40 – 49 years

District Municipality	Quantity (50 to 59 years)
Cape Winelands	19
Central Karoo	25
City of Cape Town	0
Eden	2
Overberg	5

West Coast	7
Total	**58**

The average CI values for RC culverts of age groups 50 to 59 years and 60 to 69 years were 79.7 and 81.4, correspondingly. These showed an increase with increasing age categories as opposed to a decrease. The average CI graph had a peak at the age category of 60 to 69 years. Most of the culverts in this age category were simply supported cast in-situ portal frame RC culverts and were distributed as seen in *Table 4-39*.

Table 4-39: Distribution of RC Culverts of Age Category 50 – 59 years and 60 – 69 years

District Municipality	Quantity (50 to 59 years)	Quantity (60 to 69 years)
Cape Winelands	11	10
Central Karoo	20	4
City of Cape Town	0	0
Eden	6	4
Overberg	6	4
West Coast	7	4
Total	**50**	**26**

The peaking of the average CI graph at age category 60 to 69 may be associated with several factors such as:

1. The design and use of a more durable concrete mix as well as adequate construction practices and techniques during that period.

2. The lesser quantity of RC culverts constructed (26) compared to the consecutive age categories. Construction of RC bridges increased by approximately 48 % when comparing the period of 1945 to 1954 and the period from 1955 to 1964. It is assumed that the rapid increase in construction could have compromised the quality of the RC bridges produced.

The age category of 70 to 79 years and 80 to 89 years had the lowest average CIs of 78.9 and 78.1, respectively. The bridges were the oldest and a lower average CI was therefore to be expected.

4.7 Investigation of Relationships between Predominant Defects and Span Length/Width

4.7.1 RC Bridges

A total of 777 (98 %) of RC bridges span lengths were investigated. The distribution of RC bridge span length is illustrated in the histogram in *Figure 4-45*.

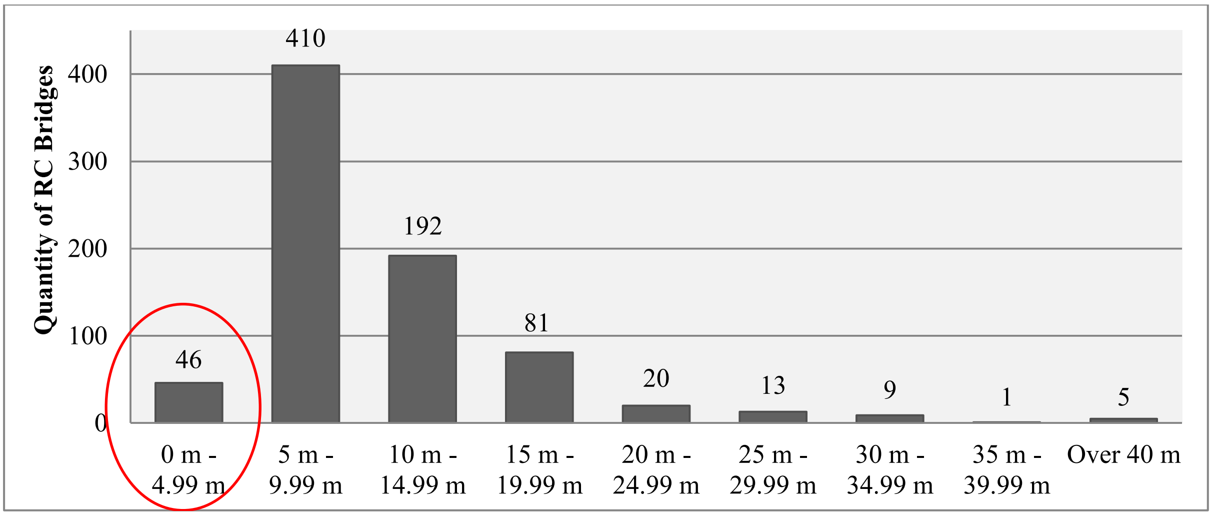

Figure 4-45: Distribution of RC Bridges by Span Length

It was found that 53 % of these bridges had span lengths between 5 m and 10 m. Only about 16 % of the span lengths were over 20 m. The average RC bridge span length was calculated to be 8.26 m. Furthermore, it may be noted that 46 of the bridges span lengths were indicated to be less than 5 m, however, these bridges satisfied the bridge criterion discussed in *section 2.2.1* as they had at least one span greater than 1.5 m and they were longer than 20 m. *Table 4-40* provides summary statistics for the RC bridge span lengths.

Table 4-40: Summary Statistics for RC Bridge Span Lengths

Summary Statistics	
Mean	8.26 m
Min	0.49 m
Max	45.7 m
Standard deviation	6.024224
Variance	36.29128
Skewness	2.069241

Kurtosis	9.240333

Table 4-41 presents results obtained from the binary logistic regression analysis to measure the significance of span length on the predominant defects on RC bridges.

Table 4-41: Results from Logistic Regression Analysis between Defects and Span Length for RC Bridges

Dependant Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
Cracking	1.011001	.0121042	0.91	0.361	.9875534	1.035005
Spalling	1.01555	.0115688	1.35	0.176	.5011644	.8780978
Joint Defects	1.032231	.0127945	2.56	0.010	1.007456	1.057614
Surface Erosion	1.011955	.0129511	0.93	0.353	.9868868	1.037659
Movement/ Rotation	1.012182	0.132263	0.93	0.354	.9865877	1.03844
Reinforcement Corrosion	1.008356	0.136904	0.61	0.540	.1395155	.2790519

The predictor variable (span) was found significant only when investigating joint defects (p-value < 0.05). Thus, a unit increase in span length was likely to increase the odds of an RC bridge having joint defects 1.032231. The continuous variable, span, was categorised as seen in *Table 4-42* and the categorical variables investigated for cracking in relation to the base category a RC bridge of span length 0 to 4.99 m.

Table 4-42: Results from Logistic Regression Analysis between Joint Defects and Span Groups for RC Bridges

Dependant Variable	**Odds Ratio**	**Std. Err.**	**Z**	**P>|z| (p -value)**	**[95 % Conf. Interval]**	
5 m – 9.99 m	2.789062	1.498584	1.91	0.056	.9729743	7.994938
10 m – 14.99 m	5.647059	3.081856	3.17	0.002	1.937685	16.45741
15 m – 19.99 m	4.052632	2.351951	2.41	0.016	1.299394	12.6396
20 m – 24.99 m	9.45	6.580625	3.23	0.001	2.413721	36.99786
25 m – 29.99 m	1.909091	1.775268	0.70	0.487	.3085263	11.81302
30 m – 34.99 m	13.125	11.16636	3.03	0.002	2.476981	63.54662
35 m – 39.99 m	1	(omitted)				
Over 40 m	2.625	3.24037	0.78	0.434	.2335518	29.50362

The results in *Table 4-42* indicate that a RC bridge with a span length between 10 and 15 m was more likely to have joint defects than a bridge with a span length between 15 and 25 m or 30 and 35 m, relative to the base category. A RC bridge of span length between 10 m – 15m was about 5.65 times more likely to have joint defects also relative to the base category.

The predominant defects were investigated for span length 0 to 30 m as only 15 (2 %) of the RC bridges were over 30 m in span length, thus the sample was considered very small and therefore omitted. The findings in *Figure 4-46* all showed a general increase in the percentage of RC bridges with defects with increasing span length. Thus suggesting an increase in the quantity of RC bridges with cracking, spalling, joint defects, surface erosion, observed movement/rotations as well as reinforcement corrosion could be assumed with increasing span (between 0 m – 30 m).

