UNIVERSITY OF CAPE TOWN

PRACTICAL IMPLEMENTATION OF THE DURABILITY INDEX-BASED PERFORMANCE APPROACH

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DATE: DECEMBER 2011

Submitted to the Faculty of Engineering and Built Environment, University of Cape Town, on the 15th of December 2011, in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering

DECLARATION

This dissertation is being submitted in partial fulfillment for the Degree of Masters of Science in Engineering at the University of Cape Town. It has not been submitted before for any degree of examination at any other university. In addition, I know the meaning of plagiarism and declare that all the work in the document, save for that which is properly acknowledged, is my own.

December 2011

ABSTRACT

The purpose of this study was to examine the validity of Durability Index (DI) based performance specifications as a practical approach in the control of quality, with regard to durability, in reinforced concrete structures. The DI-based specifications entail provisions for limiting values on the relevant DI test parameters (oxygen permeability index (OPI), water sorptivity index (WSI) or chloride conductivity index (CCI)) and cover depth, depending on exposure conditions for a given service life. These specifications have been implemented by the South African National Roads Agency Limited (SANRAL) in major infrastructure projects e.g. the Gauteng Freeway Improvement Project (GFIP), which provided the main case studies considered in attaining the research objective.

To evaluate the practicality of the DI-based specifications the following aspects were considered: extent and magnitude of variability in DI-test values (OPI and sorptivity) and cover depth readings obtained from testing both in-situ and precast elements from projects involved in the GFIP; applicability of this approach on site (obtaining test specimens) and in laboratories (proper execution of test procedures); and perception of the approach by those on site i.e. Resident Engineers (REs).

To determine the magnitude of variability in the DI test values and cover depth readings, statistical analysis was done to evaluate the distribution models of data, average values and extent of variability. The applicability of the OPI test in laboratories was evaluated from review of a report from an audit exercise. In addition, questionnaires were sent out to REs in different projects of the GFIP to evaluate their perception of the DI-based specifications on site, and if this approach has resulted in an improvement in construction processes i.e. stricter controls in execution of construction practices such as compaction and curing.

From the analysis of data the following observations were made: the average values for the projects considered (DI test values), bridges and precast median barriers (cover depth) comply with limit values in specifications; however, a significant number of values failed to comply with the limit value – for OPI, sorptivity and cover depth; the variability of OPI values was low while that of sorptivity and cover depth readings was high. The high proportion of values that fail to comply with limit values needs to be carefully considered and provided for in specifications in determining payments to be made. The analysis on strength indicated that high values were obtained, sometimes as much as twice the specified value. It was observed that high strength does not result in improved penetrability properties, as previously perceived and provided for in the specifications. From the review of the report on the audit exercise, the OPI test was observed to be robust such that despite variations in equipment and execution of test, valid results can still be obtained. The response to questionnaires by REs indicates a mixed perception on the relevance of the DI-based specifications which highlights the need to raise further awareness among the engineers, concrete producers and laboratory staff on the importance of the specifications and their application in control of concrete quality, in addition to strength measurements.

ACKNOWLEDGEMENTS

I would like to thank God for giving me strength in undertaking this research. This far the Lord has brought me.

My sincere gratitude to my supervisor Prof. Mark Alexander for guiding me through the research; I am grateful for his continuous and timely review of the work that I did. I would also like to thank my co-supervisors: Dr. Hans Beushausen for his insights in certain aspects of the research; and Associate Professor Pilate Moyo who provided me with useful advice and guidance in the use of statistical methods for the data analysis.

I would also like to thank various parties from industry who assisted me with obtaining data that was used in the analysis and evaluation to attain the research objectives. In particular I would like to thank Mias Wiese from SANRAL, Santie Gouws, Amit Dawneerangen and Bruce Raath.

I am also grateful to my colleagues in the Concrete Materials and Structural Research Unit (CoMSIRU) for their assistance in peer reviews, my friends and family for their support.

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LIST OF ABBREVIATIONS

α	-	Level of significance
ANOVA	-	Analysis of Variance
CCI	-	Chloride conductivity index
CoV	-	Coefficient of variation
DI	-	Durability Index
Max	-	Maximum
Min	-	Minimum
GFIP	-	Gauteng Freeway Improvement Project
n	-	Sample size
OPI	-	Oxygen Permeability Index
RC	-	Reinforced concrete
REs	-	Resident Engineers
RMC	-	Ready Mixed Concrete
s	-	Standard deviation
SANRAL	-	South African National Roads Agency Limited

Chapter One

Introduction

1.1 BACKGROUND

Concrete is one of the most widely consumed construction materials; its consumption is described as being second only to that of water (Aïtcin, 2000). The popularity of concrete is mainly due to its versatility as it can be tailor-made to meet a multitude of requirements in the provision of protection to man, his resources and environment (Kruger et al., 1990; Pomeroy, 1990). In the recent past, there has been rapid growth in population worldwide which is accompanied with an increasing demand for concrete infrastructure. Thus, the consumption of concrete is projected to be high in the foreseeable future to meet development requirements.

However, a pervasive issue that continually faces the concrete construction industry is the extensive premature deterioration of concrete infrastructure, mainly due to corrosion of reinforced concrete (RC) structures. In a report made by the National Materials Advisory Board, 1987 it was found that 253,000 concrete decks, some less than 20 years old, were in varying states of deterioration and 3,500 were being added to this list every year (Mehta, 1997). The costs involved in the repair and rehabilitation of deteriorating structures are significant e.g. in the United States estimates in rehabilitation of structures are in the range of $100 billion (Long et al., 2001).

The lack of durability in RC structures and high costs involved in repair and rehabilitation is a serious cause of concern; it has been described as a matter of ‘national importance’ (Skalny, 1987; Swamy, 2007), and is a widely discussed topic in both local and international conferences and workshops. The widespread deterioration of structures leads to inefficient use of the earth’s finite materials and wastage of economic resources that could have been employed in other important sectors of development, such as education, research etc (Levitt, 1990; Mehta and Burrows, 2001; Dhir et al., 2008). The repairs and rehabilitation of highways and structures, which in some cases are frequent, disrupts traffic and usage of structures leading to wastage of man-hours, and taints the image of the concrete construction industry to the general public.

1.1.1 Durability of RC structures

A fundamental aspect in ensuring durability of the reinforcement and associated damage to reinforced concrete structures is an understanding of the deterioration mechanisms (corrosion) and factors that influence it (Browne, 1986; Clifton, 1993; Hearn and Figg, 2001). Reinforced concrete structures are continually under attack by aggressive substances in the environment (oxygen, chlorides, carbon dioxide, and water) that initiate and sustain the corrosion process. The durability of RC structures is therefore dependent on protection of concrete cover to reinforcement - resistance to aggressive substances, which is dependent on its penetrability and thickness (Bentur et al., 1997). The penetrability of concrete cover is influenced by various factors: concrete mix composition (maximum water/cement ratio and minimum cement content); and proper execution of construction process (placing, compaction, finishing, curing). The cover thickness is dependent on proper placing to ensure specified depth is achieved.

The production of durable concrete structures has been described by Wood (1997) as a chain that involves the owner, design consultants, concrete producer and contractor; any weak link in this chain compromises the durability of a structure. A schematic representation of the different parties and their functions (responsibilities) in the design and construction of a durable structure is illustrated in Figure 1.1.

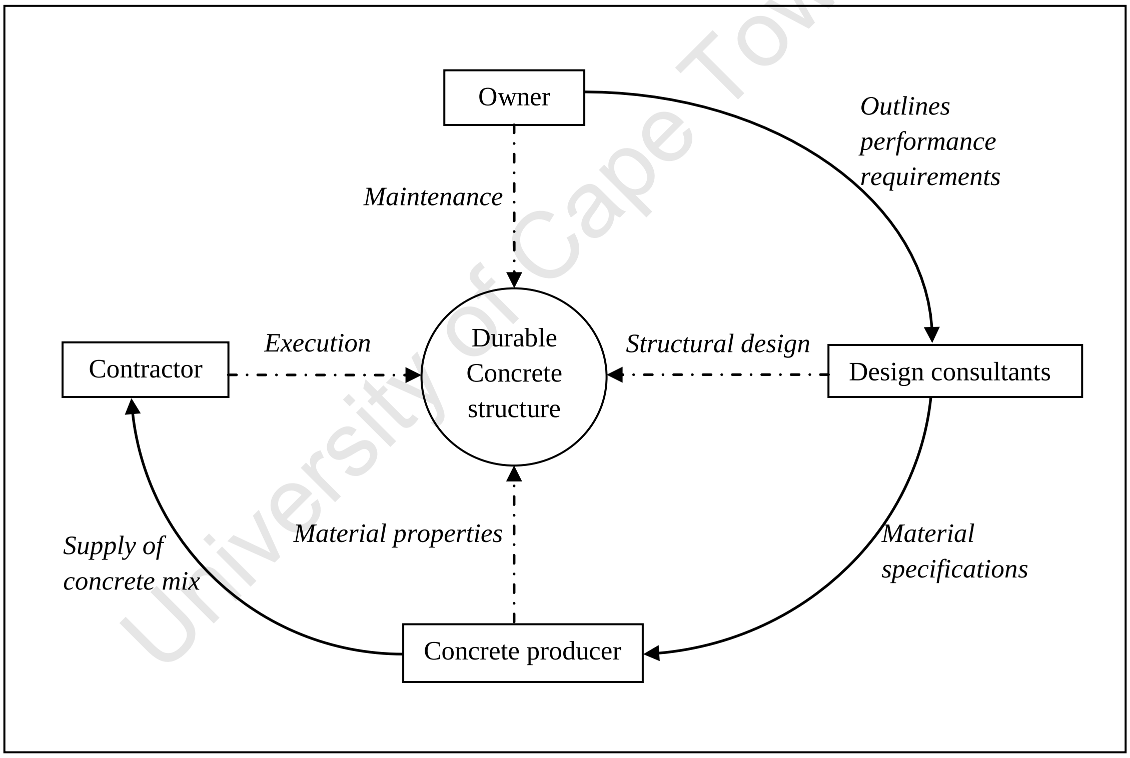

Figure 1.1: Schematic of parties involved in production of reinforced concrete structure (CEB, 1992; Wood, 1997).

From Figure 1.1, the design consultants comprise a team of architects and engineers who translate the owner's requirements into a structural design. The structural engineers write out concrete specifications by considering provisions in current standards which are mainly based on a 'deemed-to-satisfy' approach which assumes that on conforming to limiting values, e.g. maximum water/cement ratio and minimum cement content, durability requirements of a structure are attained (BS EN 206-1, 2000). The concrete producer (assuming concrete is batched and prepared by a ready mix concrete supplier) proportions the constituent materials of concrete to achieve the limiting values specified and supplies a concrete mix to site. The contractor is then required to carry out proper execution of construction processes (placing, compaction, finishing and curing of concrete); guidelines on proper execution are provided in project specifications.

1.1.2 Prescriptive approach to durability design

The approach to specification for durability of concrete structures outlined in the preceding section is described as ‘prescriptive’. The limitations that arise with prescribing concrete mix composition include:- limiting values of concrete mix composition provided are difficult to verify on site which may lead to non-compliance and a compromise on durability of concrete structures (Neville, 2001; Lobo et al., 2005); limiting values provided may not be valid in current application as they were developed from past experience and empirical relations (Clifton, 1993) which fail to consider changes that have occurred in material constituents e.g. cement which over the years has faster reaction rates and rapid gain in strength due to an increase in C_3S content and finer particles (Pomeroy, 1986; Aïtcin, 2000; Mehta and Burrows, 2001); in addition, provisions fail to consider the new range of blended cements which improve concrete properties and may be readily available at a low cost.

The penetrability of concrete cover, in addition to concrete mix provisions, is also dependent on proper execution of construction processes – the placing, compaction and curing of concrete. Majority of durability failures have been attributed to poor workmanship and failure to properly execute construction process which may lead to poor quality of concrete cover and inadequate cover depths (O'Brien et al., 1987; Mehta and Burrows, 2001; van Breugel, 2006; Bentur and Mitchell, 2008; Neville, 1998; Clark et al., 1997). In as much as guidelines are provided on the proper execution, there is no reliable measure to verify the resulting properties of concrete cover – its penetrability. The current approach to verification of concrete quality is based on a measure of strength. However, strength is an inadequate criterion as it measures bulk properties of concrete and fails to verify penetrability properties of concrete cover (Neville, 1987; Dhir et al., 1990; Skalny and Idorn, 2004). The limitations present in the current prescriptive approach to specifications have led to development of the performance-based approach.

1.1.3 Performance-based approach

This approach to durability design is quantitative. The deterioration mechanism is considered through the use of suitable mathematical models that are based on the rate of transport of aggressive substances that cause deterioration (Clifton, 1993; Somerville, 1997; Richardson, 2002; Andrade, 2006). From the application of mathematical models in design for a pre-defined working life, output parameters such as material properties (e.g. diffusion coefficient) and geometric properties (cover depth) are determined, which can be verified using suitable performance tests. The limiting values used in performance specifications (material and geometric properties) should be: clear so as to avoid ambiguity during implementation; measurable; achievable; and enforceable (Lobo et al., 2005; Bickley, 2006).

Provisions for design of structures for a given service life based on a performance-based approach are given in the following standard/code: (a) ISO 13823 (2008) which is based on a limit-state approach. To check for durability, a service life format where predicted service life should be more than or equal to specified design life, or limit-state approach where resistance capacity of structure should exceed action effect. (b) *fib* bulletin 34 (2006) where a full probabilistic approach, partial factor or deemed-to-satisfy approach may be applied.

In the development of performance specifications, a framework was proposed by Harrison (1995) which involves a seven stage process. The stages involved are: (i) definition of exposure class where prevalent form of deterioration is considered; (ii) quantitative design methodology and criteria for what defines the end of service life; (iii) test procedures which relate output parameters to design methodology; (iv) provisional limiting values which are checked against traditional durability test methods; (v) establishing limits of test applicability; (vi) determination of effective systems for production and acceptance testing; (vii) implementation of specifications in full scale trials and long term monitoring to confirm provisional limiting values.

Different performance-based approaches have been developed in an international context (Baroghel-Bouny, 2006; Hooton et al., 2005; Polder et al., 2006; Arskog et al., 2006; Torrent, 2006) and locally using the Durability Index performance-based approach (Alexander, 2008). The advantages of this approach to durability design are: it is a rational approach to design for durability as the penetrability properties of concrete cover are verified; concrete composition and means of production are not outlined which allows the concrete producer to be flexible and innovative in selection of materials; the concrete producer and contractor work together to ensure that a concrete mix that meets the required performance is designed.

1.2 PROBLEM STATEMENT

The performance-based approach is suitable in design and construction of durable concrete structures. However, its full implementation is limited due to lack of sufficient development in performance related design and test methods (BS EN 206-1; Ho and Lewis, 1988; Richardson, 2002; Skalny and Idorn, 2004: Andrade, 2006; Dhir et al., 2008).

A variety of test methods have been developed in an international context to measure properties of concrete cover: (a) permeability e.g. Figg’s air permeability test (Cather et al., 1984), Cembureau test method by Kollek (1989), Autoclam permeability test (Basheer et al., 1994), Torrent two –chamber vacuum cell (Torrent, 1992); (b) sorptivity e.g. Covercrete absorption test (Dhir et al., 1987), water absorption test developed by Kelham (1988); and (c) diffusion e.g. rapid chloride penetration tests

(ASTM C 1202; Luping and Nilsson, 1992). However, limitations are encountered in application of these tests such as: standardization of moisture conditions for samples tested e.g. in situ structures; high variability in results (poor repeatability and reproducibility of tests); lack of a valid relationship between what is measured and what occurs under real conditions; difficulties in practical application of tests on site e.g. leakages due to failure to properly clamp test apparatus onto surface tested. The practical difficulties encountered in implementation of performance tests on site makes it difficult to use them routinely in control of concrete quality, which limits the full implementation of the performance-based approach.

The South African approach to performance-based design is through the use of durability index tests; this approach and its application in design and specification is further discussed in the subsequent section.

1.3 RESEARCH KEY QUESTION AND OBJECTIVES

1.3.1 Durability Index-based Performance approach

The durability index (DI) tests consist of three tests that are used to characterize the properties of near surface concrete – its potential resistance to ingress of aggressive substances (Alexander and Mackechnie, 2001). The three tests, oxygen permeability index (OPI), water sorptivity index (WSI) and chloride conductivity index (CCI), have been developed as: simple tests with little demand on operator’s skill; conducted at an early age (28 – 35 days); having low statistical variability; and applicable both in laboratories and on site for production and quality control of concrete mixes, in addition to assessing the construction processes (effectiveness of placing, compaction, curing etc.).

The DI test values (OPI and CCI) have been empirically related to input parameters in service life prediction models; a strong correlation was established between CCI and diffusion coefficient (Mackechnie, 1996), and a strong correlation determined between OPI and carbonation depth (Mackechnie, 1999). These empirical relationships make the tests suitable for service life design. On selection of a suitable prediction model, depending on exposure conditions, limiting values of the DI parameters can be determined and checked for compliance using the tests. Alexander and Stanish (2005) describe a multi-factor approach to performance specifications using DI tests which require verification for compliance with specified cover depth and DI test values for a given exposure class and service life.

From a seven step framework proposed by Harrison (1995) for the development of a performance-based approach, the developments in the DI-based performance approach are at the final stage that involves implementation of the specifications in full scale trials. The DI-based performance specifications have been implemented by the South African National Roads Agency Limited (SANRAL) in a major infrastructure project, the Gauteng Freeway Improvement Project (GFIP).

From this project, data was collected and analysed in order to address the ***key research question*** in the thesis which is, "Are DI based performance specifications practical in the control of concrete cover quality and assessing construction processes, which are closely linked to durability of concrete structures?"

The practicality of DI based performance specifications was assessed by considering the following aspects: - (i) Extent of variability in DI test results and cover depth values of in-situ and precast elements. (ii) Applicability of tests in laboratories and on site. (iii) Perception of the tests in practice and their effects on construction processes. These aspects are further described in the subsequent section.

1.3.2 Research objectives

(a) Evaluation of extent of variability in DI test results and cover depth values: The OPI, sorptivity and cover depth values obtained from tests done on site elements were statistically evaluated to determine the magnitude of variability. The sources of variability in a test may be due to apparatus used, quality of testing, construction practices to which a structural element is subjected to and material properties of sample. An initial study to evaluate the practicality of use of durability index tests on site was done by Gouws et al. (2001) which involved carrying out the tests on concrete samples obtained from different locations in Cape Town. From the study, the tests were found to be practical and it was observed that with proper construction practice, good DI test values can be obtained.

The current study involved a further evaluation of practicality of application of DI-based performance specifications on a large scale on site through the analysis of DI test results of concrete samples from structural elements obtained from different site locations, subjected to different construction practices and tested in different laboratories. The purpose of evaluating variability in results was to quantify the amount of variation from testing of site elements and to assess the level of compliance with limit values provided in the project specifications. In addition, cover depth values from structural elements were evaluated to determine the proportion that complies with the specified values and extent of variation of these values.

(b) Applicability of the test procedures on site and in laboratories: The DI tests have been developed as simple tests that can be used in testing of samples obtained from site elements or prepared in the lab (cubes).

The samples obtained from site elements may be from either test panels or the actual structure. For the test to be effectively used in quality control, site samples should be obtained from the source after a period of 28 days and properly packaged during transport to a laboratory for testing, to ensure minimum damage. The laboratory, on receipt of elements from site, should pre-condition elements for testing - by obtaining cores of the required size and drying in the oven at 50°C for 7 days. The DI tests (OPI and sorptivity) should be properly undertaken in accordance to test procedures to ensure that reliable results are obtained.

The aspects that were considered in evaluating the applicability of these tests on site include: (i) Are samples obtained from the site elements within the required period of 28 days and is the age of these elements properly recorded? (ii) Are elements from site properly packaged during transport to laboratories to ensure that minimum damage occurs? (iii) Are any difficulties encountered in obtaining samples from site elements and transport to laboratories?

Applicability of the DI tests in laboratories was evaluated by considering:- (i) Do the laboratories have the capacity (apparatus and trained staff) to test site elements on receipt from site, to ensure that the DI tests are carried out within a period of 28 – 35 days? (ii) Do the laboratories carry out proper testing as per the test procedures which would ensure that results obtained are reliable? (iii) What difficulties are encountered during testing – are the difficulties due to lack of clarity in test procedures, recording and computation of results or apparatus used?

(c) Perception of the tests in practice: The successful implementation of the DI based performance specifications in practice is dependent on how this approach is perceived and accepted by engineers and contractors. The purpose for undertaking these tests and obtaining measurements of concrete cover depth of finished structures, additional quality control requirements to the measure of strength using cubes, should be appreciated.

1.4 SCOPE AND LIMITATIONS

1.4.1 Scope

The main objective of the study was to determine the practicality (validity) of implementation of DI-based performance specifications on a large scale in the control of concrete quality and construction practices. The aspects of practicality that were considered are based on:- (a) an evaluation of data obtained from different projects involved in the GFIP; (b) a review of a report on an audit exercise conducted in laboratories; and (c) a review of responses from resident engineers on questionnaires sent out to determine their perception of the DI-based performance approach in practice.

The DI-based performance specifications implemented in the GFIP require the verification of DI test parameters (OPI and sorptivity) and cover depth readings; these values were obtained from different projects and collated to form a database. Statistical analysis was carried out with the aim of evaluating: - average values obtained from testing of site elements and if these values comply with limiting values set in project specifications; the proportion of values from a sample of data within a project that fails to comply with limiting values set; the amount of variability in values which indicates the level of control in carrying out of the tests. In addition to the aforementioned statistical analysis, a classification system was used for DI test values on the basis of concrete grade, source of sample (test panel or cube) and type of structure (in situ or precast) in order to determine extent of variability between different classes of data. A comparison of values from different projects was also done to determine if different construction practices and their execution employed on the different sites have a significant effect on the mean values obtained.

A second aspect of practicality of the test evaluated was applicability of tests in laboratories. Proper testing in accordance to test procedures and recording of data is essential to ensure reliability of results from samples tested (Zimmerman, 2010). To evaluate the applicability of the test, a critical review of data obtained and recorded was done to establish if all data required was recorded. A review of a report from an audit exercise conducted in laboratories that carry out the testing was also done. The aim of the audit exercise was to determine if tests were correctly carried out in accordance to test procedures, the difficulties encountered in application of the tests – if these difficulties were due to ambiguities in test procedures or limitations in apparatus used.

The perception of the tests was evaluated from response of resident engineers to questionnaires sent. The aim of the questionnaire was to determine the construction practices that were employed (an aspect that is not always recorded in spreadsheets e.g. curing methods used), the effect of the tests methods on construction processes –if this approach has led to an improvement in construction practices, and in what ways improvements have occurred.

1.4.2 Limitations

The main limitation encountered in the research was the use of secondary data obtained from different projects involved in the GFIP and establishing the reliability of these data. The quality of testing in different laboratories has a large influence on results obtained. To ensure reliability and accuracy of results, laboratories should carry out tests in accordance with test procedures and equipment used should be properly calibrated. The aspect of quality or competence of laboratories in carrying out test was reviewed by considering the audit report; however, this report does not fully cover the aspects of testing the samples obtained from site (for data used in analysis) and if at the time of testing these samples proper test procedures were followed and equipment used had been properly calibrated to ensure accuracy of test results.

A second limitation encountered was the difficulty in matching concrete mix properties and corresponding DI test values determined on laboratory samples (representing ‘material potential’), obtained from Ready Mix concrete company, with sites to which concrete had been supplied. A comparison of the ‘material potential’ values with DI test values of finished structures (‘as-built’ values) would have been useful to evaluate the effect of construction practices on durability properties by determining the margin or difference in DI test values of site elements with those obtained from the same concrete mix tested under laboratory conditions. This aspect of DI tests was therefore not considered. However, the RMC data with regard to concrete mix properties, DI test values and strength were utilized to provide background information on mix constituents with regards to cement content, water/cement ratio used, strength and range of expected DI test values.

1.5 THESIS OUTLINE

The thesis is presented in five chapters. The layout and content of these chapters is briefly described below.

The first chapter provides an *introduction* to the thesis with a brief background on durability issues that are currently facing the concrete construction industry. A major reason attributed to this prevalent issue, the prescriptive approach to design with an inadequate compliance criterion based on strength measurements is briefly discussed. Developments in performance-based approach to address these limitations are outlined. The research key question and objectives are also described, in addition to scope of thesis and limitations encountered during the research.

The second chapter presents a *review of literature* with a background on the major deterioration mechanism that affects durability of concrete - corrosion of reinforced concrete structures, and the factors that influence it, concrete penetrability. A review of test methods that have been developed to measure transport mechanisms in concrete is also provided. The current prescriptive approach in design for durability is critically reviewed and a discussion of the performance-based approach and its developments given. In addition, a review of the Durability Index tests developed in South Africa - their application and use in performance-based design and specification is provided.

The *methodology* adopted in the research is presented in Chapter three. This chapter provides a review of the data collection process which involved obtaining data from: the GFIP – durability index test values and cover depth readings; review of a report from an audit exercise carried out on labs that undertake the DI tests; response from questionnaires sent out to Resident Engineers; and details on concrete mixes used in structural elements. The statistical methods utilized in analysis of data are also described in this chapter.

Statistical analysis of data obtained from the case study GFIP - DI test values (OPI and sorptivity), strength values and cover depth readings is presented in Chapter four. The data analyzed in this section is to attain one of the research objectives of determining the extent/ magnitude of variability in data. A comparison of strength values with DI test values is also made to determine if a relationship exists between the two.

The final chapter, Chapter five, presents the *conclusions* made with regards to practicality of the DI-based performance approach in the design and specification of structures for durability and *recommendations* identified that could be applied in improvement and further development of this approach.

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Chapter Two

Literature Review

2.1 INTRODUCTION

Concrete has been described as the construction material of choice; one of the reasons for its wide usage is its durability. The review of literature in this chapter provides a background on the lack of durability in reinforced concrete (RC) structures due to corrosion, a pervasive concern currently facing the construction industry. The chapter is sub-divided into three main sections. Section, 2.2 provides a review of durability of concrete structures with a discussion on corrosion and factors that cause it – penetrability of concrete to aggressive substances that initiate and sustain the process. Transport mechanisms of these aggressive substances and methods that have been developed to measure transport are also outlined. Section 2.3 gives a review of approaches that are used in design for durability with a critique of provisions in the current standards for durability. Limitations in current specifications ('prescriptive' approach), mainly the acceptance criterion based on strength, are discussed. This section ends with a review of performance-based approach and developments that have taken place in an international context in this approach to durability design. The final section, 2.4 provides a review of the performance-based approach developed and implemented in South Africa using Durability Index tests.

2.2 DURABILITY OF CONCRETE STRUCTURES

Durability is defined in ISO 13823:2008(E) as, 'the capability of a structure or component of the structure to satisfy the design performance requirements over a specified period of time, with planned maintenance, under the influence of the environmental actions in a given area'.

A concrete structure is continually under attack by aggressive substances in the environment such as oxygen, carbon dioxide, sulphate ions, chlorides, acids; these substances cause deterioration reducing service life of a structure (CEB, 1992). The forms of deterioration that may occur in concrete structures include:-

- freeze-thaw attack
- sulphate attack
- alkali silica reaction
- acid attack
- abrasion
- cracking

- sea water attack
- shrinkage
- corrosion

Of the aforementioned deterioration mechanisms, corrosion of reinforced concrete (RC) structures is the most severe. The durability of RC structures depends on near surface properties of the concrete, its penetrability and thickness, which protect reinforcement from aggressive agents that cause corrosion. The ensuing sub-sections provide a review of the corrosion process in RC structures and penetrability which influences durability in these structures.

2.2. 1 Corrosion in reinforced concrete (RC) structures

Corrosion is an electrochemical process that involves both chemical reactions and the flow of an electric current (ions and electron current). The reactions take place on the surface of embedded reinforcing steel in which the steel is a mixed electrode comprising of the anode and cathode, and ions flow through pore solution in the concrete cover which acts as an electrolyte. For the corrosion process to proceed the chemical reactions must be coupled; the number of electrons lost at the anode must be equal to number of electrons gained at cathode (Bentur et al., 1997).

The chemical reactions involved in the corrosion process are described below:-

i. Oxidation (Anode reaction): Loss of electrons takes place at the anode. The iron atoms are converted into Fe^{2+} which dissolves into pore solution surrounding the steel. The electrons on the surface of the steel result in a potential difference between anode and cathode, causing a flow of electrons through the steel.

$$2Fe - 4 \text{ electrons} \longrightarrow 2Fe^{2+} \qquad (2.1)$$

ii. Reduction (Cathode reaction): Involves a reaction of electrons with dissolved oxygen and water to form hydroxyl ions which pass into pore solution.

$$O^{2-} + H_2O + 4 \text{ electrons} \longrightarrow 4OH^- \qquad (2.2)$$

The Fe^{2+} and OH^- are transported within the pore solution and react to form iron hydroxide ($Fe(OH)_2$) which reacts further with oxygen to form ferric hydroxide, $Fe_2(OH)_3$, commonly known as rust. The rust is deposited on the surface of steel and occupies a volume several times larger that of the parent material resulting in an expansive pressure (Richardson, 2002). When this pressure exceeds the tensile strength of concrete, cracking occurs. The corrosion process is illustrated below in Figure 2.1.

For corrosion to occur the relative humidity in the environment should be in the range of 70 – 80%, this influences the moisture level in concrete pores (Neville, 1995). The moisture level in the pores determines the resistivity of concrete, and the ability of ions to move through the concrete from the cathode to anode.

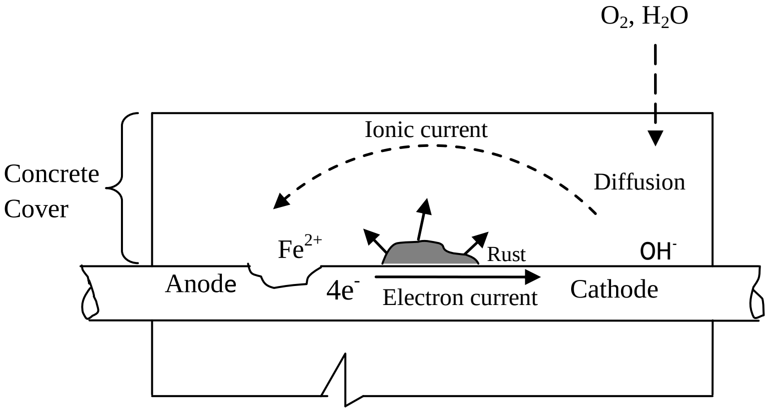

Figure 2.1: Schematic description of the corrosion process in reinforced concrete (Bentur et al., 1997).

Corrosion in RC structures attacks the steel without causing any degradation of the concrete fabric itself, other than the secondary spalling and cracking that occurs (Broomfield, 1997). The concrete cover therefore acts as a protective layer providing both chemical and physical protection. The chemical protection is due to high alkalinity in the range of 12.6 – 13 of pore solution in contact with steel which is required to maintain the stability of the oxide layer (γ-Fe_2O_3) that tightly adheres to the steel surface and keeps it in a passive state (Neville, 2007). Physical protection of concrete cover is dependent on its penetrability and resistance to ingress of aggressive agents from the environment that initiates and sustains the corrosion process (Bentur et al., 1997).

The corrosion process has been modelled by Tuutti (1982) to take place in two phases, initiation and propagation phase, as illustrated in Figure 2.2.

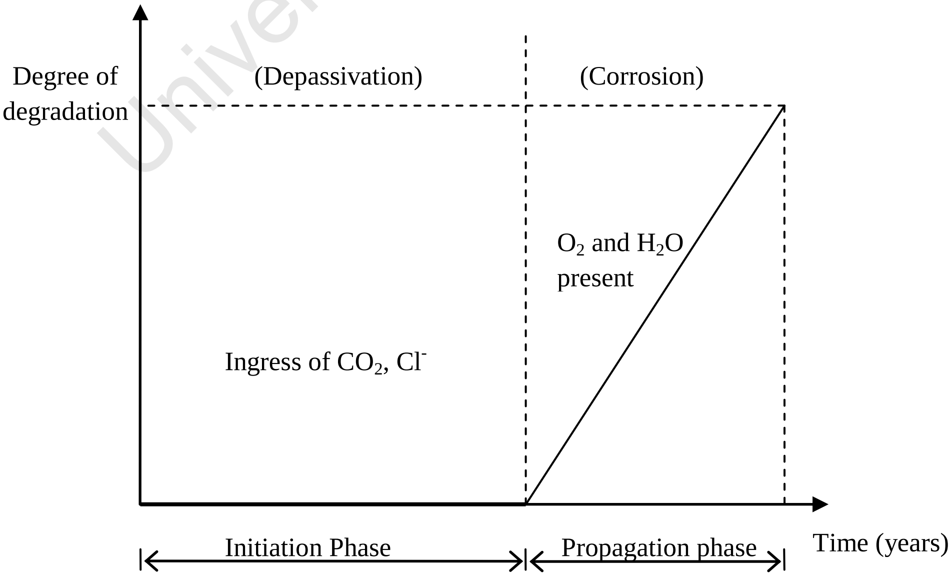

Figure 2.2: Schematic representation of the two phase corrosion model developed by Tuutti (1982).

The initiation phase involves the depassivation of steel due to: ingress of carbon dioxide that results in carbonation and a subsequent decrease in pH of pore solution; or ingress of chloride ions which destroy the passive layer. Propagation phase involves the corrosion of steel and is dependent on presence of oxygen and moisture to facilitate the reactions.

The two forms of depassivation, and type of corrosion associated with each, are described below:-

(i) Carbonation-induced corrosion: Carbon dioxide gas from the atmosphere diffuses into concrete. The rate of ingress into concrete is dependent on level of saturation in pores; diffusion is slower at high levels of saturation (Richardson, 2002). The carbon dioxide reacts with water in pore solution to form a weak carbonic acid which further reacts with calcium hydroxide to form calcium carbonate. This chemical reaction is known as carbonation and is illustrated in Equations 2.3 and 2.4.

$$CO_2 + H_2O \longrightarrow H_2CO_3 \qquad (2.3)$$

$$H_2CO_3 + Ca(OH)_2 \longrightarrow CaCO_3 + H_2O \qquad (2.4)$$

Movement of carbon dioxide takes place as a ‘carbonation front’ where its ingress into concrete is accompanied by the carbonation reaction. The formation of calcium carbonate lowers the amount of calcium hydroxide in pore solution with a subsequent reduction of pH from 12.6 to 9. The protective layer that adheres to the surface of steel is unstable at low pH which leads to the loss of passivity.

Advancement of carbonation front results in depassivation along the surface of steel which leads to cracking, delamination and spalling of concrete cover.

The carbonation process can be modelled using Equation 2.5 which considers the relationship between carbonation depth (x) and time (t). The time taken for this carbonation front to advance to surface of steel and cause depassivation is effectively the service life of the member, assuming that corrosion initiation is a limit state for durability. From Equation 2.5, it can be observed that the time taken for depassivation to occur is dependent on the cover thickness. A reduction in concrete cover would reduce the time taken for depassivation by carbonation; halving the cover reduces the life to 25% of its original design life (Bakker, 1988).

$$x = kt^n \qquad (2.5)$$

Where,

k - constant that depends on: diffusivity of the concrete, carbon dioxide concentration in atmosphere, and environmental conditions.

n - value ranges from 0.4 – 0.7, or more

A pH indicator, phenolphthalein, is used to detect carbonation in concrete. The indicator changes colour at a pH of approximately 9; above this value it remains colourless, but below this it turns purple. The test is useful to determine if a sample of concrete broken off from a structure has undergone carbonation (Richardson, 2002).

(ii) Chloride-induced corrosion: Chloride ions may be present in concrete through mix constituents, for example in aggregates or use of chloride contaminated water during mixing concrete; absorption of sea water or de-icing salts in environments that are subject to freezing and thawing (Neville, 1995). Transport of chloride ions from sea water, into and through concrete, takes place through absorption and diffusion. Chloride ions are transported in pore solution and will cause depassivation of steel even in a highly alkaline environment. The depassivation of steel is dependent on the ratio of Cl^-/OH^-; a predominance of Cl^- leads to rapid loss of Fe^{2+}, while a predominance of OH^- leads to formation of $FeOH^+$ which helps repair the oxide passivity layer (Richardson, 2002). Presence of chloride ions is associated with pitting corrosion, which takes place due to a local breakdown of the passive layer leading to a small anode region and a large cathodic region.

For chloride-induced corrosion, the rate of corrosion in RC structures is more severe in comparison to carbonation-induced corrosion due to:-

(a) The aggressive attack on the passivating layer by chloride ions; these ions are not consumed in the electrochemical reactions but react with Fe^{2+} to form $FeCl_2$ which undergoes hydrolysis, liberating the chloride and hydrogen ions. The effect of this at the anode is a rapid loss of Fe^{2+} due to a high amount of chloride ions, and lowered pH due to hydrogen ions while at the cathode the pH is raised due to formation of OH^-; the high potential difference between the two favours high corrosion rates (Kropp, 1995).

(b)Hygroscopic nature of chloride ions (tendency to absorb moisture) which results in high moisture levels within the concrete cover, reduced resistivity and a higher rate of corrosion in RC structures (Bentur et al., 1997)

The amount of chloride ions that are involved in chemical reactions is dependent on the binding capacity of the concrete. Binding of chloride ions may be through weak bonds due to physical adsorption on the surface of the hydration products or stronger chemical bonds where the ions react with calcium aluminates to form calcium chloroaluminate (Friedel's salt). In concrete that has undergone carbonation, the chemically bound chloride ions are liberated leading to an increase in the proportion of free chlorides (Bakker, 1988; Neville, 1995). A combination of carbonation and presence of chloride ions therefore leads to severe corrosion.

To determine total chloride ion content in a concrete sample, the acid soluble test is used (ASTM C 1152). The test can be used in determining the profile of chloride penetration in a concrete sample (chloride profiling) where a core is drilled at different depths to obtain dust samples that are tested to determine total chloride ion content. The proportion of chloride content is expressed as a percentage of either, the weight of cement where this is known or weight of concrete (Richardson, 2002).

2.2.2 Penetrability of concrete

Deterioration in concrete is due to ingress of water, in a pure form or containing aggressive ions (Neville, 2007). The degree to which a material permits transport through it is defined as penetrability and it determines the durability of a structure (Alexander and Mindess, 2005). Penetrability of concrete is influenced by size, distribution, connectivity and continuity of pores within the hardened cement paste (Garboczi, 1995). A transport mechanism is required for the movement of these aggressive substances from the environment, into and through the concrete.

Transport mechanisms in concrete are highly dependent on the moisture conditions in the pores. Hearn and Figg (2001) describe the influence of moisture levels in concrete pores on transport mechanisms as illustrated in Figure 2.3.