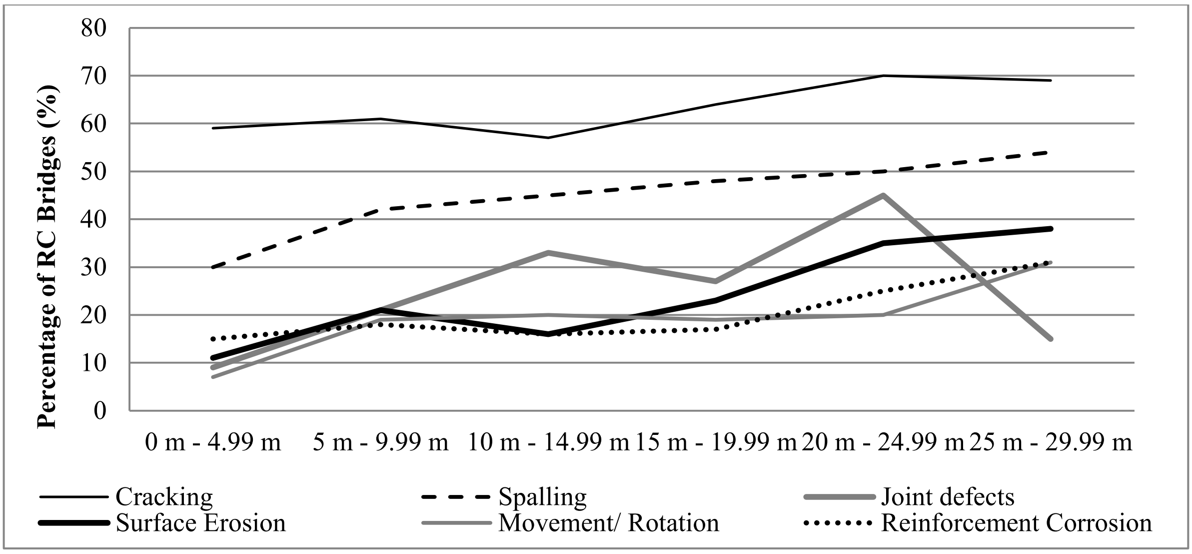

Figure 4-46: Relationship between Age and Cracking; Spalling, Joint Defects; Surface Erosion Movement/Rotation and Reinforcement Corrosion for RC Bridges between 0 m and 30 m

The average CIs and corresponding PIs for the increasing span groups showed a decrease for span lengths between 0 m and 30 m, as seen in *Figure 4-47.*

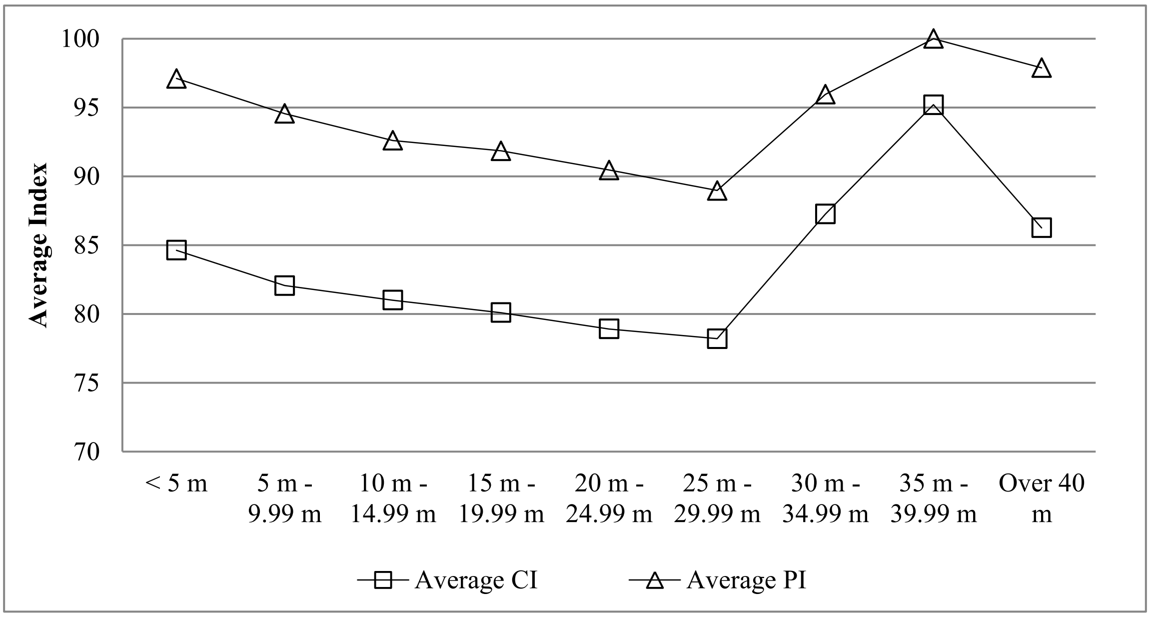

Figure 4-47: Average PI and CI for RC Bridges a Span Length Groups

It is interesting to note that for span lengths greater than 30 m the CIs showed an increase with increasing span length. This was unexpected and assumed a consequence of the lack of data (i.e. only 15 RC bridges which constituted 2 % of the total RC bridges) used to investigate relationships.

Figure 4-48, highlights observations made with respect to the average CIs for the increasing span length categories

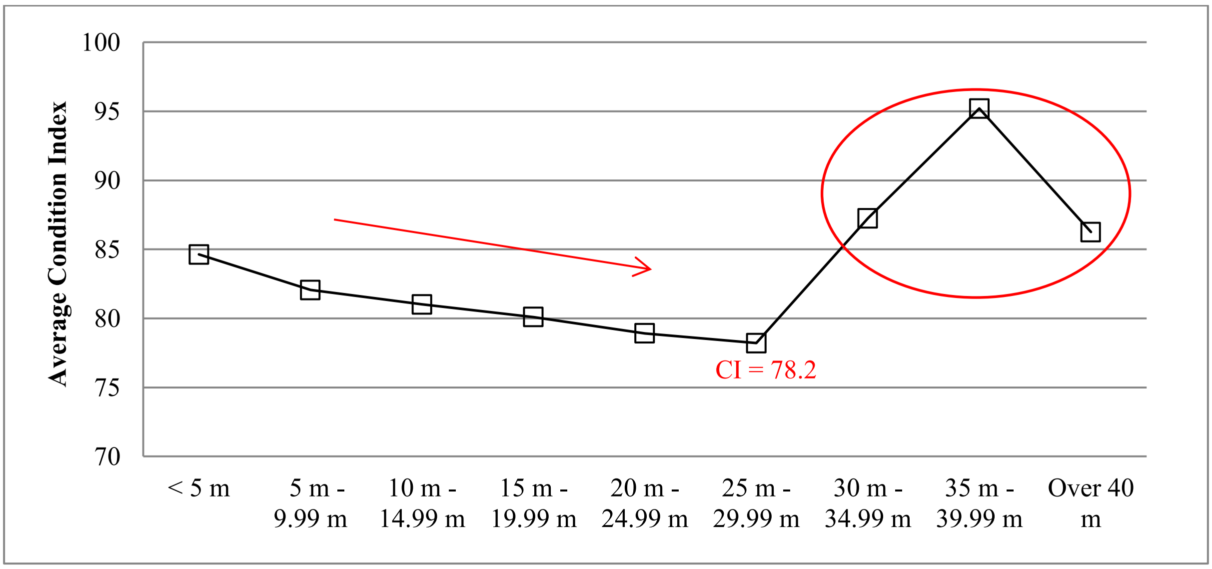

Figure 4-48: Average Condition Indices of RC Bridges as a Function of Span Length

The average CIs for the span lengths between 0 m and 30 m showed a decrease with increasing span length categories suggesting that the RC bridges indicated a decrease in condition or increased deterioration with increasing span length. The lowest average CI was that of 78.2 for span length category between 25 and 30 m. The decreasing average CIs were associated with shear effects affecting short to medium span bridges.

The average CIs for span lengths between 30 and 40 m increased with increasing span length. The average CIs peak for span length category between 35 m and 40 m and thereafter decreased for bridges over 40 m. It may be noted that because the quantity of RC bridges over 30 m only constituted 2 % of the total RC bridges, the high average CIs computed for these RC bridges may also be regarded as a result of the small sample size.

4.7.2 RC Culverts

A total of 1323 (96 %) of RC widths were investigated. The distribution of RC culvert width is illustrated in the histogram in *Figure 4-49.*

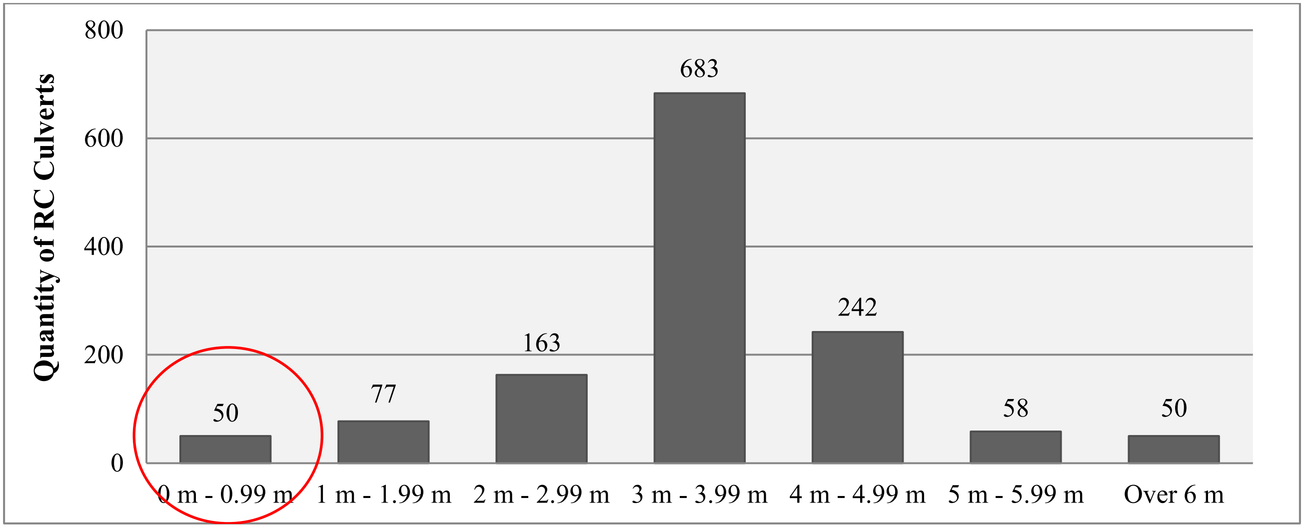

Figure 4-49: Distribution of RC Culverts by Width

It was found that 52 % of these bridges had span lengths between 3 m and 4 m. Only about 4 % of the span lengths were over 6 m. There were 50 RC culverts recorded as having a width of less than 1 m, after inspecting these using the inventory photos, they were assumed incorrectly recorded on the database.

The average RC span length was calculated to be 3.1 m. *Table 4-43* provides summary statistics for the RC bridge span lengths.

Table 4-43: Summary Statistics for RC Culvert Span Lengths

Summary Statistics	
Mean	3.1 m
Min	-
Max	6.25 m
Standard deviation	0.9885379
Variance	0.9772071
Skewness	0.3351559
Kurtosis	4.146424

Table 4-44 presents results obtained from the binary logistic regression analysis to measure the significance of width on the observed predominant defects on RC culverts.

Table 4-44: Results from Logistic Regression Analysis between Defects and Width for RC Culverts

Dependant Variable	Odds Ratio	Std. Err.	Z	P>|z| (p -value)	[95 % Conf. Interval]	
Cracking	1.111575	.0556559	2.11	0.035	1.007673	1.22619
Spalling	1.098284	.0598668	1.72	0.085	.9869981	1.222118
Defective Concrete	.8281596	.0518033	-3.01	0.003	.7326043	.9361785
Scour	.9289974	.0719647	-0.95	0.342	.7981349	1.081316
Movement/ Rotation	1.063542	.0817296	0.80	0.423	.9148351	1.236422
Reinforcement Corrosion	1.058101	.0864029	0.69	0.489	.901611	1.241752

The results show that with a one unit increase in RC culvert width, cracking and defective concrete were statistically significant. The unit increase in width increased the odds of cracking by 1.11 and the odds of defective concrete by 0.82.

Figure 4-50 showed a general increase in frequency of cracking, spalling, observed movement/rotation defects as well as reinforcement corrosion for widths between 0 and 4 m. For widths greater than 4 m, a decrease in the frequency of defects was observed thereafter.

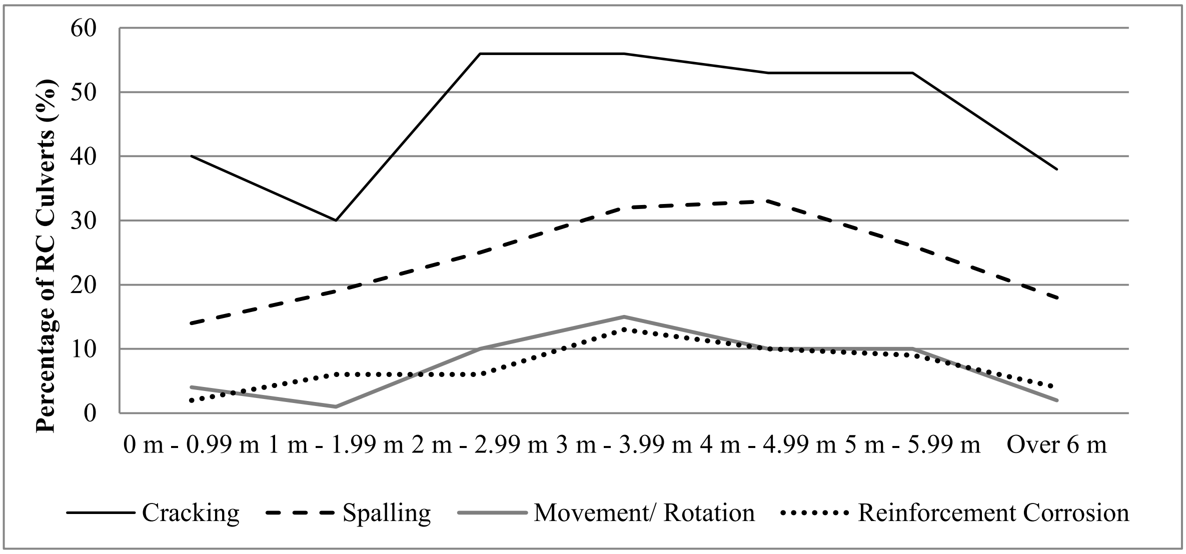

Figure 4-50: Relationship between width and Cracking; Spalling; Movement/Rotation and Reinforcement Corrosion for RC Culverts

Furthermore, a general decrease in RC culvert condition for scour and defective concrete was observed for increasing width (refer to *Figure 4-51*).

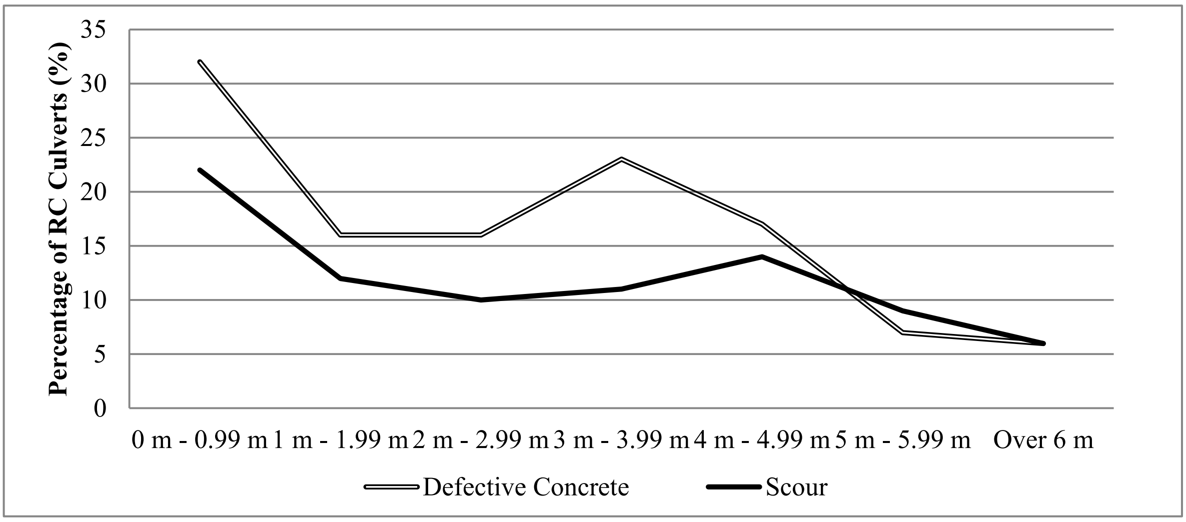

Figure 4-51: Relationship between Span and Defective Concrete and Scour for RC Culverts

The average CIs showed a relatively small general decrease with increasing width as seen in *Figure 4-52*.