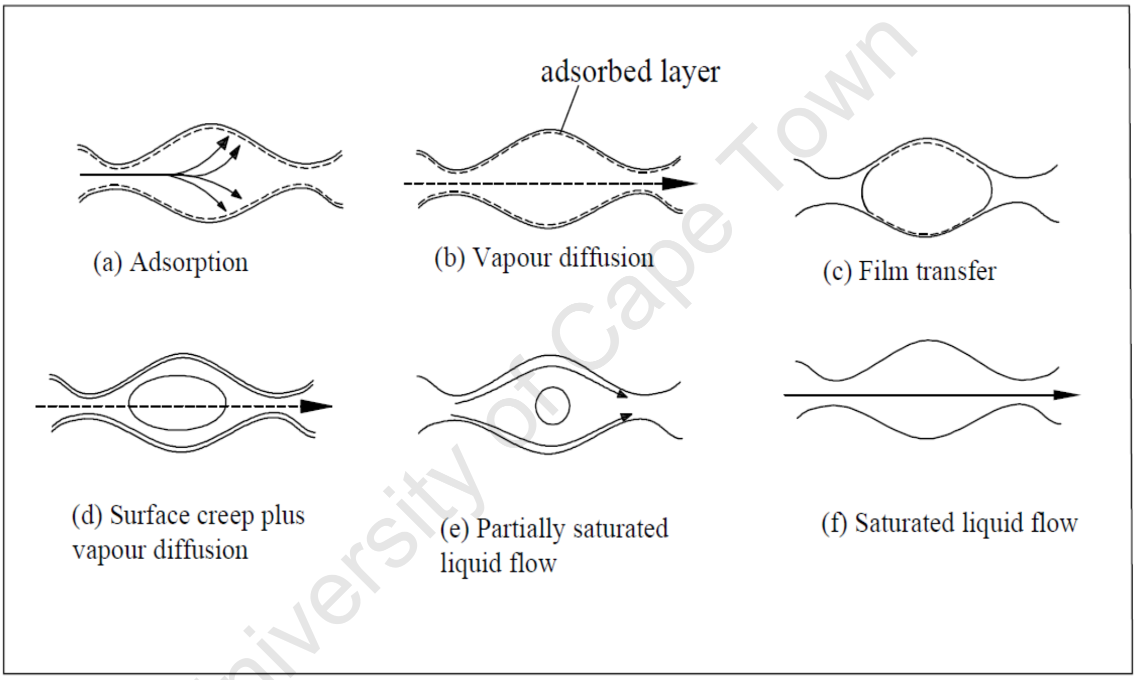

Figure 2.3: Schematic description of influence of moisture conditions on transport mechanisms in concrete pores (Hearn and Figg, 2001).

The influence of moisture levels on transport mechanisms was described as follows: at low moisture levels approaching zero, moisture is adsorbed on the surface of the capillary pores which are highly hydrophilic; as the moisture levels increase, transport mechanisms change to vapour diffusion; then to liquid assisted vapour transfer that involves formation of menisci and at high moisture levels, saturated liquid transfer.

The subsequent sub-section contains a review of transport mechanisms that take place in concrete structures, factors influencing the penetrability of site concrete structures and methods used in measuring transport mechanisms in concrete structures.

2.2.2.1 Transport Mechanisms in Concrete

Transport mechanisms involve the transfer of aggressive agents from the environment into and through the concrete, and occasionally transfer of substances from the concrete to the environment e.g. water vapour. A combination of different modes of transport may be involved in the movement of aggressive ions from the environment e.g. chloride ions are transported into concrete by both absorption and diffusion transport mechanisms (Kropp et al., 1995). The transport mechanisms in concrete are described below.

(i) Diffusion

Diffusion is the movement of molecules (gaseous and ionic) due to a concentration gradient. Gaseous diffusion involves the random movement of gases (e.g. CO_2, O_2) into concrete from the environment. The transport of gases into concrete is dependent on: moisture conditions where unsaturated or partially saturated conditions are favourable for diffusion of gases; the binding capacity of the cement matrix e.g. through carbonation influences rate of diffusion into concrete; temperature; and penetrability of hardened cement paste which is dependent on age of concrete and type of binders used (Richardson, 2002).

Fick's first law of diffusion is used to describe gaseous diffusion in concrete, Equation 2.6.

$$\frac{dc}{dt} = D\frac{dc}{dx} \tag{2.6}$$

Where x = Distance (m)
D = Diffusion coefficient (m^2/s)
c = Concentration (g/m^3)
t = time (s)

Ionic diffusion involves the movement of ions, in saturated pores, without the movement of water e.g. chloride ions are transported in pore solution by ionic diffusion. This form of diffusion is the most significant transport mechanism that determines the rate of physical and chemical deterioration (Hearn and Figg, 2001).

To describe ionic diffusion, Fick's second law of diffusion is applied, Equation 2.7. The equation considers the rate of change of concentration with time as the diffusion occurs along a concentration gradient.

$$\frac{dc}{dt} = D\frac{d}{dx}\left(\frac{dc}{dx}\right) \tag{2.7}$$

Where x = Distance (m)
D = Diffusion coefficient (m^2/s)
c = Concentration (g/m^3)
t = time (s)

(ii) Permeation

Permeation is a measure of rate of flow of a fluid (gas or liquid) when a pressure is applied, and the pores are already saturated with that fluid (Claisse, 2005). An example of permeation is the movement of water into the concrete pores in water retaining structures. It depends on viscosity of the fluid, temperature and pore structure of the concrete (Kropp et al., 1995).

The rate of flow of a fluid (liquid or gas) in pores is sufficiently slow and is considered laminar; hence Darcy's law, given in Equation 2.8, is used to describe permeation (Domone, 1994).

$$U_x = -K \frac{dh}{dx} \tag{2.8}$$

where,

U_x = mean flow velocity (m/s)

$\frac{dh}{dx}$ = rate of change in pressure head in x-direction

K = Coefficient of permeability (m/s)

In testing the permeability of concrete to water under an applied head of pressure, Equation 2.9 based on Darcy's law is used.

$$\frac{dq}{dt} = K_w A \frac{\Delta h}{L} \tag{2.9}$$

where,

$\frac{dq}{dt}$ = rate of flow (m^3/s)

K_w = Coefficient of water permeability (m/s)

A = Cross-sectional area of sample (m^2)

Δh = Drop in hydraulic head across the sample

L = Thickness of specimen (m)

In determining the permeability of a gas, a compressible fluid, the viscosity of the gas must be considered. In this case the Hagen-Poiseuille relationship for laminar flow is used to determine coefficient of permeability as is given in Equation 2.10 (Kropp et al., 1995).

$$K_g = \eta \frac{Ql}{tA} \frac{2P}{(p_1^2 - p_2^2)} \tag{2.10}$$

where,

K_g = Coefficient of air permeability

η = Viscosity of the gas

Q = Volume of air flowing (m^3)

l = Length penetrated by air (m)

A = Penetrated area

P = pressure at which Q is measured (N/m^2)

p_1 = pressure at entry of gas (N/m^2)

p_2 = Pressure at exit of gas (N/m^2)

t = time (s)

(iii) Absorption

Absorption is the rate of uptake of fluids by a solid through capillary action. The absorption properties of a solid can be determined by sorptivity and porosity measurements. Sorptivity is the rate of advance of a wetting front into concrete from the surface. However porosity provides a measure of the total volume of pores in a concrete sample that can be filled with water; it is obtained by considering the difference in mass of an oven dried and saturated sample of concrete. A measure of sorptivity is more suitable in evaluating the durability properties of concrete as it provides an indication of the rate of water uptake by the near surface concrete (Neville, 2007). The sorptivity is dependent on the density and viscosity of the fluid, radius and continuity of pores, and degree of saturation in pores.

Non-steady absorption of water takes place when a specimen is in contact with water for a short time and transport into concrete is a function of time (Kropp et al., 1995). An equation that describes the rate of water uptake by a solid is given in Equation 2.11.

$$a = \frac{\Delta m}{Af(t^n)} \qquad (2.11)$$

where

a = rate of water uptake ($g/m^2\ s^n$): s is time in seconds

A = Area of the specimen (m^2)

$f(t^n)$ = time function

The equation that is widely applied in determining sorptivity is that developed by (Hall, 1989), given in Equation 2.12.

$$i = St^{0.5} \qquad (2.12)$$

where,

i = cumulative water absorbed per unit area of surface inflow ($g/m^2\ s^{0.5}$)

S = Sorptivity

Other forms of transport mechanisms in concrete are:-

(iv) Wick action: This involves the transport of water from wetted face of concrete element to the drying face. Transport mechanisms involved are a combination of water vapour diffusion and absorption (Buenfeld, 1997). This form of transport is present in concrete structures located in marine environments that are subject to cycles of wetting and drying.

(v) Migration: This involves the transport of ions in an electrolyte due to application of an electric current which accelerates the movement of ions (Kropp et al., 1995). The general law governing this transport mechanism is Nernst-Planck equation, given in Equation 2.13, which considers movement of ions by: diffusion (due to concentration gradient), migration (due to electric field) and convection (flow of electrolyte itself e.g. by permeation).

$$J = D\frac{dc}{dx} + \frac{zF}{RT}DC\frac{dE}{dx} + CV_e \qquad (2.13)$$

where,

J = mass flux (g/m^{2}s)
D = Diffusion coefficient (m2/s)
C = Concentration (g/m^3)
Z = electrical charge
F = Faraday constant (J/V.mol)
R = gas constant (J/mol.K)
T = absolute temperature (K)
E = electrical potential (V)
V_e = Velocity of solution (m/s)

2.2.2.2 Factors influencing penetrability of site concrete

The durability of reinforced concrete structures is dependent on penetrability of the concrete cover. Penetrability of the cover is influenced by micro-structural properties of concrete, execution of construction process (placing, compaction and curing) and exposure conditions (moisture and temperatures). The factors that influence penetrability of concrete structures are further discussed below.

Micro-structural properties of concrete

Concrete is a particulate composite that consists of a cementitious phase and a dispersed phase of aggregates (Young et al., 1998). The cement reacts with water (hydration reaction) to form a hardened cement paste that firmly binds the aggregates to form a rigid mass.

In the hydration reaction, water and cement form products (cement paste) that are approximately twice the volume of the constituents. The paste formed is such that it occupies a greater volume than absolute volume of unhydrated cement, but smaller than the sum of the volume of dry cement and non-evaporable water (Soroka, 1979; Neville, 2007). The voids resulting from this reaction are known as capillary pores; the amount of these pores and their interconnectivity influence the penetrability of the cement paste.

The amount of capillary pores formed is dependent on the water/cement ratio and the degree of hydration. Neville (2007) describes the influence of water/cement ratio in concrete on hydration and amount of capillary pores formed as follows: (a) at low water/cement ratios, lower than or equal to 0.38, incomplete hydration takes due to a limit in volume for expansion, which results in unhydrated cement and an absence of capillary pores; (b) at higher water/cement ratio, full hydration of cement takes place as there is sufficient water to hydrate all cement present in the system and sufficient volume for expansion of the products; a result of full hydration is presence of capillary pores which influence penetrability. The influence of water/cement ratio and degree of hydration, and resulting volume of capillary pores, is illustrated in Figure 2.4.

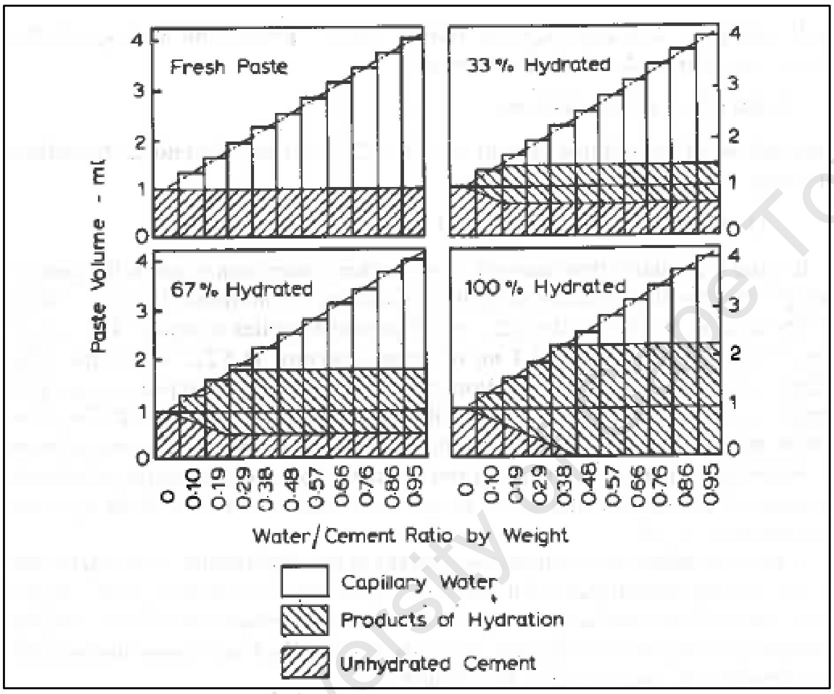

Figure 2.4: Composition of cement paste at different stages of hydration (Neville, 2007).

The micro-structural properties of concrete may be improved through the use of supplementary cementing materials, latent binders and pozzolanic materials. Latent binders have cementitious properties but need to be activated through the addition of ordinary Portland cement for hydration reactions to occur. An example of such a binder is ground granulated blast furnace slag (GGBS), a by-product of iron production process. Pozzolanic materials contain silicates which react with calcium hydroxide to form substances with cementitious properties that fill up capillary pores resulting in reduced penetrability. Pozzolanic materials commonly used in concrete include fly ash, a by-product of the coal burning process and silica fume, a by-product from the manufacture of silicon alloys (Aïtcin, 2008).

Aggregates are mainly obtained from natural sources such as crushed rock and occupy the largest proportion in concrete, ranging from 70 – 80%. Aggregates are not inert fillers in concrete, as previously considered, but have properties that are both beneficial and detrimental to the fresh and hardened properties of concrete. Aggregates reduce the penetrability of concrete as they introduce winding paths (tortuosity) and have a 'dilution effect' due to their low penetrability (Young et al., 1998). The size of aggregates (coarse and fine) has an influence on the workability of concrete; a predominance of coarse aggregates will result in a 'harsh mix' which may be liable to segregation and bleeding while a large amount of fines results in a high water requirement and a 'sticky mix' that is difficult to work with (Popovics, 1982).

The phase formed between aggregates and hardened cement paste, known as the Interfacial Transition Zone (ITZ), influences durability of concrete as it is highly porous, providing easy avenues for ingress of aggressive substances that cause deterioration (Alexander and Mindess, 2005). The use of silica fume reduces the penetrability of the ITZ due to a high surface area (fine material) which has a fine filler effect.

Execution of construction practices

The handling of concrete in its fresh state and execution of construction practices (placing, compaction, finishing and curing) influences its properties in the hardened state (strength, durability, dimensional stability properties); it plays an important role in determining the quality of concrete structures constructed. The handling of concrete and execution of construction practices in the fresh state is dependent on its workability. Workability is defined as the amount of effort required to manipulate a freshly mixed quantity of concrete without loss in homogeneity (Neville, 2007). The workability of a concrete mix influences its: flowability- ability to flow into all corners of a mould and fill it; compactability; stability - ability to remain consistent; and finishability - ability to give a smooth finish without honeycombs or blowholes when concrete is placed in formwork, while for free surfaces ability to give good response to operations such as floating and trowelling (Tattersall, 1991) . Concrete should be properly compacted to ensure that air pockets are expelled and aggregates are consolidated.

The cohesiveness of concrete is important to ensure that concrete remains homogenous. Loss of homogeneity occurs in the form of:-

(a) Bleeding which is the upward movement of water within the concrete; the negative effects of bleeding in concrete are: - (i) as the water migrates upwards it carries fine particles of sand and cement which are deposited on the surface leading to surface laitance. The high water content on the surface results in a poor quality surface that is easily penetrable. (ii)Formation of bleed channels as the bleed water moves upwards results in increased penetrability of concrete. (iii) As bleed water moves upwards, it may be trapped underneath coarse aggregates or reinforcement which results in a weak bond at the interface. Such areas of weakness within the concrete are highly penetrable (Popovics, 1982).

(b) Segregation which is the separation of the coarse aggregates from the mix where the heavier particles shift downwards. To improve consistency and cohesiveness of a concrete mix, amount of fines should be increased e.g. through use of fine sands or supplementary materials such as silica fume.

The workability of concrete can be increased, while maintaining low levels of water, through the use of water reducing admixtures (plasticizers). These admixtures adhere to the surface of cement particles and prevent flocculation; as cement particles are dispersed, trapped water is freed and lubricates the mix increasing workability (Young et al., 1998).

Curing is defined as those procedures that promote hydration of cement by preventing loss of moisture in concrete, in addition to control of temperatures (Neville, 2007). The main aim of curing is to maintain the pores in concrete at saturated levels or near saturated as possible, to promote continued hydration and filling of pores. The two main forms of curing concrete structures are: - (a) Wet curing methods which involve a continual or intermittent supply of water to the concrete surface e.g. spraying of concrete with water, ponding, use of hessian cloth, wet sand. (b) Impermeable membranes where loss of water from the surface is prevented for example through the use of curing compounds which seal the surface of concrete or polythene sheets.

Curing is essential in concrete production as it minimizes the loss of water of near surface concrete and ensures its availability for continued hydration, which effectively reduces the amount of capillary pores that influence penetrability. The use of supplementary materials influences the duration of curing; longer curing periods, than those used for ordinary Portland cement, are required with the use of these materials as they hydrate slowly (Ramezanianpour and Malhotra, 1994).

In as much as the placing, compaction and curing are essential construction practices in the production of durable concrete structures, and guidelines are provided in project documents and standards on how to effectively carry them out, it is difficult to verify if these practices are properly executed on site which influences the penetrability of the near surface concrete of a finished structure (Newman, 2003).

Exposure conditions

The humidity in an environment influences the moisture level in concrete pores which has an effect on diffusion of gases, CO_2 and O_2. High moisture levels in concrete pores results in reduced penetrability due to slower rate of diffusion of gases (Richardson, 2002).

The temperatures in a given environment influence the rate of reactions; an increase in temperature accelerates reaction rates which results in increased severity of the deterioration process (CEB, 1992).

2.2.3 Methods of measuring transport properties of concrete

The durability of reinforced concrete, with regard to corrosion, is determined by the quality of the near surface concrete and its resistance to penetration of aggressive substances. The current approach in design and assessment of concrete quality is based on determination of strength using either cubes or cylindrical specimens. However strength is a bulk property and such measurements fail to consider the near surface properties of concrete, 'skin of concrete', which determines the durability of RC structures (Bentur and Mitchell, 2008).

To address this shortcoming, researchers have over the years developed methods of determining the penetrability properties of concrete by obtaining a measure of its permeability, absorption and diffusion. Some of the approaches developed are discussed in the subsequent section. The test methods discussed in this section are those developed in an international context; a review of the South African approach in determining penetrability properties of concrete cover, by the use of durability index tests, is discussed in Section 2.4.

(i) Permeation measurements

The tests used in determining the permeability of concrete can either be laboratory or site based. In this discussion the permeation measurements are broadly classified into three categories: drilled-hole methods, surface testing methods and core-testing methods.

Drilled-hole methods

These tests involve drilling a hole into the surface of a concrete structure or a cube and measuring the rate of decrease in pressure applied or vacuum with time to determine permeation characteristics of samples or concrete structures.

The first drilled hole test method was the Figg's air permeability test developed by Figg (1973). The test involved drilling a hole 30 mm deep with a 5.5 mm diameter onto the surface of a concrete structure. The permeability was determined by observing the time taken for pressure to increase from 15 kN/m^2 to 20 kN/m^2. However the test was found to have high variability and from recommendations by Cather et al. (1984) improvements were made. The improvements included increase in diameter of drilled hole to 10 mm, increase of depth to 40 mm and use of higher pressure range from 50 to 55 kN/m^2. This test set up is illustrated in Figure 2.5.

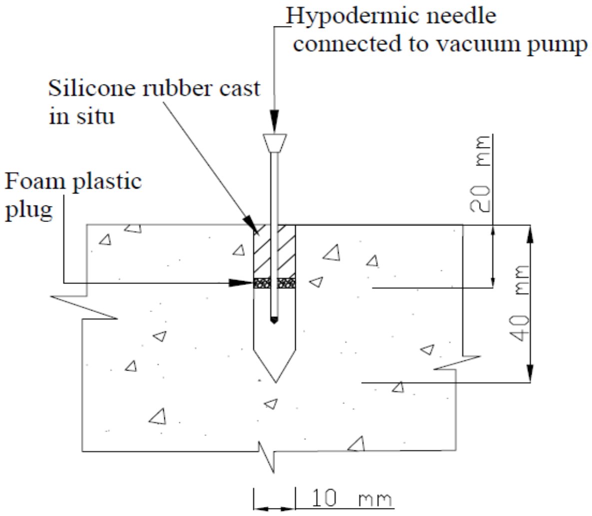

Figure 2.5: Schematic representation of Figg's air permeability test (Cather et al., 1984).

An air permeability test developed by Parrott and Hong (1991) and similar test to the Figg’s drilled hole method where a cavity 20 mm in diameter and 35 mm deep was drilled into the concrete. The permeation was determined by observing time taken for pressure to fall from 50kPa to 35kPa. Further modifications have taken place for in-situ drilled hole methods, e.g. Neves and Goncalves (2006) seal the hole using a steel plug which is connected to a Poroscope that enables one to obtain automated readings of the time taken for vacuum to decay.

Drilled hole methods carried out using cubes include that developed by Martin (1986) which involves the use of a 100 mm cube with a cast in hole of 5 mm through its centre. An initial pressure was applied and time taken for pressure to fall as gas permeated through the cube is recorded in a multi-channel data logger; from this, plots of pressure decay against time were made. A similar test, the overpressure method developed by Dinku and Reinhardt (1997) involved the use of a 150 mm cube. A hole 45 mm deep and 14 mm in diameter is drilled on the surface of a cube and gas applied at a high pressure. The rate of decay of pressure can be used to determine permeability of concrete either: qualitatively by observing time taken for pressure to decrease over a pre-determined interval; or quantitatively by applying Hagen-Pouiseuille’s equation to determine coefficient of permeability.

In application of drilled-hole test methods, some limitations are encountered. Firstly, it is difficult to determine geometry of flow of the gas. For example in the Figg’s air permeability test it is difficult to determine the area of concrete that is influenced by the air flow or decay of vacuum. Gas flow does not only influence the drilled area in concrete but spreads out at a given distance from the test area. This area is difficult to determine and evaluate in making computations of air permeability. Dinku and Reinhardt (1997) also identify this limitation in determining the values of area and length of the sample affected by air flow and resort to using approximate values. Claisse et al. (2003) approach to this limitation is modification of the Figg’s test by use of the three hole test method where additional holes are used to compute a distance x influenced by air flow; this value of x is applied in permeability coefficient computations. However, computations used to determine the coefficient permeability are complex which may make it difficult to apply this test in routine testing of concrete for permeability.

Secondly, the flow of air into the hole is influenced by the nature of aggregates surrounding the hole. Presence of large aggregates adjacent to this hole may result in a reduction of the air flowing into the hole and thus influence the permeability measurements obtained.

Thirdly, for the in-situ drilled hole methods, the drilling action may create cracks within concrete which increases the permeability measurements.

Surface test methods

These tests involve the determination of permeation properties of in-situ concrete and are generally non-destructive in nature.

The Schoenlin permeability test was developed by Schoenlin and Hilsdorf (1987). The test apparatus consists of a 50 mm diameter vacuum chamber which is connected to a vacuum pump and a digital pressure gauge; the test set-up is illustrated in Figure 2.6. The pressure in the chamber is reduced and time taken for pressure to increase from 50 mbar to 300 mbar gives a measure of permeability.

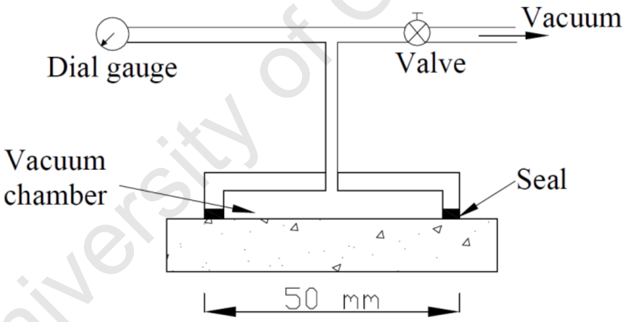

Figure 2.6: Schematic representation of Schonlin permeability test (Schoenlin and Hilsdorf, 1987).

Basheer et al. (1994) developed the Autoclam Permeability test which is used to determine both the air and water permeability of concrete. The test apparatus comprises of a base ring which isolates a test area of 50 mm diameter. The base ring is bound to the test surface using epoxy resin onto which the body of the apparatus is placed and firmly secured using mounting screws. The test set up is illustrated in Figure 2.7. Pressure is applied and the rate of decay is observed over a period of fifteen minutes or until pressure drops to zero. Permeability index is then determined as the slope of a plot of the natural logarithm of pressure against time.

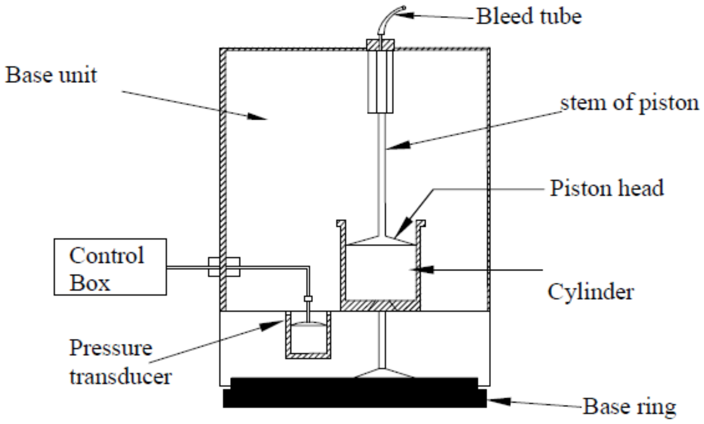

Figure 2.7: Autoclam permeability test set-up (Basheer et al., 1994).

Torrent's two-chamber vacuum cell is based on a guard ring principle and consists of two chambers - an inner and outer chamber (Torrent, 1992); the test set up is illustrated in Figure 2.8. The inner chamber is subjected to a unidirectional flow of air with any excess air being channelled out through the outer chamber. The change in pressure in the inner chamber is recorded using a pressure meter and a plot of the increase in air made by a graphic recorder or data logger. Computations are then made to determine the coefficient of permeability (k).

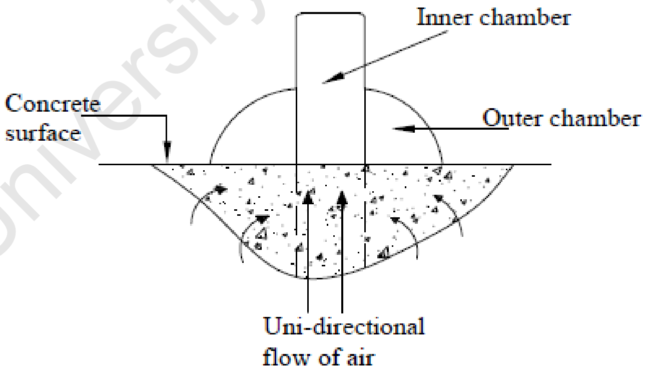

Figure 2.8: Schematic representation of Torrent's test set up (Torrent, 1992).

Core sample methods

These tests involve testing of concrete core samples, obtained from structural elements or cubes, in the laboratory.

Cabrera and Lynsdale (1988) developed the Leed's Permeability cell test which involved testing of samples with diameter of 25 mm and thickness ranging from 10 to 50 mm. The samples were pre-conditioned by oven drying at 105ºC for a period of 24 - 36 hours. Test samples are held in a permeability cell within a rubber silicon cylinder which is inserted into a stainless steel cylinder. An upper cap which consists of the pressure gauge and gas inlet is placed on the permeability cell. The rate of flow of gas at a given pressure is measured using a bubble flow meter to determine permeability.

The Cembureau method developed by Kollek (1989) involves the determination of permeability of a concrete sample 150 mm diameter and 50 mm thick, by obtaining a measure of rate of flow of oxygen gas. The specimen is fitted into a rubber collar which is then inserted into a cell made of steel, aluminium or plastic clamped together with bolts; test set-up is illustrated in Figure 2.9. Pre-conditioning of specimen may be carried out either at 20ºC and relative humidity of 65% for 28 days or oven dried at 105ºC for 7 days. Pressure is applied over a range of 1.5 to 3.5 bars; increase in pressure is carried out at intervals of 0.5 bars. The flow rate at each pressure stage is noted and an average of the K values used in determining the permeability of the specimen. From a comparative study on different test methods for permeability by RILEM TC 116-PCD (1999), this test was found to be very reliable with good repeatability and easy to handle.

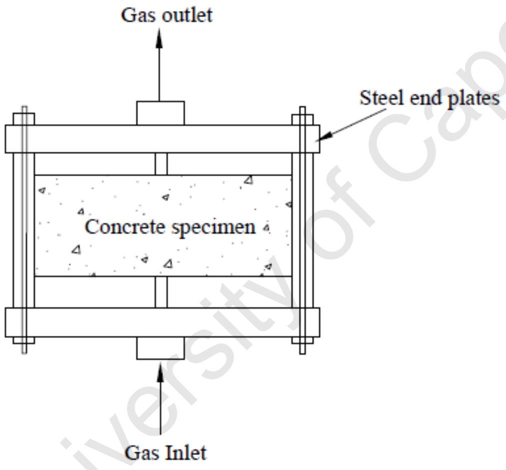

Figure 2.9: Permeability test set up for Cembureau method (Kollek, 1989).

The permeability tests discussed in the preceding section were developed on different principles, some tests measure the rate of decay of a vacuum (e.g. Figg’s test, Torrent’s test) while others measure the rate of decay of an applied pressure (e.g. Leeds permeability cell test). These tests are beneficial as they can be used in control testing where concrete mixes are designed and tested to determine their permeation properties. The tests have also been proposed for use in performance control for example the overpressure method developed by Dinku and Reinhardt (1997) where a minimum value of coefficient of permeability has been proposed for use in performance control.

The main limitation in application of the permeability tests (drilled-hole methods, surface tests and core sample methods) is the standardization of moisture conditions. The moisture in concrete influences its permeation properties where high moisture content within concrete, which is highly permeable, could lead to low permeability test values which are misleading. To ensure reliable results are obtained, steady moisture conditions should be established within samples. This is however difficult to achieve, especially for concrete tested on site. For laboratory prepared samples, the samples may be pre-conditioned by oven drying. However, a conflict exists on the temperatures to use and the duration of oven drying; high temperatures in the range of 105ºC could lead to microcracks which results in high permeability values that may provide misleading permeability values whereas lower temperatures could result in long drying periods of concrete samples.

(ii) Sorptivity measurements

The determination of rate of water uptake by a sample of concrete is essential to determine its durability properties. Hall (1989) argues that sorptivity measurements are more representative of near cover penetrability properties than permeation measurements as concrete pores are not in a saturated state on site. In determining sorptivity of concrete, the important aspect to consider is the rate of water uptake by a sample of concrete and not the total amount of water absorbed (porosity); however, most tests measure the two aspects.

Tests used to measure sorptivity can be either in situ or laboratory-based, and are briefly reviewed below.

The Initial Surface absorption test (ISAT), described in BS 1881: Part 5 but now out-dated, involved fixing a gasket cap on the test surface through which an inlet tube and outlet tube were connected. Water from a reservoir, with an applied head of 200 mm, flowed into the concrete from the inlet tube. The outlet tube is connected to a capillary tube which takes measurements by observing the distance moved by the meniscus. Measurements are taken at time intervals of 10, 30, 60 and 120 minutes. The Figg's drilled hole test was also used to determine permeability where an applied head of 100 mm is used. Limitations were encountered in application of the two tests described e.g. in the ISAT readings obtained were influenced by surface treatments such as curing membranes , while high variability was encountered in application of the Figg's test.

To address the limitations in the two tests above, Dhir et al. (1987) developed the Covercrete absorption test (CAT). The test involves use of a 100 mm diameter concrete core with a hole (50 mm deep and 13mm wide) drilled on the surface. A gasket cap was then placed over the hole through which inlet and outlet tubes were connected; the test set up is illustrated in Figure 2.10. A measure of water absorption is obtained as the movement of meniscus in capillary tube between 10 and 11 minutes after water first comes into contact with concrete.

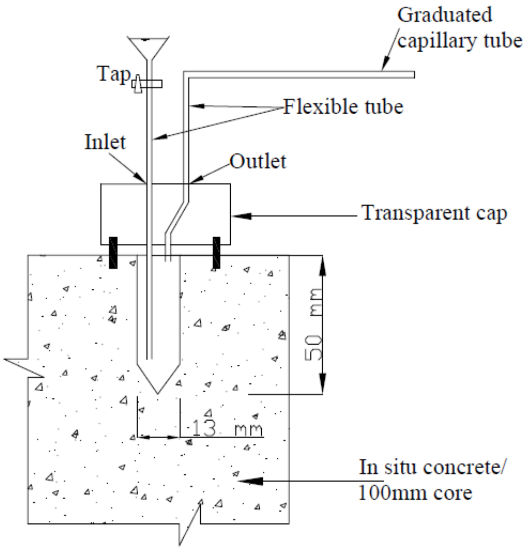

Figure 2.10: The Covercrete absorption test (CAT) test set-up (Dhir et al., 1987).

Majority of the water absorption (sorptivity) tests carried out involve subjection of a concrete sample to unidirectional absorption of water and recording the mass of water absorbed over a given period. The sorptivity of samples tested are then obtained from the slope of a plot of mass gain against time. In addition to obtaining a measure of sorptivity, porosity can also be determined by obtaining the difference of oven dried and saturated mass of specimen.

The sorptivity test developed by Kelham (1988) involved the use of a concrete sample 150 mm in diameter and 50 mm thick. The circular edges are sealed by applying bitumen and covering with waterproof adhesive tape; the test set-up is illustrated in Figure 2.11. Readings of mass gain are taken until specimen is near saturated conditions; this is indicated by small increments in mass gained, as recorded in data logger. The porosity of the sample is determined by saturating the entire specimen.

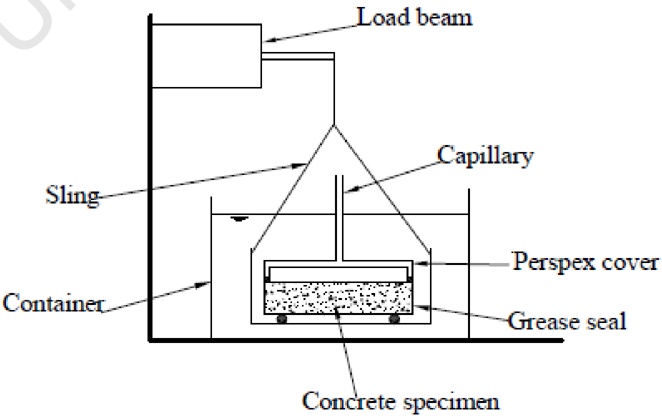

Figure 2.11: Test set up of water absorption test (Kelham, 1988).

A similar test was developed by Sabir et al. (1998) where a test specimen – 52 mm in diameter and 15 mm thick – is suspended over a reservoir of water such that the test surface just makes contact with water; the test set up is illustrated in Figure 2.12. The gain in mass is recorded over a total period of one hour, where readings are taken at intervals one minute. The porosity of the sample is determined by re-conditioning test sample by placing in an oven at 105ºC for a period of 24 hours, then saturating sample in water for 24 hours.

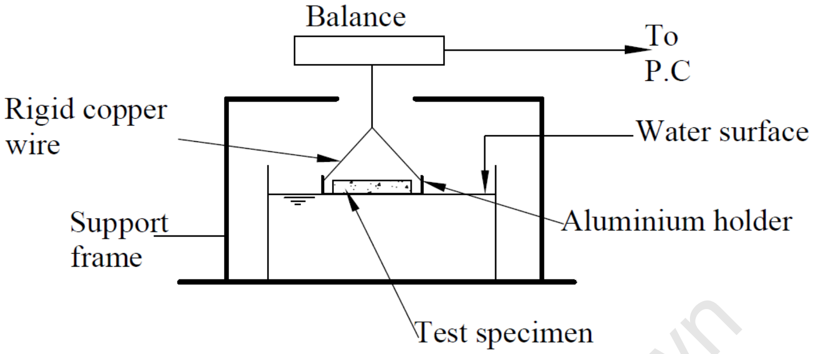

Figure 2.12: Test set-up for water sorptivity (Sabir, et al. 1998).

Parrott (1992) developed a sorptivity test carried out using 100 mm cubes. The test involves sealing five faces of the cube using bituminous waterproofing coat while the exposed face is placed on a wire grid that is 1 mm below a water surface. A record of mass gained by the specimen is made after 1, 2, 4, 6, 24 and 30 hours of wetting. Dias (2004) developed a sorptivity test carried out using a specimen 95 mm in diameter and 25 mm thick. The specimen is pre-conditioned by oven drying at 50ºC for 3 days and sealing its circular edges using paraffin wax. The test surface is exposed to a shallow tray of water; mass of water absorbed is determined at periods of 0.25, 1, 4, 9, 77, 125 and 144 hours.

A standard test, ASTM C 1585-04 is used in the determining the rate of absorption of water by samples with 100 mm diameter and 50 mm thick. The circular edges are sealed using a suitable sealing material such as epoxy paint. The test is carried out over a total period of seven days. The sample is placed on support devices such as rods or several layers of blotting paper in a pan; the test set-up is illustrated in Figure 2.13. Water is added to a depth of 1 – 3 mm above the top of support device and the mass of water absorbed at different time intervals recorded.

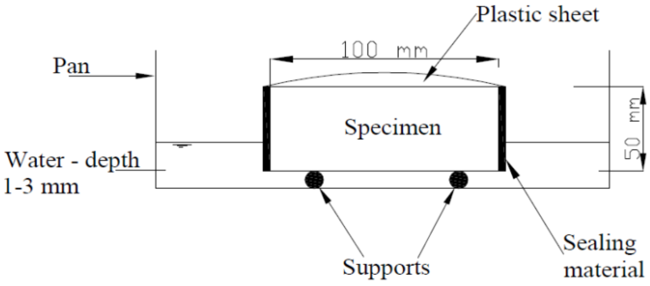

Figure 2.13: Water absorption test set-up (ASTM C 1584-04).

The Autoclam test set up can also be used to determine sorptivity. The test involves filling of the apparatus with water using the priming valve. The piston is pushed downwards to raise the pressure to 0.02 bars; this pressure is maintained by advancing the piston. Movement of the piston is monitored every minute for fifteen minutes. By considering the movement of the piston and area of the cylinder, mass of water absorbed is determined. A plot of mass of water absorbed against the square root of time is used to determine sorptivity of a concrete sample.

The laboratory sorptivity tests are all based on the same principle of subjecting a test specimen to a unidirectional flow of water. The difference in tests mainly arises from pre-conditioning of test specimen to attain uniform moisture conditions and duration of obtaining measurements of mass of water gained where some tests are carried out over a long duration, which would reduce their effective implementation in quality control.

Limitations of in situ sorptivity tests are: the flow of water into concrete takes place in form of stream lines which flow laterally as wetting front advances into concrete; due to this a representative value of sorptivity of concrete is not obtained. In addition, moisture conditions on site are not standardized which influences sorptivity measurements obtained.

(iii) Diffusion measurements

Diffusion is the movement of ions from a region of high to low concentration and is the main mode of ingress of absorbed chloride ions in concrete. Tests used to measure diffusion in the laboratory can either be: classical diffusion tests which are conducted over a long duration e.g. bulk diffusion test which takes at least 35 days; or accelerated tests, carried out within a short period and involve the application of voltage to accelerate movement of ions in concrete samples.

A review of diffusion tests that have been developed over the years is given below. These tests are mainly laboratory based and are carried out under non-steady conditions where a concentration gradient exists.

The diffusion test developed by Page et al. (1981) utilizes a 3 mm thick cement paste specimen which is placed in a diffusion cell, illustrated in Figure 2.14. One compartment of the cell contains 1M sodium chloride while the other contains calcium hydroxide solution. The concentration of chloride ions in the second compartment is determined at intervals. The diffusion coefficient is then determined by applying Fick’s first law of diffusion. The limitations with this approach it that a cement paste specimen is used, which is not representative of concrete and the influence of aggregates on penetrability (diffusion).

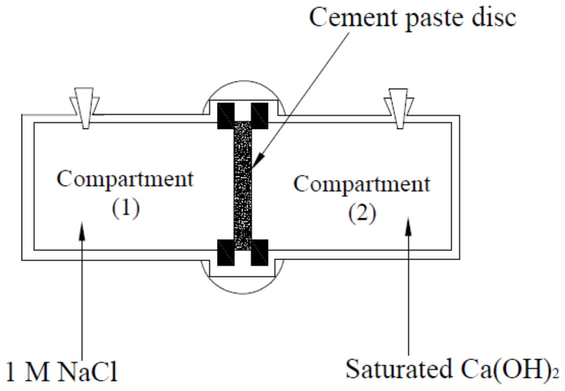

Figure 2.14: Diffusion test cell set up (Page et al., 1981).