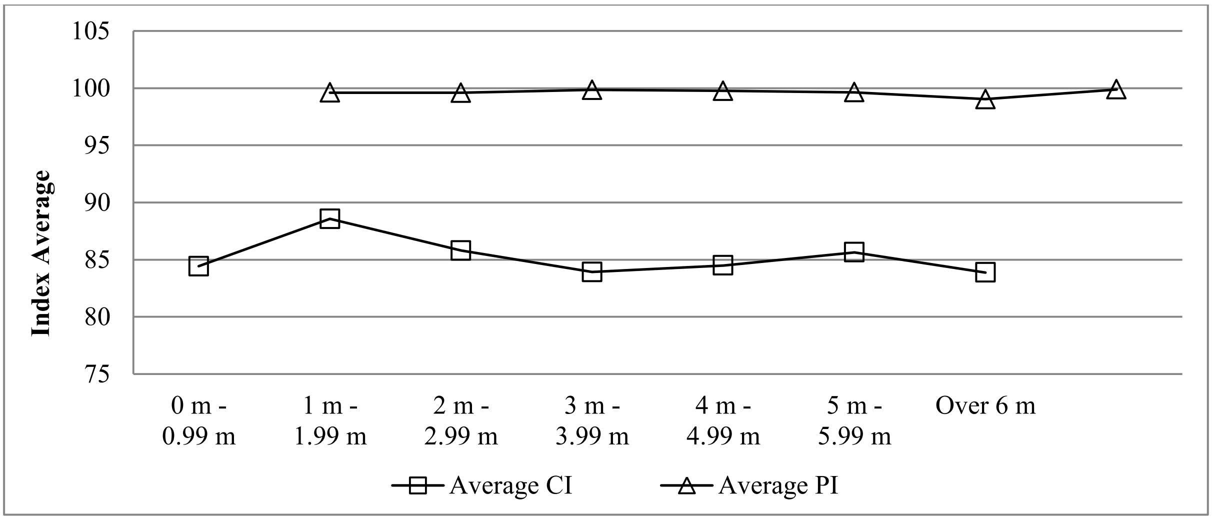

Figure 4-52: Average PI and CI for RC Culverts of Various Width Groups

Figure 4-53, highlights observations made with respect to the average CIs for the increasing span length categories

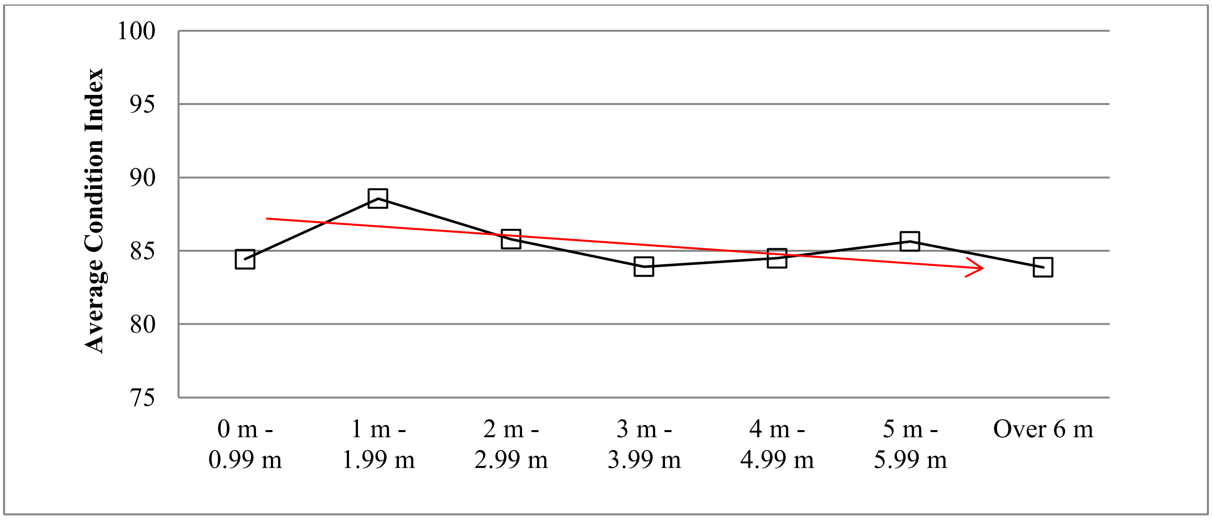

Figure 4-53: Average Condition Indices of RC Culverts as a Function of Width

The highest average CI was that of RC culverts of widths between 1 m and 2 m which was 88.6. It may be noted that he average CIs were fairly constant for the various span groups.

4.8 Overall Condition of RC Structures

The distribution of PIs and CIs of all inspected RC culverts is presented in *Figure 4-55* and *Figure 4-56.*

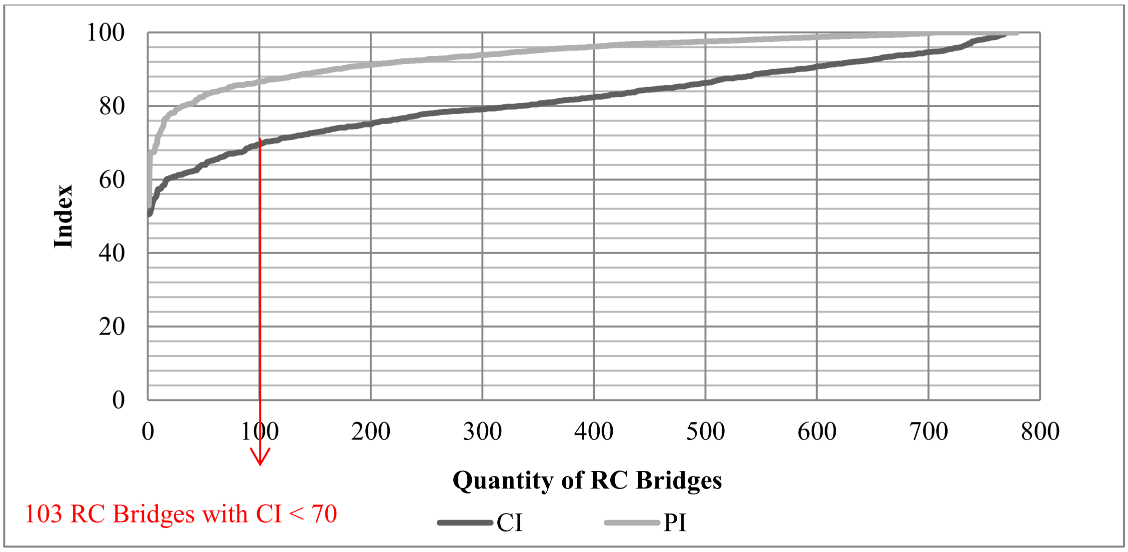

Figure 4-54: CIs and PIs for RC Bridges

The average CI and PI for RC bridges was calculated to be 81.7 and 93.8 respectively, suggesting that the structured were in good condition as the average PI > 75. There were 103 RC bridges which had CI < 70. These were within the warning level indicating that they would be prioritized for remedial work activities.