The rapid chloride penetration test, ASTM C 1202, provides a measure of total charge passed through a concrete test specimen (102 mm and thickness of 51 mm) that is related to its resistance to chloride ion penetration; the test set-up is illustrated in Figure 2.15. One face of the concrete specimen is exposed to sodium chloride solution while the other face to sodium hydroxide solution. A direct voltage of 60 V is applied and a measurement of current taken every 30 minutes; the total duration of the test is 6 hours.

The limitations that arise are: test does not provide a measure of chloride diffusion but of amount of charge passed through a concrete sample, which is related to resistance to chloride ions; high voltage applied in the test results in high temperatures which influence the diffusion process.

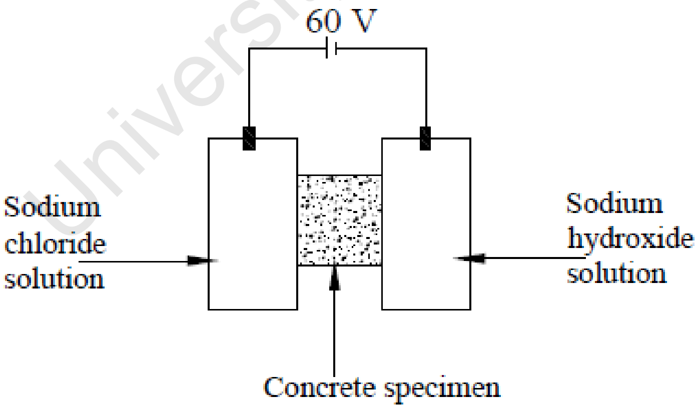

Figure 2.15: Schematic of test set up for rapid chloride penetration test (ASTM C 1202).

Luping and Nilsson (1992) developed a diffusivity test carried out using a concrete specimen 70 mm diameter and 50 mm thick; the test set up is illustrated in Figure 2.16. Before the test is commenced, the concrete specimen is soaked in saturated lime water for 24 hours. It is then placed on a plastic support and immersed in 3% sodium chloride dissolved in saturated lime water. A direct current of 30 V is applied for a given period. On removal of the concrete specimen, it is sliced into half; one half is tested for penetration depth using colorimetric method while the other half is sliced into smaller portions that are grinded and tested for chloride content. The diffusivity of the specimen is determined using Equation 2.14. This test is similar to that described in NT Build 492.

$$D = \frac{RT}{zFE}\frac{x_f}{t} \tag{2.14}$$

Where R, T, z, F, E are as defined in Equation 2.13.

x_f = Penetration depth

t = Duration of test

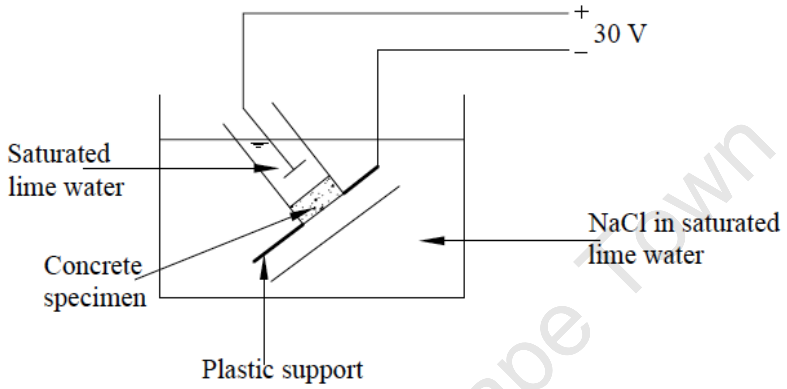

Figure 2.16: Test set up for rapid chloride penetration test (Luping and Nilsson, 1992).

The bulk diffusion test, described in ASTM C 1556-04 and NT Build 443, involves exposing one face of a concrete specimen to a solution of concentrated sodium chloride solution. This test is a conventional diffusion test where movement of chloride ions is only due to concentration gradient. The total exposure time for the specimen should be at least 35 days after which the apparent diffusion coefficient of a concrete specimen is determined by chloride profiling. A similar test that involves exposure of concrete specimen to chloride ions over an extended duration of time is ASTM C 1543-02 which involves setting up a dike on the perimeter of a slab specimen. After a period of 3 months, a core is drilled from the slab and apparent diffusion coefficient determined by chloride profiling.

Dhir et al. (1990) describe a test that provides a rapid estimation of chloride diffusion coefficient using a single sided diffusion cell; this is illustrated in Figure 2.17. In carrying out the test, the diffusion cell is filled with de-ionized water and placed in an immersion tank containing a solution of 5 M sodium chloride solution. A 10 V direct current is applied to accelerate movement of ions. The concentration difference in diffusion cell and immersion tank are measured, and using Fick’s first law of diffusion, the diffusion coefficient determined.

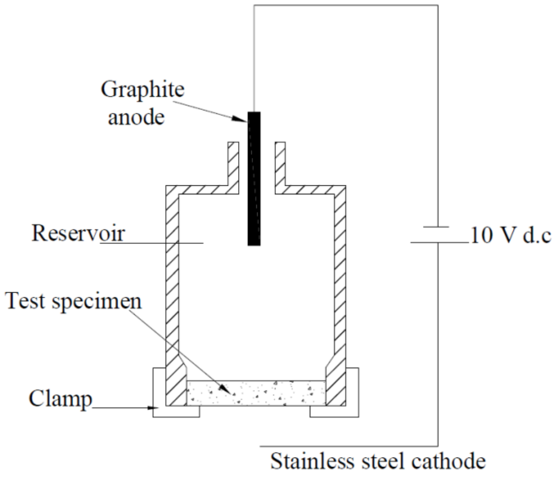

Figure 2.17: Diffusion cell used in rapid determination of chloride diffusion coefficient (Dhir et al., 1990).

The diffusion tests reviewed above are based on non-steady conditions where a concentration gradient exists in carrying out the test. To accelerate the tests, a potential difference may be applied where in addition to diffusion there is migration of ions through the concrete specimen. The duration of tests varies from 6 hours to longer test durations of up to 3 months.

The permeability, diffusion and sorptivity tests discussed in the preceding section are useful in design for durability and may be applied in two ways: design of concrete mixes which are characterized with the tests to determine penetrability properties; and control of concrete quality through assessment of penetrability of concrete cover of finished structures, new and existing. The routine application of tests on site is however limited due to difficulties encountered in practice such as standardization of moisture conditions, high variability, complex equations to determine penetrability properties and long durations of some tests which may make them ineffective to routinely apply for quality control (Ho and Lewis, 1988; Hooton et al., 2005; Andrade,2007).

2.3 DESIGN FOR DURABILITY

The design of concrete structures for durability requires that they remain functional for a given service life. Service life is defined as the period in which a structure should have adequate resistance to withstand the environmental actions that cause deterioration, ensuring that it delivers a particular level of performance without extending a pre-determined level of failure (Polder et al., 2006).

In the design of RC structures for durability, essential aspects that should be considered include the environment (exposure conditions) and form of deterioration that occurs in such an environment, penetrability properties of the concrete cover which are influenced by concrete mix constituents and cover depth thickness.

2.3.1 Approaches to durability design

The approaches used to design structures for durability are: (i) Outlining concrete mix requirements based on empirical relationships and past experience (ii) Accelerated testing and (iii) Mathematical modelling. These approaches are further described below.

(i) Empirical relationships and past experience

This approach is based on prescribing the mix constituents using specifications that limit the maximum water/cement (w/c) ratio, minimum cement content and concrete grade e.g. limiting values given in the U.K. complementary standard to EN 206-1, BS 8500-2 for service life of 50 or 100 years. The basis of these limiting values is laboratory and field tests, empirical relationships and past experience (Clifton, 1993). The use of National durability grades was proposed by Deacon and Dewar (1982) where relationships between concrete grade, cement content and water/cement content were established from mix design data obtained from a nationwide survey carried out in the U.K. By selecting a given strength grade of concrete, one could achieve the durability requirements of minimum cement content and maximum w/c ratio.

This approach does not provide reliable prediction of the service life of a structure as it is based on the assumptions that nature and performance of materials remain the same, environmental conditions remain constant and expectation of the service life remains similar (O'Brien et al., 1987).

These assumptions are however not valid as :- (a) the properties of cement have changed over the years with an increase in proportion of tri-calcium silicate (C_3S) and increased fineness of cement particles. This results in faster reactions and earlier gain in strength in comparison to older cements with which these limiting values were established (Pomeroy, 1986; Aitcin, 2000; Mehta and Burrows, 2001). (b) The environments in which concrete is used have become more severe e.g. increased use of concrete in marine environments, tropical climates and the Gulf regions where temperatures and humidity conditions are high, which accelerates the deterioration of concrete structures (Neville, 1987; Clifton, 1993; Idorn, 1997). (c) There is a growing need to increase the service life of structures from the conventional 50 years to 100 years and more, with the aim of efficient utilization of resources – both material and economic (Rostam, 1996).

(ii) Accelerated testing

This approach involves the use of elevated ‘actions’ (temperature, humidity, carbon dioxide concentration) to accelerate deterioration of concrete. The accelerated degradation mechanism used should be similar to that found in service. The short term tests are then correlated with long term in-service tests and from this an acceleration factor K, computed using Equation 2.15, can be determined and used in long term predictions (Clifton, 1993).

$$K = R_{AT}/R_{LT} \qquad (2.15)$$

where,

R_{AT} - rate of degradation in accelerated tests

R_{LT} - rate of degradation in long term in-service testing

The limitations in application of this approach are: firstly, the difficulty encountered by a designer in reliably selecting the exposure factors responsible for degradation in a given environment and thus accurately simulating deterioration mechanisms (Master, 1986). Secondly, there is lack of sufficient long term data which can be used to validate the relationship between short and long term performance. Thirdly, the Equation 2.15 that relates the short term accelerated tests and long term tests assumes that the two are linearly related, which in most cases is not true; due to this limitation it is preferable to consider the non-linearity by use of mathematical models.

(iii) Mathematical modelling

This approach to service life prediction and durability design is based on understanding the deterioration mechanism affecting a concrete structure and factors that influence the rate of deterioration - in this case the corrosion of RC structures. Corrosion is mainly influenced by transport of aggressive agents into and through the concrete cover. An understanding of the rate at which these transport mechanisms occur forms the basis of mathematical modelling (Clifton, 1993; Richardson, 2002).

The mathematical model applied to determine the carbonation depth for carbonation-induced corrosion is based on the fundamental diffusion relationship, as given in Equation 2.6. A mathematical model used in predicting the carbonation depth (X_c) at a given period of time has been developed in the DuraCrete model, given in Equation 2.16. (Richardson, 2002).

$$X_c = \sqrt{\frac{2K_1.K_2.D_{eff}.C_s}{a}}\sqrt{t}\left(\frac{t_o}{t}\right)^n \qquad (2.\ 16)$$

where,

K_1 = Constant related to execution, considers influence of curing on carbonation resistance

K_2 = Constant related to exposure that considers influence of relative humidity

D_{eff} = Effective diffusion coefficient (m^2/s)

C_s = Concentration of carbon dioxide in environment

a = chemical buffering capacity (estimates are obtained from literature)

t_o = Age at which D_{eff} is determined

t = Required service life of structure (years)

n = Constant related to environment

For chloride-induced corrosion, the mathematical model often applied is based on Crank's modified solution to Fick's second law of diffusion, given in Equation 2.17.

$$C_x = C_s\left(1 - erf\,\frac{x}{2(D_{ca}t)^{0.5}}\right) \tag{2.17}$$

where,

C_x = Chloride concentration at depth x and time t (% by mass of cement)

C_s = Surface chloride content (% by mass of cement)

D_{ca} = Apparent diffusion coefficient (m^2/s)

erf = error function

t = time of exposure (years)

The mathematical models given in Equations 2.16 and 2.17 above are selected depending on the exposure conditions and applied by a designer to rationally design for a given service life by determining suitable geometric properties (cover depth) and material properties (based on diffusion coefficients) for a structure. The cover depth and diffusion coefficient can be used as specifications for durability and verified for conformance using suitable test methods. This approach of mathematical modelling and testing for compliance with specifications forms the basis of the performance-based approach to durability design.

The design for durability using mathematical models may be considered in a probabilistic/ stochastic approach where the variability of parameters in models (cover depth, diffusion coefficients, environmental factors) are considered, such that in addition to average values, the distribution of these parameters are also considered. This approach increases the reliability of service life design (Clifton, 1993; Sarja and Vesikari, 1996; Richardson, 2002).

Limit state approach to durability design

In ISO 13823 (2008(E)), a limit-state approach to design for durability is considered. Limit state is defined as that state beyond which a structure fails to satisfy the performance requirements. The limit states considered could be: ultimate limit state (ULS) which is associated with structural failure; serviceability limit state (SLS) associated with failure to meet serviceability requirements e.g. presence of cracks; and initiation limit state (ILS) where significant deterioration of the structure occurs e.g. a corrosion level of 10%, it precedes SLS and ULS. To check for durability of a structure, two formats are used: - (i) Service life format where the predicted service life (t_S) should be more than or equal to the specified design life (t_D). (ii) Limit state format where the resistance capacity of a structure should exceed the action effect (environmental loads).

Service life design

The *fib* bulletin 34 (2006) provides a service life design process to durability, illustrated in Figure 2.18.

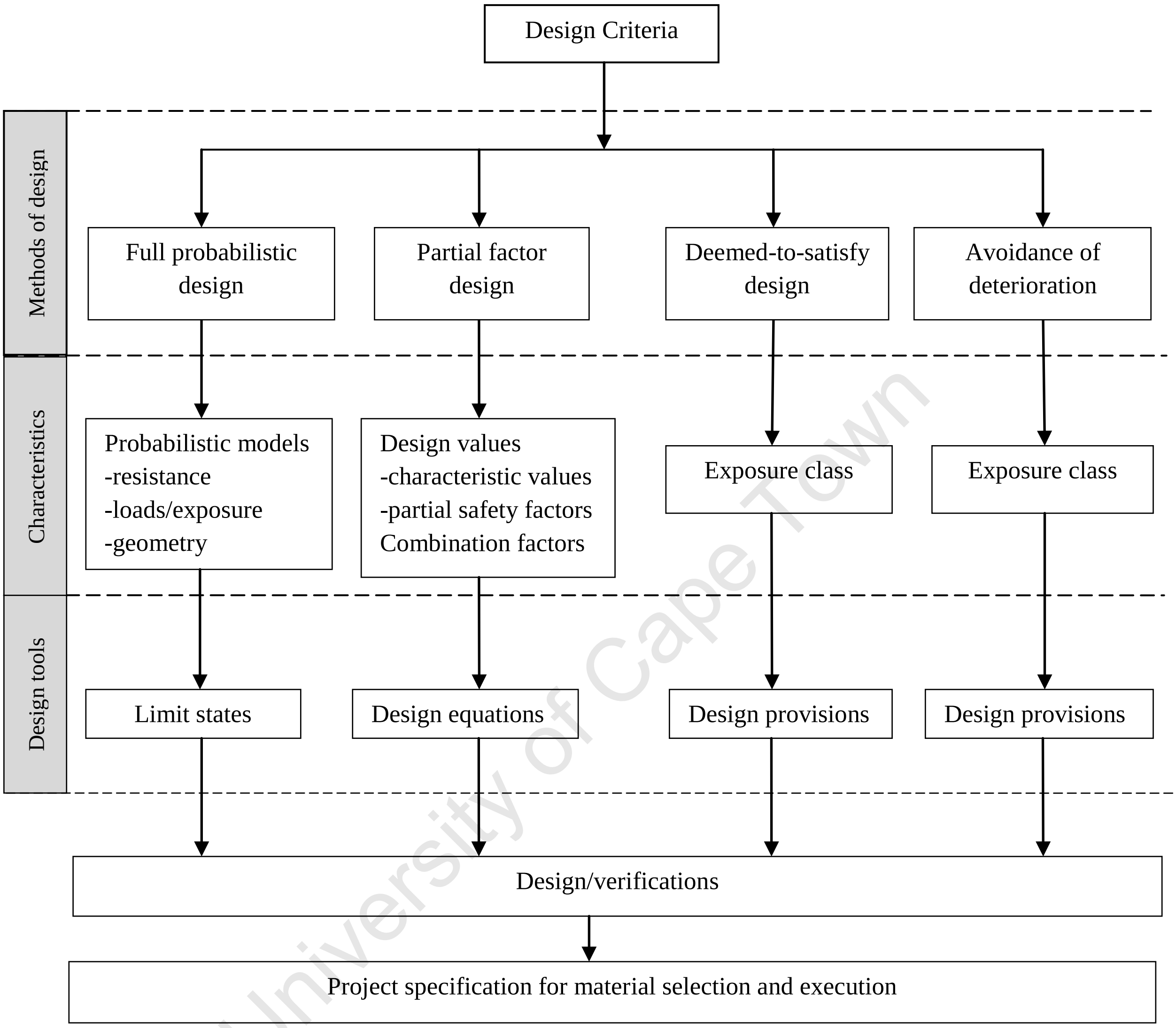

Figure 2.18: Flow chart of Service life design process (fib bulletin 34, 2006).

Four levels in the design approach have been developed which are:- (i) Full probabilistic design approach which involves application of models that have been validated to give realistic and representative values, with consideration of uncertainties (variability) in model parameters; (ii) Partial factor design approach where partial factors are used to consider variability in action values, materials and geometric aspects; (iii) Deemed-to-satisfy approach that entails a set of rules for dimensioning, execution and material selection which ensure relevant limit state is not exceeded. These rules (values) are based on statistical evaluation of experimental data and field observations (full probabilistic approach) or from calibration to a long term building experience ; and (iv) Avoidance of deterioration approach that considers conditions that will reduce possibility of deterioration taking place e.g. through the use of non-reacting materials like stainless steel.

From the design approaches reviewed in the preceding section, the use of limiting values and measure of strength based on empirical relationships and past experience is the approach currently used. The mathematical approach is preferable as it is based on a rational basis of considering deterioration mechanisms and rates at which they occur. However, application of this approach is limited due to complexity of service life models and lack of reliable, consistent and easily applicable performance tests.

2.3.2 Critical review of provisions in Current Standards for Durability

The durability of a RC structure is dependent on the environment in which it is located and resistance of concrete cover, which acts as a protective layer to reinforcement, to ingress of aggressive substances in this environment. The protective nature of the concrete cover is dependent on: its penetrability which in influenced by concrete mix properties, execution of construction practices such as curing and compaction; and actual thickness of concrete cover of the finished structure.

The provisions for durability in current South African, American and European approach are compared and critiqued in the ensuing section. The aspects considered are: classification of structures in exposure classes, mix constituent provisions, execution of construction processes, cover depth requirements and quality control measures.

Environmental classification

In a given environment, physical and chemical actions exist that cause deterioration of concrete structures. The initial step in designing a structure for durability is to consider the environment in which it is located and the form of deterioration that takes place in this environment. Richardson (2002) describes the evolution of durability provisions with regards to exposure classification in the European standards from an all-encompassing approach that was based on qualitative description of the environment to the current deterioration specific approach as provided in EN 206-1. This standard provides a total of eighteen exposure classes which provides adequate guidance to a designer in selection of environment for a structure.

The South African approach, provided in SANS 10100-2, to exposure classification is qualitative in nature where exposure conditions are described as mild, moderate etc. The main limitation of such a classification is that is subjective and may be ambiguous. The American approach, as provided in ACI 318M-08, considers the severity of environment and has three classes for RC structures dependent on severity of environment; this standard however only considers chloride-induced corrosion, and fails to consider carbonation-induced corrosion. A summary of exposure class provisions in the standards is given in Table 2.1.

Table 2.1: Exposure classification as provided in standards

Standard	Category	Condition
SANS 10100-2	Mild	Exposed to unpolluted air e.g. indoors
	Moderate	Sheltered from severe rain, buried in non-aggressive soil
	Severe	Wet conditions, water mildly aggressive, salt laden air (marine areas)
	Very severe	Highly corrosive fumes, abrasive action under wet conditions
	Extreme	Wet conditions with water highly aggressive
EN 206-1	Carbonation-induced corrosion	XC1 - XC4: Condition dependent on extent of exposure to moisture conditions
	Chloride-induced corrosion	XS1 - XS3: Source of chlorides from sea water. Conditions dependent on distance of structure from sea water
		XD1 - XD3: Source of chlorides other than from sea water e.g. industrial waters, deicing salts
ACI 318M-08	Corrosion protection	
	C0	Concrete dry/ protected from moisture
	C1: Moderate	Concrete exposed to moisture but not external source of chlorides
	C2: Severe	Concrete exposed to moisture and an external source of chlorides e.g. sea water, deicing chemicals

Concrete mix provisions

The properties of concrete are dependent on the suitability of constituent materials used (aggregates, cement, mixing water, admixtures etc). The provisions in the standards for material composition, with regards to durability, are based on limiting values for a maximum water/cement (w/c) ratio and minimum cement content in BS EN 206, while in ACI 318M-08 provisions are given with regards maximum w/c ratio. The South African standard does not make provisions for limiting material values. These provisions relating to concrete structures exposed to chloride induced corrosion are given in Table 2.2.

Table 2.2: Concrete mix requirements for concrete structures exposed to chloride-induced corrosion

	EN 206-1			ACI 318M-08
	XS1	XS2	XS3	C2
Maximum w/c ratio	0.50	0.45	0.45	0.40
Minimum cement Content (kg/m^3)	300	320	340	-

The requirements of a maximum w/c ratio are important as it influences penetrability of concrete cover due to amount of capillary pores formed (Neville, 2007), while a minimum cement content is required to ensure that there is sufficient paste to fill voids in between aggregates and increase probability of attaining the required water/cement ratio (Harrison, 1990). However in as much as these values are provided in specifications, they are difficult to verify for compliance on site (Neville, 2001). In addition, these limiting values may not be valid for current applications as they are based on past experience and empirical relationships, which fail to consider changes that have occurred in constituent materials e.g. for cement there is rapid gain in strength due to increased fineness and higher C_3S content, compared to older cements (Aitcin, 2000; Mehta and Burrows, 2001).

The provisions in the standards limit materials that can be used in production of concrete e.g. it places limits on the maximum permissible proportion of supplementary binding materials, as illustrated in Table 2.3 for the ACI 318M-08 and EN 206 provisions.

Table 2.3: Maximum percentage permissible of pozzolans and latent hydraulic binders

	ACI 318M-08	EN 206-1
Fly ash	25	33
Slag	50	-
Silica fume	10	11

The standards however do not make provisions for new or unconventional building materials that could improve properties of concrete (Baroghel-Bouny, 2006; Dhir et al., 2008). This may limit the concrete producer's innovation and flexibility in the use of suitable, locally available materials which in addition to improving the concrete properties reduce cost of production through the use of cheaper materials (Lobo et al., 2005).

Execution of construction practices

The penetrability of concrete cover is largely influenced by execution of site practices, the placing, compaction and curing of concrete (Browne, 1986; Levitt, 1990; Bentur, 2008). Guidelines are provided in SANS 10100-2, EN 13670 and ACI 318M-08 on proper execution of these construction practices. For example, a comparison of the curing periods in the three standards is given in Table 2.4. The South Africa and European approach are similar where minimum curing duration is dependent on the ratio of strength development at an early age and at 28 days, while in ACI 318M-08 a minimum curing period is given.

Table 2.4: Minimum curing duration for concrete structures

SANS 10100-2	EN 206-1	ACI 318M-08
$f_{cm3}/f_{cm28} > 0.5$ minimum 7 days	$f_{cm2}/f_{cm28} > 0.5$ minimum 1 day	Maintain in moist conditions minimum 7 days

Although these construction practices are specified in a project and guidelines provided on their execution, there is no means to verify if they are properly executed which influences the resulting penetrability properties of concrete cover in finished structure.

Cover depth provisions

A minimum cover depth is required to ensure protection of embedded reinforcement in RC structures. The minimum cover depth provisions are: in South Africa, the exposure conditions and strength of concrete are considered (SANS 10100-1); in European standards, consideration of exposure conditions and structural class; the structural class is determined by consideration of working life of structure and strength (BS EN 1992-1 (2004)); in the US, cover depth is dependent on the thickness of reinforcement used (ACI 318M-08). Table 2.5 provides minimum cover depth provisions in these standards; for SANS 10100-2 a severe exposure condition is considered while for BS EN 1992, exposure conditions considered are for chloride-induced corrosion (source of chlorides from sea water, XS).

Table 2.5: Minimum cover depth provisions for durability

	SANS 10100 – 2			ACI 318M-08		EN 1992-1		
	Concrete class (MPa)			Bar size		Exposure class		
	30	40	50	≥ 20 mm	≤ 16 mm	XS1[#1]	XS2[#2]	XS3[#2]
Minimum Cover (mm)	N/A	60	50	50	40	45	50	55

[#1]Concrete class C30/C37: [#2]Concrete class C35/45

In SANS 10100-2, as the concrete class (strength) increases, the minimum cover depth provided decreases. In the ACI 318M-08, minimum cover depth decreases with reduced bar size, while in the EN 1992-1, as the severity of exposure class increases the cover depth provided increases, with a corresponding increase in strength.

Quality control

The assessment of quality of structures in the three standards considered is based on measurement of strength on cube specimen for the South Africa and European approach, and on cylindrical specimen for the American approach. Limiting values are provided on compressive strength which if attained, concrete is 'deemed-to-satisfy' durability requirements.

The main limitation encountered with application of this approach in current standards is the compliance criteria used to verify durability requirements – strength measurements. Strength is an inadequate compliance criterion and does not guarantee durability; it is based on bulk properties and fails to consider the near surface properties of concrete which are influenced by concrete mix composition (w/c ratio and minimum strength) and execution of construction practices (Neville, 1987;

Dhir et al., 1990).The lack of a reliable means to verify limiting material provisions, coupled with the changes that have taken place in cement over the years that have resulted in rapid development of strength, provides a lee-way for concrete producers to use lower proportions of cement (reducing their cost and increasing profits) as they are still able to meet the specifications of minimum strength requirements (Pomeroy, 1986; Wood, 1997; Bentur and Mitchell, 2008). A reduction in proportion of cement used however results in high water/cement ratio and increased penetrability of concrete, which compromises durability.

In addition, the responsibility for production of quality (durable) concrete in the current standards is not clearly defined. For example, in a project the structural designer writes specifications for the durability by selecting limiting values of concrete mix and minimum cover depth as provided in the standards. He/she then passes on these instructions to the concrete producer who designs and produces a mix that will attain specifications required e.g. maximum w/c ratio and slump class. The concrete mix so designed is then supplied to a contractor who is required to place, compact and cure concrete in accordance to project specifications. However, the concrete mix supplied may fail to satisfy the requirements of contractors e.g. lack of sufficient workability to enable proper placing and compacting, resulting in concrete that has voids and of poor quality. In such a case, the responsibility for production of low quality concrete should lie on the design consultant who specifies the concrete, yet the blame in most cases falls on the contractor (Levitt, 1990).

The provisions in standards discussed in the preceding section are described as ‘prescriptive’ where concrete mix requirements and guidelines on execution of construction practices are provided. However in spite of these provisions, which over the years have been made more stringent, the lack of durability of RC structures (a pervasive issue in the construction industry) persists. This raises the question as to whether the specifications for durability requirements are deficient or if widespread deterioration is due to failure in properly following the specifications (Skalny, 1987).

To address durability problems encountered some designers have adopted use of a high amount of cement in concrete which increases strength in addition to reduction of penetrability. However due to the high cement content, an increased incidence of thermal deformations that result in cracking on the surface takes place which reduces penetrability of concrete in the long term (Long et al., 2001).

2.3.3 Performance-based Approach to Durability Design

Performance based durability design is a rational approach that considers the deterioration mechanisms and requires the verification of performance parameters of concrete that influence durability (its penetrability). Somerville (1997) proposed a quantitative approach to design of structures for durability that is similar to that adopted in structural design. The performance-based approach, which he describes as an ‘engineering approach’ to durability design, should be based on consideration of five aspects: Quantification of ‘loads’ in an environment and the predominant deterioration mechanism; performance criteria of a structure – the service life; prediction models that consider the rate of deterioration; factors of safety that consider variability in environmental loads, precision of models; and presence of specifications and quality assurance systems that verify compliance with the required performance.

A shift from the current ‘prescriptive’ to ‘performance-based’ specifications has been proposed by Dhir et al. (2008), Hooton et al. (2005), Alexander et al. (2001), Day (2005), to mention but a few. Performance specifications are defined as a set of clear, measurable, achievable and enforceable instructions of the functional requirements of hardened concrete depending on its application (Lobo et al., 2005). These functional requirements may for example include strength, durability, and dimensional stability.

The features of performance specifications are: - (i) the functional requirements should be clearly defined to ensure that the parties involved in their implementation (concrete producers and contractors) do not interpret them differently (Hooton et al., 2006). (ii) The material requirements of concrete mix are not given but the concrete producer and contractor work together in the design of the concrete mix; this allows for flexibility in selection of materials and ensures that concrete produced and supplied will meet contractor’s requirements of workability allowing for proper placing and compaction (Bickely et al., 2006). (iii) Verification of compliance with performance specifications by using tests that are reliable, repeatable, accurate and easily applicable on site (Skalny and Idorn, 2004; Hooton et al., 2005). (iv) A means to enforce compliance with the specifications e.g. through the use of penalties when specifications are not met.

2.3.3.1 Developments in Performance-based approaches

The performance based approach to durability design has developed over the years with various researchers proposing different approaches to performance-based design, testing and specifications. The following section provides a review of performance-based approaches that have been proposed, and in some cases implemented in an international context.

Hooton et al. (2005) describe the performance specifications considered for use in Canada based on an end result specification (ERS) system. The responsibility for production of durable concrete is shared by the concrete supplier and contractor. The test used in assessing compliance with performance requirements is rapid chloride penetration test, ASTM C 1202, where limiting values are given for concrete at 56 days. A pre-qualification of concrete mix supplied is required to ensure that concrete producer attains the required performance; the responsibility of the producer ends on delivery of concrete. The contractor is then required to ensure proper handling of concrete supplied on site to ensure compliance with performance requirements. Payment to the contractor is based on attaining performance requirements - the end result. Failure to comply with specifications results in reduced payments through penalties.

Baroghel-Bouny (2006) proposes an approach to performance specifications in France through the use of universal durability indicators (DIs) that consider different transport mechanisms which control ingress of aggressive agents into concrete. The panel of universal DIs selected are used in obtaining measurements of initial calcium hydroxide content, porosity, chloride ion diffusion coefficients and permeability. In addition to use of DIs, complementary parameters (CPs) are obtained which though not mandatory, provide useful information in confirming or interpreting experimental results obtained from DIs e.g. determination of carbonation depth using phenolphthalein. The limiting values (threshold values) of the different parameters are determined from laboratory testing of a wide range of concrete mixes cured at 90 days or less; these values are then ranked on the basis of ‘potential’ durability with ranges from very low to very high. The specifications set limiting values on DIs for RC structures based on: environmental classification defined in EN 206-1 for chloride-induced and carbonation induced corrosion; the target service life; and minimum concrete cover thickness. As the severity of the environment and target service life of a structure increases, the number of parameters verified using the DIs increases.

Polder et al. (2006) describe the proposed method discussed in Netherlands for service life design. The penetration of chloride ions into RC structures is considered to be the limiting factor in service life and modeled using the DuraCrete model. The input parameters in the model that can be tested and verified for a given sample of concrete are the diffusion coefficient and cover depth. Determination of diffusion coefficient was carried out using the Rapid Chloride Migration test (RCM) (Luping and Nilsson, 1992) which was found to have a strong correlation with the conventional immersion test NTBuild 443. Limiting values of diffusion coefficient are given on the basis of service life of structure, binder type, exposure condition and minimum concrete cover thickness.

Torrent (2006) proposes a performance-based approach in Switzerland that considers the provisions in current standards of maximum water/cement ratio and cover depth as given in EN 206 and ACI 318. On selection of the maximum water/cement ratio for a given environment, an empirical equation proposed by the author is applied to convert the maximum w/c ratio into a value of gas permeability (K_g), which can be verified using the Torrent air permeability test. The cover depth and K_g are parameters that can be used in performance-based specifications and against which conformity can be determined.

Arskog et al. (2006) discuss the performance-based approach used in quality control of concrete structures for durability in Norway. The authors highlight the need to measure the quality of concrete of site elements so as to fully grasp the influence of construction quality and variability. A probabilistic approach to design is considered where a risk level of 10% for steel corrosion to occur within a given period is required. Testing of concrete elements was done using electrical resistivity, which is a rapid test making it suitable for quality control, to determine chloride diffusivity. The diffusivity was obtained from both laboratory prepared samples and site elements. To reduce the amount of coring of structures, samples were obtained from ‘reference’ or ‘dummy’ elements which are cast and cured in the same way as the actual structure. The measured values of cover depth and chloride diffusivity are then documented and used in a new risk analysis to provide a basis for evaluating potential durability of a structure.

Gjørv (2010) describes the implementation of this probabilistic based approach in construction of structures exposed to sea water. The cover depth and chloride diffusivity of this structures were determined using a service life prediction model for a structure with a design life of 150 years and probability of corrosion within this period was required to be less than 10%. A measure of the chloride diffusivity of the built structure was obtained and documented. The probability of risk to steel corrosion was re-computed using the documented data and found to be less than 10%. The documentation of durability attained by structures is required by the owners from contractors; it has been observed that from implementation of this approach workmanship has improved.

The performance-based approach provides a valid (rational) compliance criterion for use in design for durability as it is based on consideration of mathematical models where input parameters (material properties and thickness of concrete cover) are verified using suitable performance test methods. The approaches discussed in the preceding sections are those developed in an international context; the DI-based performance approach in South Africa is reviewed in the subsequent section.

2.4 DURABILITY INDEX-BASED PERFORMANCE APPROACH IN SOUTH AFRICA

Durability Index (DI) tests characterize near surface properties of concrete and give a measure of its potential resistance against fluid and ionic transport mechanisms that initiate and sustain the corrosion process (Alexander and Mackechnie, 2001). The DI tests are oxygen permeability index (OPI), water sorptivity index and chloride conductivity index (CCI). These tests were developed as: simple and quick tests that are easy to operate; applicable both on site elements and in laboratories; requiring minimum sample preparation; and conducted at an early age (typically 28 to 35 days) (Alexander, 2004).These aspects of the tests makes them suitable for use in quality control of concrete cover properties and assessing the quality of construction as the tests are sensitive to construction practices e.g. curing and compaction. Furthermore, the tests are useful in optimization of materials in a concrete mix (Alexander and Mackechnie, 2003) and in service life design where correlations have been established between index values and input parameters used in prediction models (Mackechnie, 1999; Mackechnie and Alexander, 2002).

This ensuing section will provide a review of: - (i) the durability index tests with a brief description on how measurements are made and the benefits of applying the tests; (ii) past studies carried out to determine applicability of tests on site and their precision; and (iii) development of performance-based design and specifications using DI tests for use in service life design.

2.4.1 Durability Index tests

The DI tests are carried out using circular specimens with an approximate diameter of 70 mm and thickness of 30 mm. Laboratory samples are obtained by coring cubes of at least 100 mm while site samples may be obtained from either coring the structural element or test panels. The test specimens are preconditioned by placing in an oven at 50°C for a period of seven days to ensure uniform moisture conditions (DI Test Manual, 2009).

(i) Oxygen permeability index test (OPI)

The test was developed by Ballim (1991) and involves testing the permeability of a concrete sample when subjected to a falling head of pressure. After preconditioning of a concrete sample, it is placed in a rubber collar which is fitted into a rigid sleeve; this ensures that the sample is firmly held in place in the permeability cell ensuring no leakage of the gas. The test set up is illustrated in Figure 2.19.

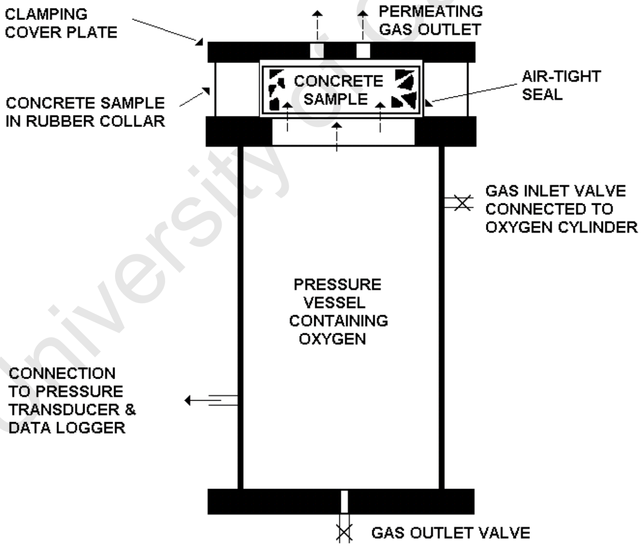

Figure 2.19: Oxygen permeability cell (Alexander and Mackechnie, 2001).

An initial pressure of 100 ± 5 kPa is applied and allowed to decay to a pressure of 50 kPa, or over a period of 6 hours, whichever occurs first. The change in pressure readings and time are automatically recorded in a data transducer, and using computer software these readings are transferred to Excel spreadsheets where computations are carried out to determine permeability (k) using Equation 2.18.

$$k = \frac{\omega V g d z}{R A \theta} \qquad (2.18)$$

Where, k = coefficient of permeability of test specimen (m/s)
ω = molecular mass of oxygen
g = acceleration due to gravity (9.81m/s^2)
z = slope of line determined in regression analysis[1]
R = Universal gas constant
A = Area of concrete specimen (mm^2)
θ = Absolute temperature (K)

An average of four test determinations of *k* are taken (from four specimens) and converted into a log scale by obtaining the negative log of the average *k* value, Equation 2.19.

$$\text{OPI} = -\log_{10}(k_1+k_2+k_3+k_4)/4 \tag{2.19}$$

The OPI test is useful in characterizing concrete mixes and the effects of concrete grade, curing method and type of binder used. Mackechnie (1996) observed that the OPI value increases with increase in concrete grade and with the use of wet curing method. The typical range of values for OPI test, on the log scale, is 8 to 11 where a higher OPI value indicates lower permeability (Alexander, 2008).

The test is also sensitive to micro-structural defects such as voids and is useful in evaluating the effectiveness of compaction of site concrete (Alexander and Mackechnie, 2001; Beushausen and Alexander, 2009). From studies by Ballim (1993) and Mackechnie (1999) a strong correlation between OPI and carbonation depth was observed which makes the test suitable in predicting service life of a structure.

(ii) Water sorptivity index

The test determines a potential measure of the sorptivity and porosity of concrete. The specimen used in the OPI test is re-used in this test. The edges of the specimen are sealed with tape and the initial mass determined, after which it is placed over paper towels soaked with calcium hydroxide solution. This is illustrated in Figure 2.20.

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[1] z is the slope of the line of best fit of a plot of ln (P_0/P_t) against time (s), P_0 is the initial pressure and P_t is pressure at a given time, t.

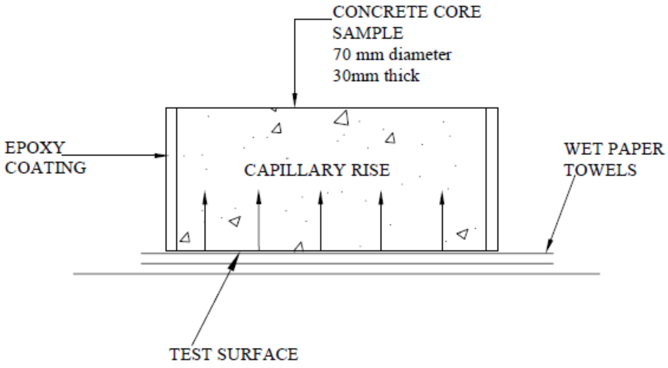

Figure 2.20: Schematic representation of water sorptivity test (Alexander and Mackechnie, 2001).