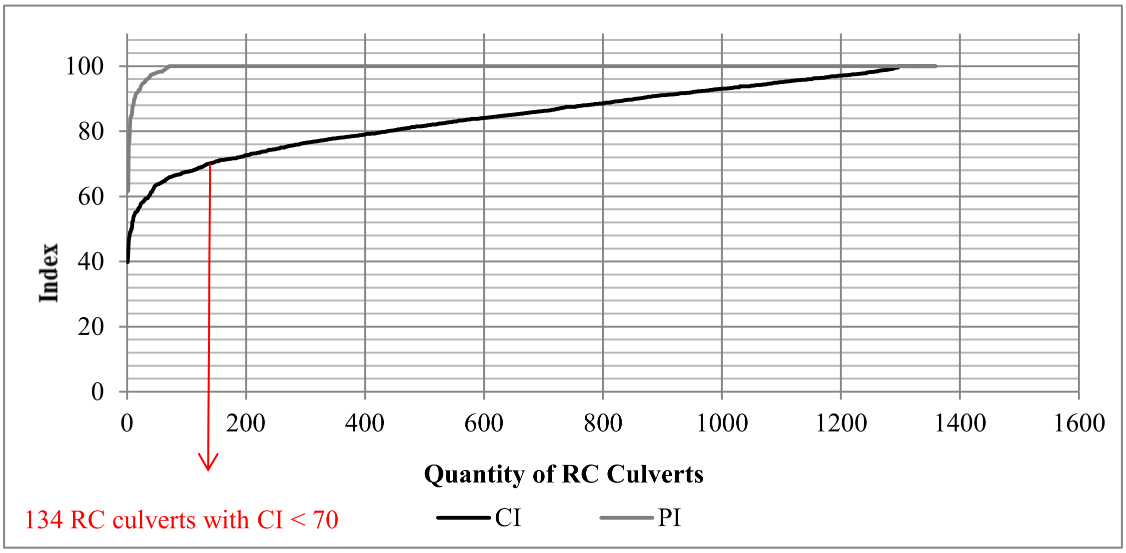

Figure 4-55: CIs and PIs for RC Culverts

The RC culverts had an average CI of 84.5 and an average PI of 99.7. There were 134 RC culverts with a CI < 70. These would also be prioritized for remedial work activities.

4.9 Summary

A total of 788 RC bridges and 1370 RC culverts were investigated. The most predominant defects among RC bridges were cracking, spalling, joint defects, surface erosion, movements/rotations, as well as reinforcement corrosion. The most predominant defects among RC culverts were cracking, spalling, defective concrete, scour, movement/rotations as well as reinforcement corrosion. Cracking and spalling were the most common of the predominant defects and were distributed mostly on abutments and decks/beams of RC bridges and wing walls of RC culverts.

The cracking, spalling and reinforcement corrosion were assumed characteristic of the mechanism of chloride ingress; common to the coastal areas of the Western Cape. Also, carbonation was regarded a possible cause as some of the RC structures were old enough to have been experiencing its effects. Joint defects as well as movements/rotations were associated with several causes that included small movements such as daily thermal cycles and larger movements such as traffic induced movements (including those due to overloading). Surface erosion was assumed a consequence of the environment and defective concrete a result of inadequate construction practices and techniques. Lastly, scour was attributed to the increased outlet velocities and large variations in vortices and eddy currents experienced downstream during the heavy rainfall days of the Western Cape winter season.

The findings from the data mining activities showed that the Central Karoo DM had the greatest percentage of RC bridges with cracking, spalling and reinforcement corrosion. The state of these RC bridges was attributed largely to their high mean age of 50.1 years, which also had a low standard deviation relative to the other municipalities. On the other hand, the Eden DM had the highest percentage of RC culverts with cracking and spalling. However, a very small portion of these exhibited reinforcement corrosion. These findings were assumed characteristic of their lower mean age of 25.2 years. The LRA indicated that RC structures from the Central Karoo DM had the greatest likelihood of having the predominant defects relative to RC culverts in the base category (the Cape Winelands DM) when compared to other municipalities. Additionally, the RC structures of the Central Karoo DM were found to have the lowest average CIs; 77.1 for RC bridges and 76.8 for RC culverts. Thus the defects on the RC structures in the Central Karoo DM could be assumed more severe (see *Figure 4-*

57). It may also be noted that the average CIs for RC culverts were generally higher than those for RC bridges.

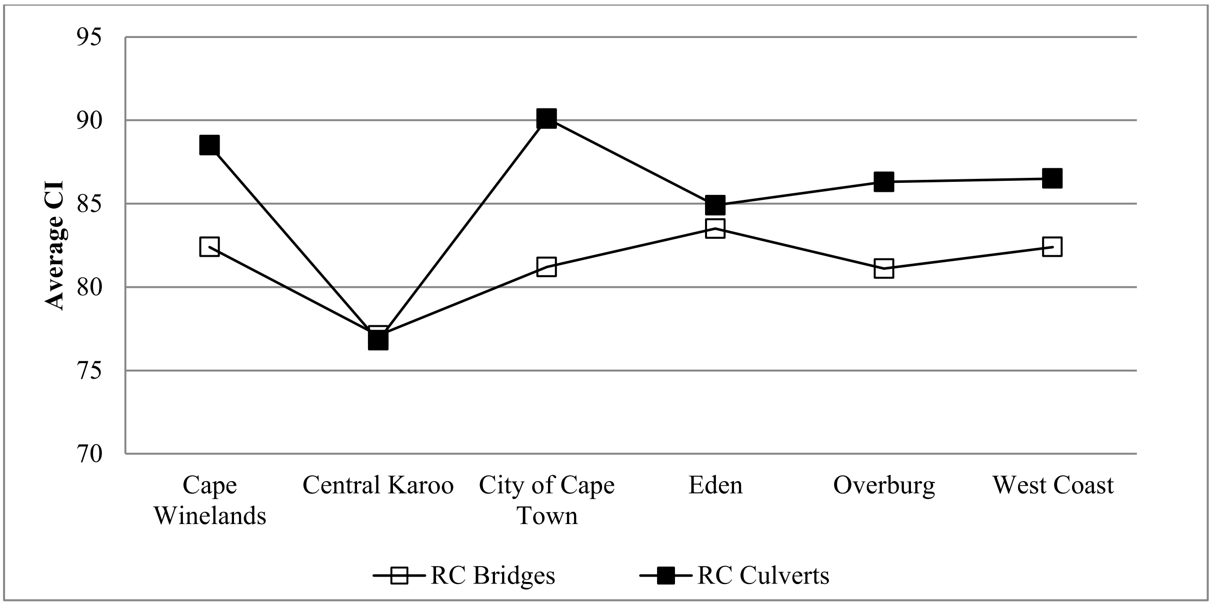

Figure 4-56: Average CI for RC Bridges and RC Culverts per District Municipality

None of the results pertaining to the various RC structure types from the LRA were statistically significant as the p-values obtained were greater than 0.05; some of the 95 % confidence intervals were zero and some odds ratios did not differ from 1. However, statistically significant relationships were identified when investigating the most predominant defects on simply supported and continuous RC bridges (the most common RC bridge types) as well as the predominant defects on precast and simply supported in-situ portal frame RC culverts (the most common RC culvert type). The base category was selected as the RC bridge or culvert structure type under investigation located in the Cape Winelands DM. The likelihood for RC bridges to have cracking, relative to an RC bridge in the base category, was highest for simply supported RC bridges as opposed to continuous RC bridges. The likelihood of RC culverts to have cracking was highest for simply supported in-situ portal frame RC culverts than precast portal frame RC culverts. Furthermore, RC structures located in the Central Karoo DM generally had higher odds ratios compared to other municipalities, implying they had a higher probability of having defects.

The data mining activities employed showed that the simply supported RC bridges were the most common RC bridge structure type (81 %). This structure type also had a relatively high mean age (48.7 years) as well as a relatively low average CI (81.3). Though the standard deviation of the age indicated a larger spread in age in comparison to the other structure types, their high mean age and location (60 % were located along the costal district municipalities) were considered contributory factors to their poorer condition suggested by the relatively lower CI. The portal frames were the most common RC culvert structure types (90 %); 72 % simply supported in-situ portal frames and 12 % precast portal frames. The cast in-situ RC portal frame culverts had a mean age (50.5 years); over twice the age of those which were precast. They also had the lowest average CI compared to all the other RC culvert structure types. Furthermore, a larger sample of the RC culverts cast in-situ (60 %), compared to those that were precast (53 %), were located along the costal municipalities. Thus the combination of the high mean age, their location and the consequences of casting in-situ were assumed the contributing factors to their poorer condition.