The mass of specimen is determined at time intervals of 3, 5, 7, 9, 12, 16, 20 and 25 minutes. A plot of the mass gained against square root of time is made by applying Equation 2.20. The slope of the line is used to determine sorptivity.

$$M_{wt} = F\sqrt{t} \qquad (2.20)$$

where,

M_{wt} = mass gained determined as, ($M_{st} - M_{so}$): M_{st} – mass at a particular time, M_{so} – mass at initial time

F = Slope of line of best fit, gives sorptivity of specimen (g/√hr)

t = time (hours)

To determine porosity, the sample is subjected to a vacuum, and then saturated with calcium hydroxide solution over a period of 18 hours. The porosity (n) of concrete is determined using Equation 2.21 where saturated mass is compared with oven dry mass.

$$n = \frac{M_{sv} - M_{so}}{Ad\rho_w} \times 100 \qquad (2.21)$$

where,

M_{sv} = Vacuum saturated mass (g)

M_{so} = Mass at start of the test (g)

A = Cross-sectional area of specimen (mm^2)

d = thickness of specimen (mm)

ρ_w = density of water (10^{-3} g/mm^3)

The test provides a measure of the sorptivity of near surface concrete and is useful in assessing the effectiveness of curing of site elements. It is also useful in characterization of concrete mixes where it has been observed that sorptivity values reduce with increase in grade of concrete and use of wet curing methods (Mackechnie, 1996). Typical ranges of sorptivity values vary from 5 mm/√hr for well cured Grade 30 – 50 concrete and 15-20mm/√hr for Grade 20 poorly cured concrete (Alexander, 2008).

(iii) Chloride conductivity index

The test was developed by Streicher and Alexander (1995) and provides a rapid indication of conductivity of a concrete sample. The test apparatus consists of two cells which are filled with 5 M sodium chloride solution, illustrated in Figure 2.21. The two cells screw onto the central portion which contains the concrete sample. This sample is vacuum saturated with the 5 M sodium chloride solution which ensures steady conditions (no concentration gradient exists) during the test such that migration of ions is only due to applied voltage.

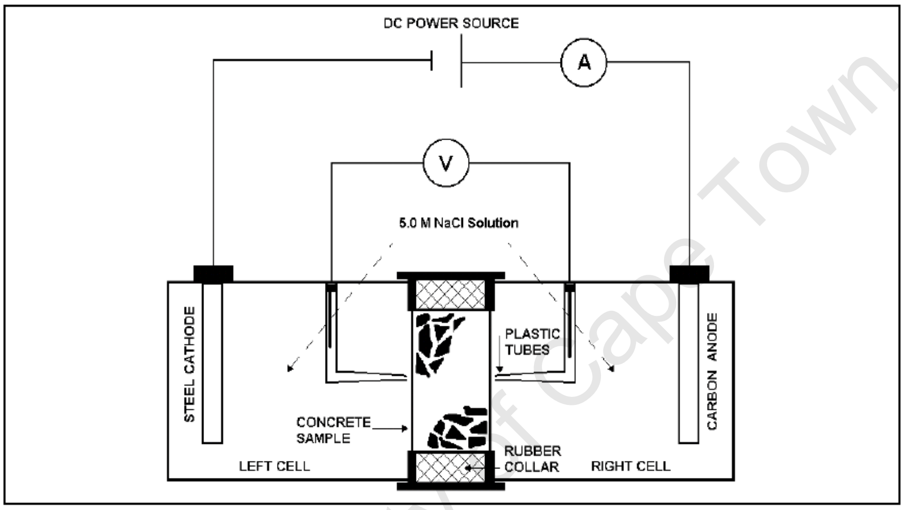

Figure 2.21: Chloride conductivity test set up (Streicher and Alexander, 1995).

A 10 V potential difference is applied from a direct current power source and amount of current passing through the specimen measured. The conductivity of a sample is computed using Equation 2.22.

$$\sigma = \frac{i}{V}\frac{t}{A} \qquad (2.22)$$

where,

σ = conductivity (mS/cm)

i = electric current (mA)

V = Voltage difference (V)

t = thickness of specimen (cm)

A = Cross-sectional area of specimen (cm^2)

The diffusivity of the concrete specimen (D) is determined by applying the diffusibility relationship of a porous material given in Equation 2.23.

$$Q = \frac{D}{D_o} = \frac{\sigma}{\sigma_o} \tag{2.23}$$

where,

Q = Diffusibility of porous material (constant)

D = Diffusivity of porous material

D_o = Diffusivity of ion through porous solution

σ = Conductivity of material, obtained from CCI test

σ_o = Conductivity of pore solution

The test is useful in the optimization of a concrete mix to ensure that it has high resistance to penetration of chloride ions for concrete structures in marine conditions. Mackechnie (1996) observed that with the use of blended cements containing fly ash and slag (GBBS), the chloride conductivity was substantially less than that of plain ordinary Portland cement. The values of chloride conductivity were also observed to reduce with time. Typical values of chloride conductivity are 2.5 mS/cm or greater for plain OPC, to 0.5 mS/cm or less for highly chloride resistant blended concrete (Alexander, 2008).

2.4.2 Application of Durability Index tests: Site-based studies

The durability index tests were developed for use on elements prepared both in the laboratory and on site. Several laboratory based studies have been undertaken where the effects of different concrete blends, curing methods and concrete grade on DI test values have been determined (Ballim, 1993; Streicher and Alexander, 1995; Mackechnie, 1999). The section below provides a review of application of DI tests in site based studies and observations made on the practicality of these tests in quality control and their effect on construction practices.

2.4.2.1 Practical implementation of Durability Index tests

An initial study on the practical application of DI tests on site was carried out by Bouwer (1998). The aspects evaluated in the study were: - (i) application of DI tests in laboratory and a critical review to determine if tests are correctly applied; (ii) practicality of implementing the DI tests in site conditions and the ability to get valid DI test values; (iii) evaluation of DI test results obtained from Ready-mixed concrete and comparison of these values with those of site prepared concrete. These aspects are further described below.

(i) Application of DI tests in the laboratory: This was carried out with the main objective of critically reviewing the clarity of the test methods and if they are correctly applied. The exercise was carried out by evaluating DI test results of two laboratories – University of Stellenbosch (US) and University of Cape Town (UCT). The DI tests were carried out on three concrete mixes prepared with plain OPC, fly ash and slag (GBBS). The single operator coefficient of variation (1s %) computed from this evaluation for the US lab is given in Table 2.6. The variability in the three tests undertaken in the lab was generally low indicating that consistent results can be obtained under controlled laboratory conditions.

Table 2.6: Single operator coefficient of variation for laboratory samples (1s%) (Bouwer, 1998).

Concrete source	OPI	Water sorptivity	Chloride conductivity
Laboratory (%)	1	5	4

It was also observed that the tests were in some cases not correctly applied and some aspects of test procedures were uncertain and subject to different interpretation. Recommendations were made on improvement of various aspects of the test methods and the need to properly train the users in order to ensure correct testing, that influences reliability of test results, was highlighted.

(ii) Practical implementation of DI tests on site where the experimental work involved carrying out the tests on structures located in six different locations using site prepared concrete mixes, at which different construction practices were used. The samples used in testing were obtained from four sources: actual structure; wet cured cubes; beam elements prepared to simulate actual structure with one beam subjected to the curing method used for actual structure and the other without any curing applied. The beam elements were prepared to determine the possibility of using these elements to obtain samples for testing instead of coring the actual structures.

From the observations made, the wet cured samples generally yielded better DI test values in comparison to site elements. However, in some cases, the results obtained from site were better than those of wet cured cubes which demonstrated that if concrete is properly placed, compacted and cured, favourable DI test values can be obtained from site elements. The test results of beam elements did not have a strong correlation with results of actual structure, and thus may not be suitable to simulate the actual structure. A summary of single operator variance of DI test values of elements tested on site and wet cured samples prepared from concrete prepared on site is given in Table 2.7.

Table 2.7: Single operator coefficient of variation for site-based samples (1s%) (Bouwer, 1998).

Concrete source	OPI	Water sorptivity	Chloride conductivity
Actual structure (%)	3	13	14
Wet cured site mixed concrete (%)	2	12	7
Wet cured ready mixed concrete (%)	1	7	5

(iii) DI test results from samples prepared using ready mixed concrete (RMC) were evaluated. A total of nineteen sets of RMC with different water/cement ratio, compressive strength and mix constituents were used. From observations made, the variability in DI test values was lower in comparison to that of site prepared cubes; this is illustrated in Table 2.7. This demonstrates that with a higher level of control on production of concrete, good DI test values are obtainable.

2.4.2.2 Evaluation of curing efficiency

Alexander et al. (1997) carried out a site based study that involved testing of median barriers used in highways in Cape Town. These elements were cast in situ and cured using a water based wax emulsion compound. The DI tests were carried out during the summer and winter season on samples subjected to three different curing methods; samples from median barriers on which curing compound had been applied, samples with no curing compound and wet cured cubes used as controls. The observations made were that DI test values of wet cured samples where better than those of site elements, with samples on which curing compound had been applied yielding better DI values than those with no curing compound during the summer period. However, during the autumn season which experienced light rains, the DI values of elements with no curing compound were better than those of cured samples due to moisture in environment that leads to additional curing.

du Preez and Alexander (2004) carried out a site based study to investigate the effectiveness of different curing methods. The investigation involved casting of panels on a site, located in the Port of East London, using three concrete mixes of OPC blended with fly ash, slag (GBBS) and silica fume (CSF). The curing methods used were air curing, hessian cloth, curing compound and damp sand; wet cured cubes were cast with the same concrete mixes for use as controls. The DI test values were obtained at an age of 28 and 120 days. The observations made were that the DI values of wet cured samples were better than those of site cured samples at 28 days. However, at 120 days the DI values for all the different site cured elements had improved with a significant improvement for the fly ash concrete mix and slight improvements on blends with CSF and GBBS. This improvement was due to continued curing of the elements due to environmental moisture. Figure 2.22 illustrates the effects of curing methods on OPI values.

From the two case studies it is observed that moisture in the environment leads to further curing of concrete on site; the use of curing compounds however shields concrete from environmental moisture. The long term durability of site elements, as measured with DI tests, therefore depends less on curing method used, and more on the environmental conditions. Thus environmental conditions were determined to be an essential aspect to consider in developing performance specifications for reinforced concrete structures.

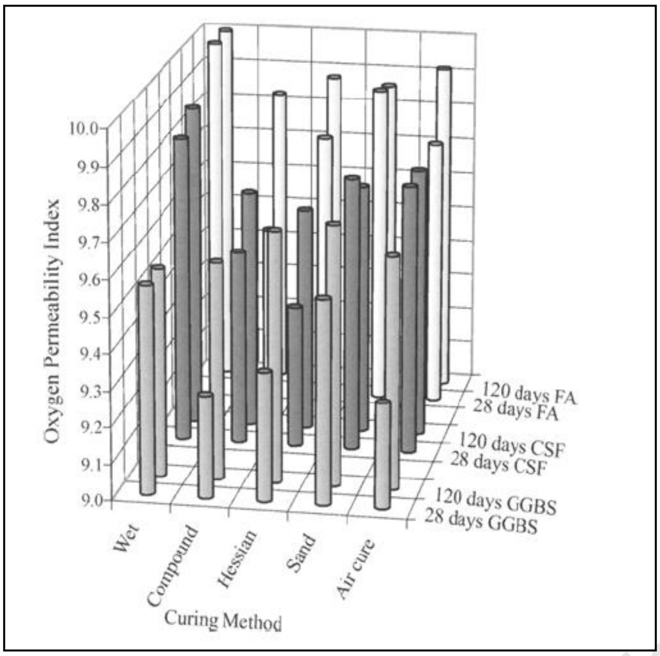

Figure 2.22: Effect of curing methods on OPI values at 28 and 120 days (du Preez and Alexander, 2004).

2.4.2.3 Evaluation of precast unit elements quality

Ronnè (2000) reports on a study carried out to determine the quality of near surface properties of concrete of precast units manufactured by obtaining measures of DI values.

The assessment of quality of precast unit elements was made from:-

(i) Industrial visits where concrete mixes were obtained from two precast unit manufacturers and used to prepare cubes to obtain DI values for concrete characterization at 28 days. Samples were also obtained from the actual units manufactured, and values from the two sources compared.

(ii) A study on effect of use of fly ash in a concrete mix and its effect on concrete properties as characterized by DI test. Three manufacturers (A, B and C) were considered. The construction methods used by manufacturer A and B were shutter vibration and steam curing while manufacturer C used poker vibrators and a resin based curing compound.

The DI values obtained from this study are summarized in Table 2.8. It was observed that values obtained from laboratory samples are higher (or lower) than for actual units manufactured, indicating the effect of construction processes on quality of concrete cover. The use of fly ash also improved quality of concrete as indicated by higher values for OPI and lower values of sorptivity and chloride conductivity.

Table 2.8: DI values from precast unit manufacturers (Ronne, 2000)

		Laboratory prepared sample			Precast unit sample		
		DI test values			DI test values		
	Concrete properties	OPI (log scale)	Sorptivity (mm/hr)	CCI (mS/cm)	OPI (log scale)	Sorptivity (mm/hr)	CCI (mS/cm)
Source: Factory visits							
Manufacturer 1	Rapo[#1]	10.46	5.60	1.52	10.03	6.20	1.57
Manufacturer 2	OPC[#1]	10.32	5.40	0.70	9.53-10.27	4.5-7.7	1.07-1.84
Source: Investigation study on use of fly ash							
Manufacturer A	0.47	10.44	5.70	1.07	9.65	5.10	1.21
Manufacturer B	0.40	10.58	4.10	0.67	10.08	4.60	0.72
Manufacturer C	0.40	10.41	5.50	0.73	10.21	5.10	1.20

[#1] Rapo – rapid hardening cement: OPC – ordinary Portland cement.

In addition, a comparison was made on the quality of cast in place and precast elements where it was observed that the quality of the latter was better, indicating control in manufacture of precast elements is stricter than on site which results in higher quality.

2.4.3 Precision of the Durability Index tests

Inter-laboratory exercise

An important aspect in development of test methods is the determination of their precision, defined as a measure of magnitude of variability expected between test results when test is carried out in one or more reasonably competent laboratories (ASTM E 177-06b). Precision is evaluated by carrying out an inter-laboratory study (round robin test) where the repeatability and reproducibility are determined. Repeatability is variability of individual test results when tests are carried out on same material by the same operator with the same apparatus while reproducibility is variability of results carried out in different laboratories using materials that are as nearly identical as possible (ASTM C 670-03).

Grieve et al. (2003) provide a report on an inter-laboratory exercise carried out due to concerns raised on differences in results on the same test between laboratories, and difficulties in achieving required test values under site conditions. The exercise involved a total of seven laboratories which were required to test two concrete mixes subjected to two different curing methods, making a total of four samples, for the three DI tests. The range of results from the exercise are given in Table 2.9. These values indicate low variance for the OPI test and larger variance for water sorptivity and CCI tests. It was recommended that test methods be rewritten, giving more detail to reduce the possibility of different interpretation.

Table 2.9: Summary of range of results from inter-laboratory exercise (Grieve et al., 2003)

	OPI	Water sorptivity	Chloride conductivity
Repeatability (%)	0.43 – 2.84	0.17 – 17.82	1.39 – 57.44
Reproducibility (%)	0.48 – 2.80	11.43 – 22.60	10.90 – 39.71

The test methods were revised, in order to make them simpler and improve clarity, by a working group that involved representatives from both industry and academics. The revised test methods were published in 2004 and a training workshop held. The laboratories were then given time to familiarize themselves with the test method before another inter-laboratory exercise test was conducted by Stanish et al. (2004). This exercise involved a total of nine laboratories which conducted the three tests on ten different concrete mixes. The results from the exercise are summarized in Table 2.10.

Table 2.10: Test repeatability and reproducibility (Stanish et al., 2004)

	OPI	k-value	Water Sorptivity test		Chloride conductivity test	
			Sorptivity	Porosity	CCI$^{\#1}$	Porosity
Repeatability (%)	1.4	32.2	9.9	5.5	9.1	5.5
Reproducibility (%)	1.8	36.6	12.8	6.4	21.1	8.9

$^{\#1}$Chloride conductivity index

The variability of OPI test was observed to be low while variability for sorptivity and CCI tests were higher. The reason for high variability in sorptivity tests was attributed to failure of the labs to maintain vacuum correctly during saturation. The possible reasons cited for high variability in CCI were failure to properly carry out test procedures, improperly trained technicians and limitations in test equipment.

2.4.4 Durability Index based Performance design and specifications

The durability index tests provide a quantifiable measure of the near-surface concrete penetrability which makes them suitable for use as limiting values against which compliance can be evaluated; this makes the tests suitable for use in performance-based specifications. An earlier version used in classification of concrete for durability based on a range of values, given in Table 2.11, was proposed by Alexander and Mackechnie (2001). Limitations in this approach were identified by Grieve et al. (2003) as it fails to give estimates of expected service life of a structure and does not take into consideration environmental conditions.

Table 2.11: Suggested range for durability classification using DI values (Alexander and Mackechnie, 2001)

Durability class	OPI (log scale)	Sorptivity (mm/hr$^{0.5}$)	Conductivity (mS/cm)
Excellent	>10	<6	<0.75
Good	9.5-10	6-10	0.75-1.50
Poor	9.0-9.5	10-15	1.50-2.50
Very poor	<9.0	>15	>2.50

Subsequently, Alexander et al. (2006) proposed adopting the framework for performance specifications developed by Harrison (1995). This framework is based on a seven step process which involves: - (i) Definition of exposure class where prevalent form of deterioration in a given environment is considered (ii) A quantitative design methodology and criteria for what defines the end of service life. (iii) Test procedures which relate to output parameters of the design methodology (iv) Provisional conformity requirements or limiting values which are checked against traditional durability test methods. (v) Establishing limits of test applicability (vi) Determination of effective systems for production and acceptance testing. (vii) Implementation of specifications in full scale trials and long term monitoring to confirm provisional limiting values determined.

A framework for durability index studies has been developed by Alexander et al. (2006), illustrated in Figure 2.23.

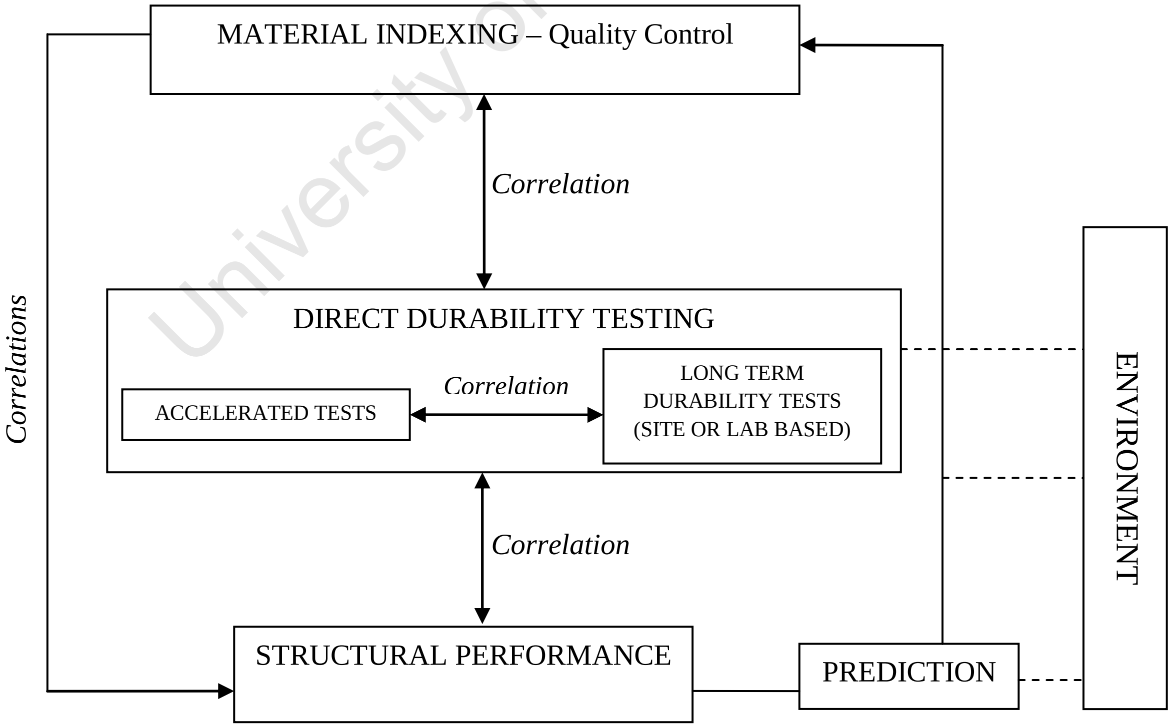

Figure 2.23: Framework for durability studies (Alexander et al., 2006).