The average age of the RC bridges and culverts was 42.8 years. The LRA indicated that there were statistically significant relationships between cracking and increasing RC bridge span length as well as cracking and spalling and increasing RC culvert width. The data mining activities suggested that there was a general increase in RC structures with predominant defects with increasing age categories. Though there was a general decrease in average CIs per increasing age category, when the average CI graph for all bridges was compared to that of the RC bridges the following were noted (also see *Figure 4-57*):

1. The average CI for all bridges less than 10 years was lower than for those of age category 10 to 19 years. Whereas, RC bridges of less than 10 years had the highest average CI compared to the preceding age categories.

2. The age category of all bridges between 30 to 39 years was the turning point for the average CI graph for all bridges. On the other hand, the turning point for the average CI graph of RC bridges was shifted to the age category of 40 to 49 years.

3. The average CI for the age category of 60 to 69 years peaked for both graphs.

4. The graph of RC bridges generally had relatively higher average CI values than the graph for all the bridges.

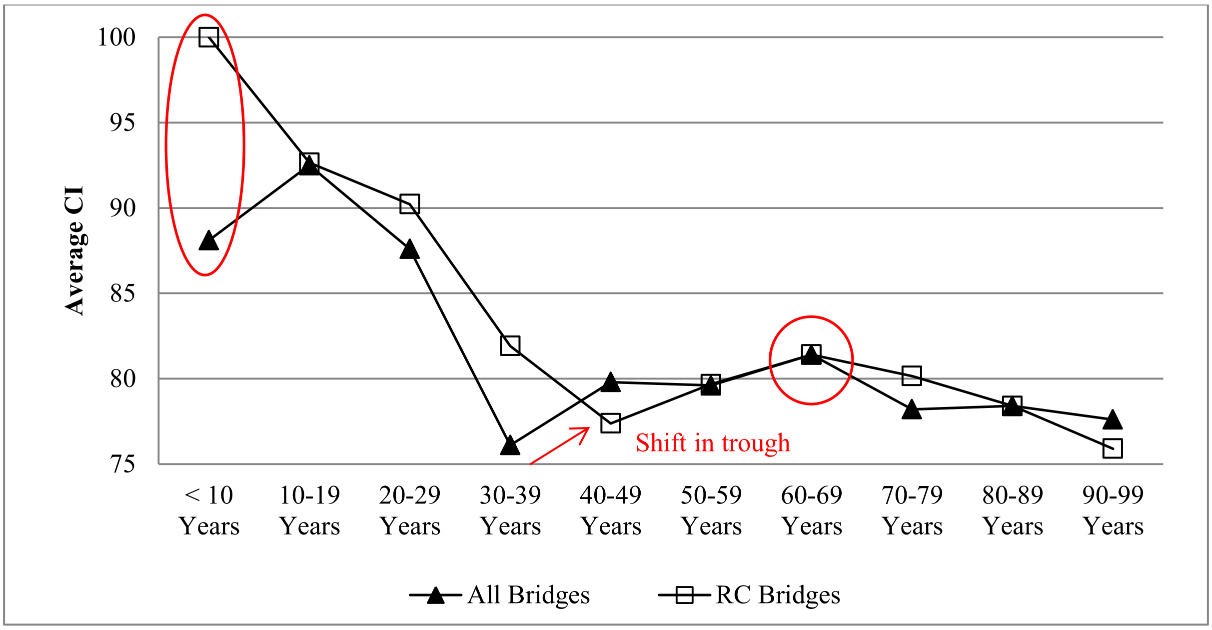

Figure 4-57: Average CIs for All Bridges and RC Bridges

When comparing the average CI graph for all the culverts to that of the RC culverts the following was noted:

1. Both graphs followed the same shape and hence they both peaked at ages 60 to 69 years and had troughs at 40 to 49 years as well as 70 to 79 years

2. The graph of RC culverts generally had relatively higher average CIs than the graph of all the culverts (see *Figure 4-58*).

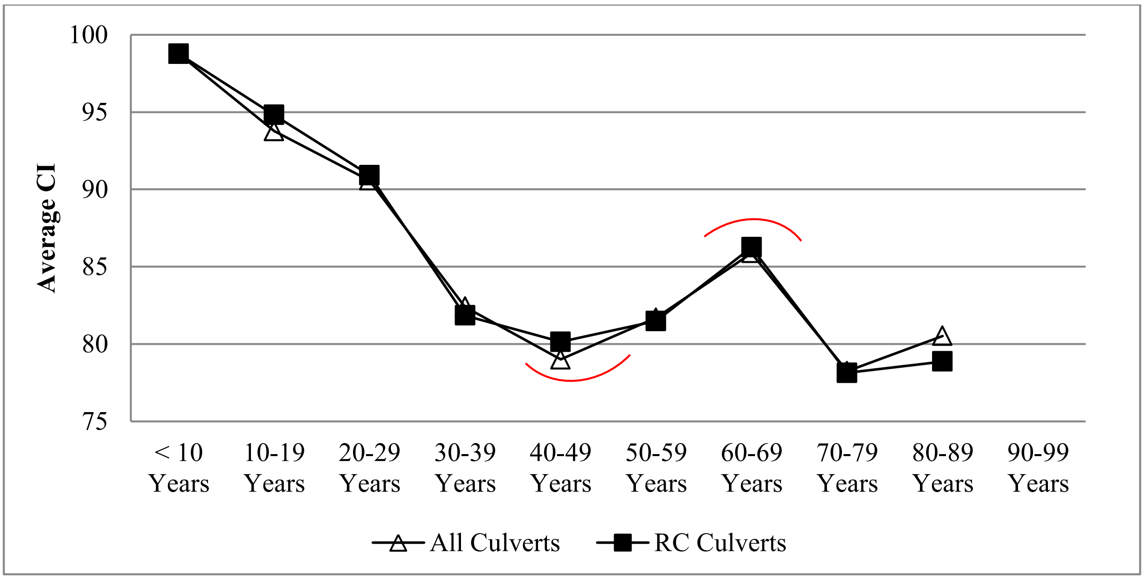

Figure 4-58: Average CIs for All Culverts and RC Culverts

Lastly, when comparing the average CI graphs for RC bridges and RC culverts, the following observations were made (also see *Figure 4-59*):

1. Both graphs followed a similar pattern and showed a clear decrease in average CI for RC structures between the ages of 0 and 50 years.

2. The average CIs of age category 40 to 49 years were turning points in both graphs. These were relatively lower than the average CIs of other age categories. A large percentage of the RC bridges (25 %) and culverts (43 %) in this age category were from the Central Karoo DM. Thus implications of employing inadequate construction practices and techniques in a semi-arid environment such as the Central Karoo DM as well as the relatively older age of the structures were assumed to have influenced the average CI.

3. The increase in average CIs between 50 and 70 years for both graphs was assumed attributed to the use of more durable concrete mixes during this period. The peak average CI at age category 60 to 69 years was assumed characterised by the smaller quantity of RC structures constructed during this period compared to successive age categories. Therefore more care could have been given to their construction hence the peak in the CIs.

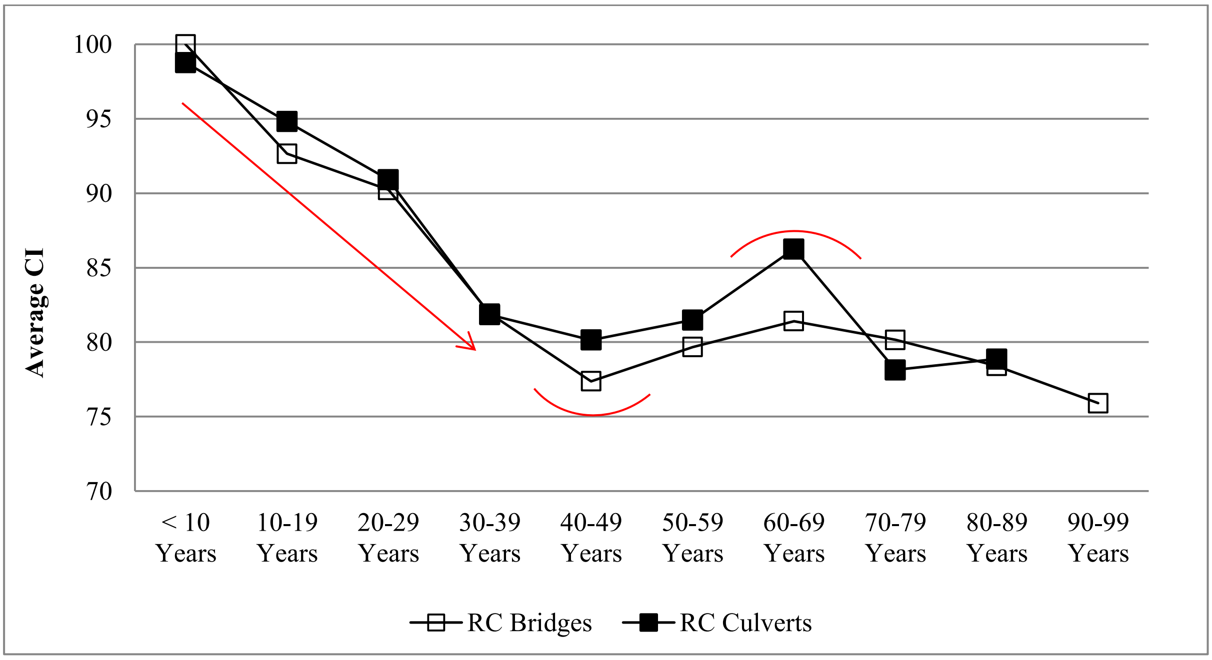

Figure 4-59: Average CIs for RC Bridges and RC Culverts per Age Category

The average CIs for RC bridges and culverts were 83.8 and 84.0, respectively. Thus RC culverts were generally in better condition than RC bridges.

Lastly, LRA identified a few statistically significant relationships between defects and increasing span length/width. Additionally, the data mining activities suggested that there were generally increasing percentages of RC structures with predominant defects with increasing span length/width. The average CI graph for RC bridges showed a gradual decrease for bridges of span lengths between 0 m and 30 m. Thereafter it increased and peaked for RC bridges of span length between 35 m and 40 m. The decrease in CI was attributed to the shear effects affecting medium to short span RC bridges. The RC culverts showed relatively smaller general decrease with increasing width.

Chapter 5

5 Conclusions and Recommendations

5.1 Conclusions

To enhance the current knowledge of the of state and condition of RC bridges and culverts in the Western Cape Province, an analysis of the Struman BMS database was conducted. The main objective of the research was to identify the various types of defects on RC structures, determine those that were predominant and discuss their associated deterioration mechanisms and/or causes. In addition, the relationships between the predominant defects and the relevant inventory data such as; location (in terms of district municipalities); structure type, age and span length/width were investigated. The data mining techniques and the Logistic Regression Analysis (LRA) procedure employed in the study were well rooted in providing a deeper understanding of the relationships that exist between the inventory and inspection data of RC structures. Although the severity of the defects was disregarded, the investigation of the CIs in relation to the inventory data provided a good indication of the state of the structures in terms of their condition.

The RC structures were found to have similar defects. Of the 15 defects investigated, the most predominant among these structures were that of cracking, spalling, movement/rotations and reinforcement corrosion. Furthermore, a high percentage of RC bridges exhibited joint defects and surface erosion, whereas a high percentage of RC culverts exhibited defective concrete and scour.

The deterioration mechanisms associated with cracking, spalling and reinforcement corrosion were largely those of chloride ingress (common in the coastal areas of the Western

Cape) and carbonation. Several causes for joint defects were identified and these were also associated with the observed defects due to movements and rotation. These included traffic induced loading and overloading as well as the accumulation of debris in joints as a result of inadequate routine maintenance. Surface erosion was attributed to deterioration due to environmental factors and defective concrete was associated with poor mix designs as well as the use of inadequate construction practices and techniques. Lastly, scour was attributed to the increased outlet velocities experienced downstream during periods of heavy rainfall.

Location, structure type, age, span length and width were all found to be important contributors to the prevalence of predominant defects of both RC bridges and culverts. Regarding the relationships between predominant defects and inventory data (location, structure type, age and span length/width), the following conclusions were drawn:

i. Structure location was found to be the most statistically significant in predicting, by means of odds ratios, the likelihoods of RC structures to have various predominant defects relative to the selected base category. However, when attempting to investigate the distribution of predominant defects through data mining, both the mean age of the location and the standard deviation of the age should also be considered, as the combination better explains variations in the percentages of RC structures with defects in the various locations.

   Thus the structures in the Central Karoo DM generally not only had a greater likelihood of having predominant defects, but they also had a relatively higher mean age and a lower standard deviation. Hence they have a low average CI.

ii. Simply supported RC bridges and simply supported in-situ portal frame RC culverts constituted too large a percentage of the RC structures to enable conclusions to be drawn on the other structure types.

   Precast portal frame RC culverts were generally in a better condition than simply supported cast in-situ portal frame culverts and simply supported RC bridges. The discrepancies in their average conditions were associated with inadequate construction site practices associated with casting in-situ versus the care taken when pre-casting.

iii. The percentages of RC structures with predominant defects generally increase with increasing age. Furthermore, there was an overall decrease in the condition indices of the structures with increasing age thus implying that the RC structures condition worsened with increasing age.

iv. Lastly the percentages of RC bridges with predominant defects generally increase with increasing span length for the medium to short span RC bridges. Hence their condition generally deteriorates with increasing span length. The percentages of the RC culverts with predominant defects also increased with increasing width. However their condition showed relatively small variations with increasing width.

Finally, the RC culverts are generally in a better condition than the RC bridges. Nonetheless, all the RC structures are considered to be in a generally good condition.

5.2 Recommendations

During the period of the study significant topics worth further investigation were identified and the following recommendations are made:

i. Much of the existing BMSs rely extensively on visual inspection data derived from inspections conducted by several individuals. It is acknowledged that these individuals are highly trained and experienced and generally thorough in their work. There is however an opportunity to develop computerized applications, such as structurally built-in sensors which would collect and/or verify data so as to obtain and maintain a higher degree of consistency.

ii. The study shows that BMS databases contain significant amounts of bridge and culvert data which may provide useful information. Data mining procedures and analysis tools such as those employed in the study should be in-built applications within BMS software. This would improve the outputs of the BMS by ensuring that the information contained in the data is utilized more effectively. Furthermore, the applications could easily be executed whenever data is collected or when necessary. Only the interpretation of the results produced by the applications may be manual.

iii. The derivation of the Struman BMS CIs for structures is independent of the structure's material, type and environmental conditions. While each parameter presents a unique influence on the condition of the structure, interpretation of the CI is constant for all structures regardless of these parameters. Efforts should be undertaken to investigate CI indicator limits that would take these into consideration and hence develop indicators specific to environment, material and structure type.

iv. The study involved the investigation of a broad range of defects, taking into account only their frequency but not their severity. Further research should be conducted on the severity of these defects. Additionally, cracking was identified as one of the predominant defects among the RC structures. Further research pertaining to the various types of cracks (structural and non-structural) and their predominance may be conducted using the specified methodology. Additionally, the approach could be used on other datasets such as those of SANRAL, and the Namibian Roads Authority, to validate the research findings.

Chapter 6

6 References

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Appendix A: Inventory Data for RC Structures

Table A-1: Distribution of RC Structures

District Municipality	RC Bridges	RC culverts
Cape Winelands	193	293
Central Karoo	88	275
City of Cape Town	78	22
Eden	162	360
Overberg	110	188
West Coast	170	238
TOTAL	796	1376

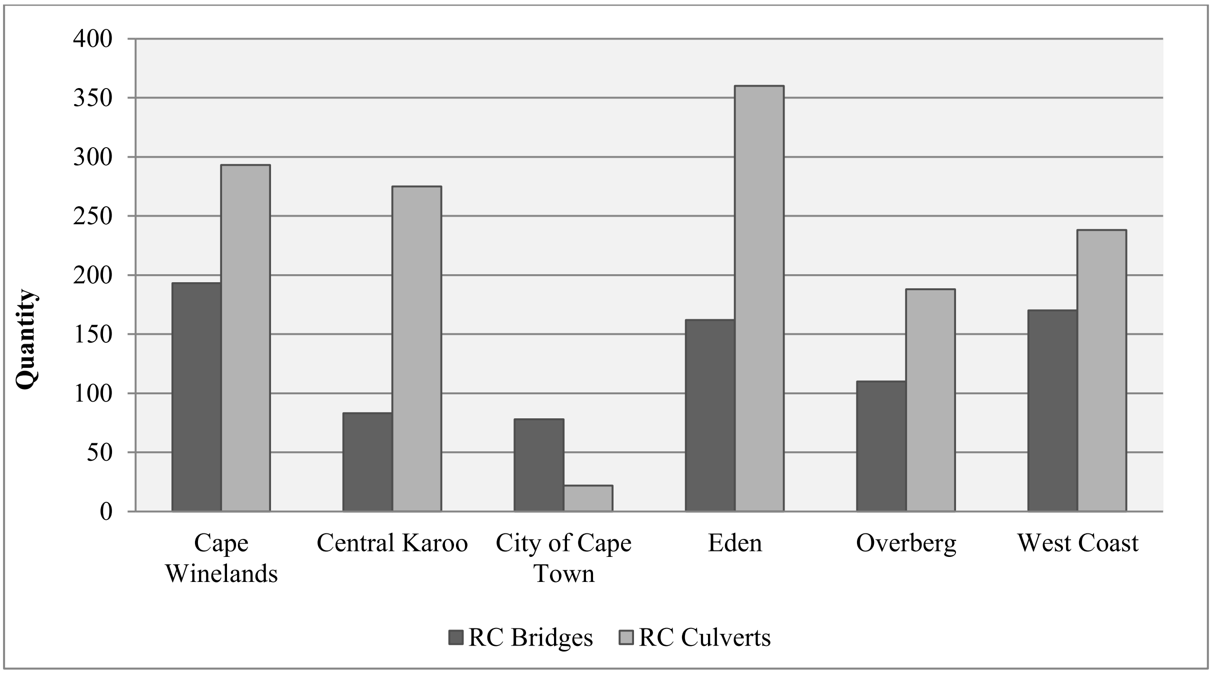

Figure A-1: Distribution of RC Structures

Table A-2: Distribution of RC Bridge Structure Types per District Municipality

Structure Types	**District Municipalities**					
	Cape Winelands	**Central Karoo**	**City of Cape Town**	**Eden**	**Overberg**	**West Coast**
Arch	1	0	2	5	0	1
Cantilever	1	0	2	2	2	1
Cellular	1	7	15	1	0	4
Continuous	16	76	1	27	9	9
Portal Frame	2	0	55	3	0	15
Simply supported	172	0	0	124	99	137
TOTAL	193	83	78	162	110	167

Table A-3: Distribution of RC Culvert Structure Types per District Municipality

Structure Types	**District Municipalities**					
	Cape Winelands	**Central Karoo**	**City of Cape Town**	**Eden**	**Overberg**	**West Coast**
Cellular	3	3	6	3	2	3
Concrete Pipe	1	24	0	19	9	4
Continuous	20	4	0	17	11	5
Precast Portal Frame	57	185	8	63	34	26
Simply Supported in-situ Portal Frame	211	59	8	256	132	197
TOTAL	292	275	22	360	188	238

Table A-4: Distribution of RC Bridge Age Categories per District Municipality

Age Categories	District Municipalities					
	Cape Winelands	Central Karoo	City of Cape Town	Eden	Overberg	West Coast
< 10 years	0	0	1	5	0	0
10 – 19 years	16	2	5	0	0	1
20 – 29 years	11	0	7	0	0	4
30 – 39 years	23	3	6	2	0	22
40 – 49 years	30	24	2	0	94	17
50 – 59 years	25	20	4	1	16	34
60 – 69 years	30	3	0	27	0	17
70 – 79 years	6	5	0	3	0	7
80 – 89 years	5	0	0	124	0	3
90 – 99 years	1	0	0	0	0	0
Unclassified	45	26	53	0	0	64
TOTAL	192	83	78	163	110	169

Table A-5: Distribution of RC Culvert Age Categories per District Municipality

Age Categories	District Municipalities					
	Cape Winelands	Central Karoo	City of Cape Town	Eden	Overberg	West Coast
< 10 years	1	0	4	22	5	2
10 – 19 years	10	0	3	6	8	6
20 – 29 years	5	1	0	8	8	3
30 – 39 years	9	3	1	2	1	1
40 – 49 years	19	25	0	2	5	7
50 – 59 years	11	20	0	6	6	7
60 – 69 years	10	4	0	4	4	4
70 – 79 years	4	2	0	1	0	7
80 – 89 years	2	0	0	2	3	2
90 – 99 years	0	0	0	0	0	0
Unclassified	221	220	14	307	148	199
TOTAL	292	275	22	360	188	238

Table A-6: Distribution of RC Bridge Span Length per District Municipality

Span Length	District Municipalities					
	Cape Winelands	Central Karoo	City of Cape Town	Eden	Overberg	West Coast
< 5 m	4	5	0	8	0	16
5 m – 9.99 m	95	62	31	70	0	96
10 m – 14.99 m	56	8	30	42	2	37
15 m – 19.99 m	18	7	11	30	0	8
20 m – 24.99 m	6	1	2	7	9	3
25 m – 29.99 m	5	0	2	1	0	5
30 m – 34.99 m	4	0	0	1	99	2
35 m – 39.99 m	0	0	0	0	0	0
Over 40 m	1	0	1	1	0	1
TOTAL	189	83	77	160	110	168

Table A-7: Distribution of RC Bridge Width per District Municipality

Span Length	District Municipalities					
	Cape Winelands	Central Karoo	City of Cape Town	Eden	Overberg	West Coast
< 1 m	0	26	0	0	6	5
1 m – 1.99 m	11	13	0	37	9	7
2 m – 2.99 m	28	26	4	44	20	41
3 m – 3.99 m	170	175	10	152	79	97
4 m – 4.99 m	52	25	3	68	47	47
5 m – 5.99 m	13	5	3	20	9	8
Over 6 m	9	0	1	9	4	10
TOTAL	283	270	21	360	174	215

It may be noted that RC structures which didn't have their locations (in terms of district municipalities) indicated on the database have not been included in the figures and tables.

Appendix B: Inspection Data for RC Structures

Table B-1: Quantity of RC Bridges with Defects per District Municipality

Defects	District Municipalities					
	Cape Winelands	Central Karoo	City of Cape Town	Eden	Overberg	West Coast
Reinforcement Corrosion	37	31	7	14	33	34
Cracking	88	67	57	97	83	100
Spalling	91	44	42	72	55	35
Defective Concrete	16	20	7	28	16	26
Approach Settlement	33	6	6	25	15	6
Bearing Defects	10	40	24	36	28	15
Surface Erosion	3	1	15	8	6	0
Scour	1	6	3	26	5	5
ASR	42	5	16	7	17	12
Impact Damage	8	7	13	10	8	17
Honey Combing	11	12	3	18	13	0
Embankment Erosion	2	3	9	15	6	23
Joint Defects	28	12	40	53	29	31
Movement/Rotations	64	29	4	9	32	0
Fire Damage	2	0	5	1	1	0

Table B-2: Quantity of RC Culverts with Defects per District Municipality

Defects	District Municipalities					
	Cape Winelands	Central Karoo	City of Cape Town	Eden	Overberg	West Coast
Reinforcement Corrosion	24	60	0	12	18	31
Cracking	101	45	8	228	97	81
Spalling	56	114	5	126	66	37
Defective Concrete	35	157	0	29	6	53
Approach Settlement	5	0	0	12	3	1
Bearing Defects	0	0	2	67	38	1
Surface Erosion	2	0	0	1	0	1
Scour	7	0	0	74	29	6
ASR	7	44	0	0	3	15
Impact Damage	29	4	0	11	3	10
Honey Combing	12	48	0	45	18	2
Embankment Erosion	0	26	0	38	4	19
Joint Defects	18	0	0	12	3	33
Movement/Rotations	34	14	0	11	3	0
Fire Damage	0	76	0	0	0	0