The performance based approach involves use of suitable prediction models (carbonation or chloride-induced service life models) depending on the environmental conditions. The service life models are used to determine limiting durability index values which are used for material indexing and quality control. The early age DI test values are validated by establishing correlations with: - (i) direct durability tests which may involve use of accelerated tests, or long term laboratory or site-based tests; and (ii) actual structural performance over a long period of time.

The subsequent section provides a review of: - (a) the DI-based performance design used in service life predictions in South Africa for structures subject to chloride and carbonation-induced corrosion, and (b) the DI-based performance specifications developed which consider a matrix of factors (service life, cover depth, exposure conditions and material properties) and are based on a two-level approach where compliance with limiting values is required for both the material supplier and contractor.

2.4.4.1 DI based performance design

Service life prediction models for chloride and carbonation induced corrosion have been developed for use in South Africa where input parameters in this models have been empirically related to durability index test values (CCI and OPI).

Carbonation model

Ballim (1993) evaluated the relationship between 28 day OPI values with carbonation depth of 10 month concrete samples; a strong correlation was established between the two. Mackechnie (1999) carried out further investigations on this relationship by comparing 28 day OPI values with carbonation depth of cores from concrete samples exposed to outdoor conditions for a period of 1, 4 and 6 years. A strong correlation was observed between OPI and carbonation depth and based on this relationship, a carbonation prediction model was proposed, given in Equation 2.24.

$$d_c = k_c\, t^x \qquad (2.24)$$

Where,

d_c = carbonation depth (mm)

k = material coefficient (mm/year^x)

t = exposure time

x = varies from 0.1 – 0.4

Chloride model

Mackechnie and Alexander (2002) developed a pragmatic model for use in marine environments. This model considers material properties (effect of use of different binders), construction process (curing methods) and environmental processes and the effect that these aspects have on chloride ingress. The model is based on the modified solution to Fick’s second law of diffusion, given in Equation 2.25.

The diffusion coefficient was modified to take into account reduction of diffusion with time due to long term effects of chloride binding and maturity.

$$C_x = C_s[1 - erf(\frac{x}{2\sqrt{D_i t^{(1-m)}}})] \tag{2.25}$$

Where,

C_x = chloride content at a depth x (% by mass of binder)

C_s = Surface chloride content

x = cover depth (m)

D_i = Modified diffusion coefficient (m^2/s)

m = material coefficient (dependent on type of binder)

Mackechnie (1996) carried out a comparison of chloride conductivity index values with diffusion coefficients of concrete samples exposed to marine environments for two years. A weak correlation was found between the two but on modifying CCI values to take into account binding effects and maturity a good correlation was established, thus making CCI suitable for use in the prediction model.

Further developments in service life models using DI tests have taken place. For example Muigai et al. (2009) proposed a framework for a probabilistic approach for the chloride-induced corrosion model. The uncertainties in the physical, statistical and model aspects were considered. The use of a probabilistic approach would be beneficial as it refines the prediction model and increases its reliability.

2.4.4.2 DI based performance specifications

The limiting DI values used in performance specifications consider a matrix of factors in what is described as a 'multi-factor' approach (Alexander and Stanish, 2005). The factors considered are: environmental exposure conditions which are adapted from environmental classification given in EN206-1 and modified for South African conditions; cover depth provisions; required service life of structure which depends on use of the structure; and material properties of concrete.

The limiting DI values used in specifications can be determined using either: - (i) a rigorous approach where limiting values are determined by use of the relevant service life prediction model. This approach is suitable for durability critical structures and allows a designer flexibility in selecting the service life of a structure and cover depth. However, it is not suitable for routine use as it requires expertise in its application to ensure that it is correctly used and results correctly interpreted. Or (ii) a "deemed-to-satisfy" approach where for typical construction scenarios (e.g. service life and cover depth) limiting values are determined using service life models, and tabulated. The DI tests are applied to evaluate conformance with a limiting DI value, which if satisfied, a structure is "deemed-to-satisfy" the durability requirements. This approach is simple to use and suitable for routine application (Alexander et al., 2008).

Two illustrations are provided below of limiting values that can be applied in the "deemed-to-satisfy" approach. For carbonating environments the limiting OPI values, given in Table 2.12, are provided for exposure conditions XC3 and XC4 (EN 206); for exposure conditions XC1 and XC2, provided a minimum cover depth of 30 mm is achieved carbonation-induced corrosion is unlikely. From the table, it is observed that a designer can reduce the limiting OPI value by increasing cover depth.

Table 2.12: Limiting DI values ('deemed-to-satisfy' approach) for carbonating conditions (Alexander et al., 2008)

	Common structures	Monumental Structures	
		1	2
Service life (years)	50	100	100
Minimum cover (mm)	30	30	40
Minimum OPI	9.70	9.90	9.70

For marine environments, the limiting CCI values are determined by considering cover depth, service life and binder type used. The use of blended cement is recommended as they give more resistance to chloride ingress in comparison to plain ordinary Portland cement. Table 2.13 gives limiting CCI values for monumental structures (service life of 100 years) and a cover depth of 50 mm.

Table 2.13: Maximum chloride conductivity values (mS/cm) (Alexander et al., 2008)

EN 206-1 Class	70:30 CEM I: Fly ash	50:50 CEM I: GBBS	50:50 CEM I: GGCS	90:10 CEM I: CSF
XS1	2.50	2.80	3.50	0.80
XS2a	2.15	2.30	2.90	0.50
XS2b, XS3a	1.10	1.35	1.60	0.35
XS3b	0.90	1.05	1.30	0.25

A two-level approach has been developed for DI based specifications where the responsibility of production of durable concrete is shared by the material supplier and contractor (Alexander et al., 2006). The material supplier (considered to be a Ready Mix supplier) is required to select appropriate constituent materials and proportion them suitably which determines the 'material potential' of concrete. The contractor is then required to properly process the concrete on site (placing, curing and finishing) to ensure that it attains the limiting DI value provided in specifications. The measured DI value represents finished ('as-built') quality of a structure.

2.4.4.3 Cover depth measurements

The durability of RC structures is dependent on cover depth thickness. Studies on cover depth measurements in South Africa have been undertaken e.g. Ronne (2000) reported a study on assessment of durability of precast elements used as culverts; one of the aspects of durability considered was the provision of adequate cover depth. Two case studies on cover depths were considered: cover depth of precast elements in an existing culvert on site and cover depth of precast elements manufactured.

In the first study, a cover survey on culvert on site was done at positions inside the culvert along its width, as illustrated in Figure 2.24.

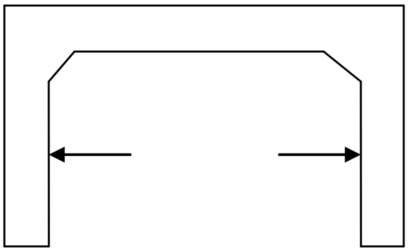

Figure 2.24: Positions along culvert width (as indicated by arrows) at which cover depth was checked (Ronne, 2000).

From the cover depth analysis, the mean coefficient of variation of all elements surveyed was 23%. Coefficients as high as 70% were obtained which indicates poor control. A comparison of coefficient of variation and cover depth obtained from this survey is provided in Figure 2.25; it was observed that with a higher cover depth, the coefficient of variation was lower. The incidence of cover depths below the required minimum of 20 mm was found to range from between 1 – 11 of cover depth readings obtained.

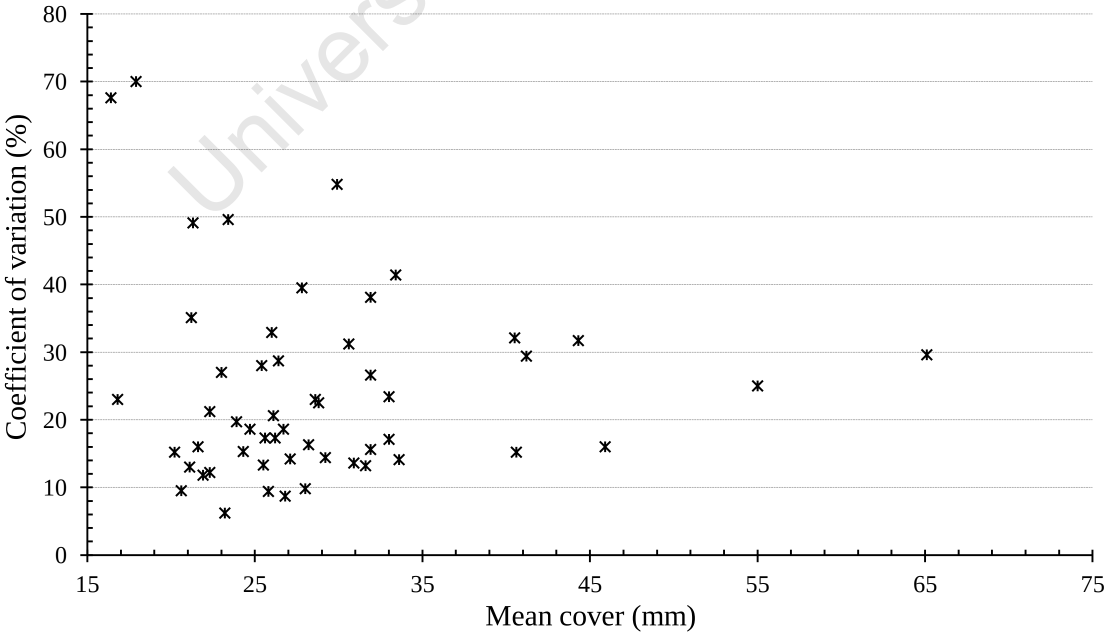

Figure 2.25: Variability of cover depth from cover depth survey (Ronne, 2000).

In the second study, quality (cover depth) of precast elements manufactured was assessed. The specified cover depth of these elements was 40 mm. The sampling procedure used involved a random selection of ten precast elements manufactured which were checked for cover depth in a similar procedure to that used in the first study; checks on cover were done on both the inside and outside of the elements. A summary of cover readings obtained from cover survey on these elements is given in Table 2.14.

Table 2.14: Cover depth readings from precast elements manufactured (Ronne, 2000)

	Manufacturer A		Manufacturer B	
	Inside	Outside	Inside	Outside
Mean (mm)	29.6 - 35.2	29.8 - 67.3	48.3 - 58.2	45.9 - 56.8
s[#1] (mm)	11 - 22.9	4.6 - 27.3	4.6 - 8.9	1.6 - 7.9
CoV[#1] (%)	35.9 - 67.6	6.8 - 56.6	9.3 - 18.1	3.1 - 14.3
Incidences below specified	13 - 24		1 - 2	

[#1]: s - standard deviation; CoV – coefficient of variation

The summary of values in Table 2.14 indicates that for Manufacturer A, the range of readings was below the specified values for readings obtained inside the culvert and amount of variability was high. The Manufacturer indicated that failure to achieve the required depth and high incidence of values below specified, was because the elements were manufactured to meet specifications of minimum cover depth of 20 mm instead of specified cover depth in the project of 40 mm. Manufacturer B however attained the required cover depths and amount of variability, as indicated by standard deviation and coefficient of variation, was low. This indicates that with proper control the required cover depth values can be attained.

From a comparative study on cover depth readings in buildings and bridges by Ronne (2005), it was observed that the variability of cover depth readings were significantly higher than those on an international context. The reason attributed to high variability was lack of a system to evaluate compliance of cover depths in finished structures. Recommendations were made to conduct cover surveys on site routinely to ensure compliance with specifications. A proposal to conduct cover surveys with a minimum of 45 individual readings over a survey area of 1.5 – 2 m^2 per structural member was made.

2.4.4.4 Implementation DI based performance specifications in construction

The DI based performance specifications, described in the preceding section, have been implemented by the South African National Roads Agency Limited (SANRAL) on major infrastructure projects in the country. Knecht (2009) carried out a study to evaluate the effect of implementation of the DI-based specifications on the quality of construction. His study involved assessing construction practices employed by three manufacturers of precast barriers used on freeways. The manufacturers were observed to have improved the construction practices employed, with one manufacturer investing in a new water curing facility while another trained operators on the use of compaction machines to reduce fluctuations in compaction quality. The implementation of these specifications also improved quality of precast units produced with one manufacturer observing a reduction in elements rejected from 1% to 0.2%. This demonstrates the practicality of the DI based specifications in improving quality of execution of site processes, which is important in ensuring that durable structures are constructed.

However, some limitations were observed in implementation of the DI based specifications such as delays in testing at laboratories due to shortage in apparatus and lack of properly trained staff. This highlights the need to increase capacity of laboratories in terms of apparatus and staff to ensure that testing is carried out promptly, within the required 28 to 35 days, for the tests to be effectively used in quality control.

The implementation of the DI tests and performance specifications has led to an improvement in construction practices and quality of concrete structures which highlights practicality of the tests in design of durable concrete structures. In addition, the tests have been found to compare favourably with other internationally applied test methods (Beushausen and Alexander, 2008), which further confirms that these tests are practical in assessing near surface concrete quality.

2.5 SUMMARY

The Chapter presents a review on durability in reinforced concrete structures which is dependent on the quality of the concrete cover – its penetrability and thickness. A fundamental aspect to consider in the design of structures for durability is the penetrability of concrete cover to transport of aggressive substances by permeation, diffusion and absorption. A myriad of tests have been developed by researchers, locally and in an international context, for measuring transport mechanisms in concrete; however, limitations arise with application of these tests such as standardization of moisture conditions and high variability which makes it difficult to routinely apply the tests in practice.

Different approaches are employed in design of structures for durability. The common approach widely used, and provided in current standards, is based on a ‘deemed-to-satisfy’ approach that limits concrete composition and requires verification for compliance with durability requirements through a measure of strength. However, strength is an inadequate criterion for durability as it measures bulk properties and does not verify penetrability properties of concrete cover.

The limitations in the current prescriptive approach have led to development of the performance-based approach which involves the use of: mathematical models in design for a given service life, performance specifications and performance tests. Different approaches to performance-based approach have been discussed, both in an international context and locally in South Africa through the use of DI-based performance approach. This approach is practical in design of concrete mixes and on site in the control of quality of reinforced concrete structures and construction processes (e.g. curing).

The DI-based performance approach has been implemented on a large scale infrastructure project. The main objective of this research is to evaluate the practicality (validity) of implementation of this approach in site for control of quality of reinforced concrete structures and construction processes. These aspects of practicality of the approach are further discussed and presented in Chapters Three and Four.

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Chapter Three

Methodology

3.1 INTRODUCTION

The durability index based (DI) performance approach in South Africa requires the verification of concrete cover penetrability (using DI tests) and measures of concrete cover thickness. This approach has been adopted by the South African National Roads Agency Limited (SANRAL) in infrastructure projects such as the Gauteng Freeway Improvement Project (GFIP). The DI test values (OPI and sorptivity) and cover depth readings from this project form the main source of data to be used in this study to assess practicality of implementation of the DI performance approach in control of quality of near surface concrete and evaluation of construction practices. This Chapter presents a description of the project, the data collection process and a review of statistical methods used in carrying out analysis of data.

3.1.1 Project description

SANRAL is responsible for the development and maintenance of mainly the national road network in South Africa. To alleviate traffic congestion problems experienced in the recent past in Gauteng Province, SANRAL developed the Gauteng Freeway Improvement Project (GFIP) which aimed at upgrading the road network through expansion by adding to the number of lanes in existing freeways, improvement and construction of interchanges, and construction of concrete median barriers on the freeways (SANRAL, 2011). The GFIP was initiated in 2007 and involves improvement of a total of 560 km of the road network; the first phase, currently underway with majority of the work completed, was undertaken on a 185 km road network. The construction over this road network was sub-divided into fifteen projects; a plan of the area covered by GFIP is illustrated in Figure 3.1.

The GFIP has implemented the DI based performance specifications where quality control for concrete structures designated as ‘Class W’ involves measures of strength, DI values and concrete cover. The values obtained from DI testing of structural elements in these projects form the main source of data to be used in attaining the research objectives which are outlined in the ensuing section.

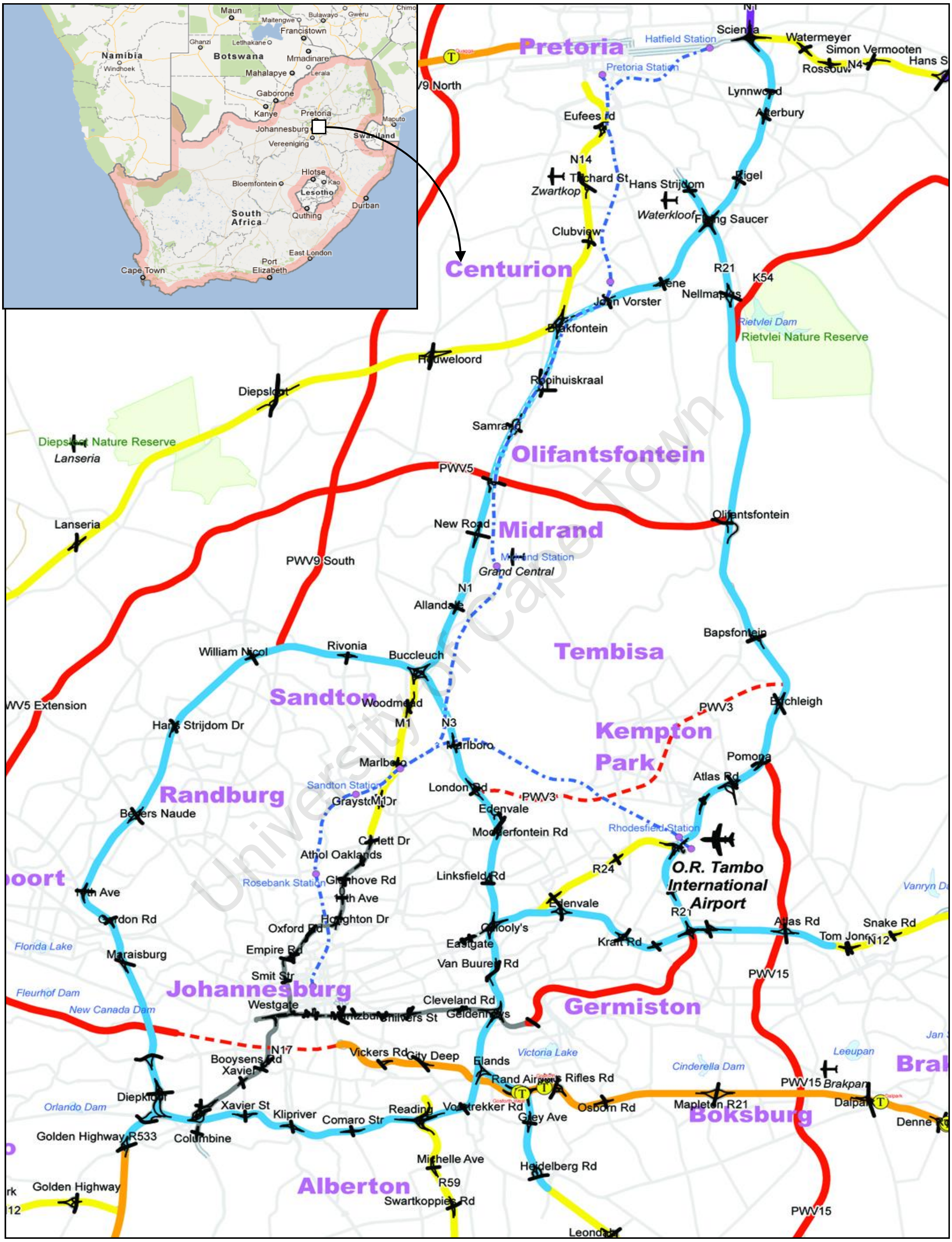


Figure 3.1: Schematic representation of the area covered in the first phase of the Gauteng Freeway Improvement Project (GFIP) - highlighted in blue (SANRAL, 2010).

3.1.2 Research objectives

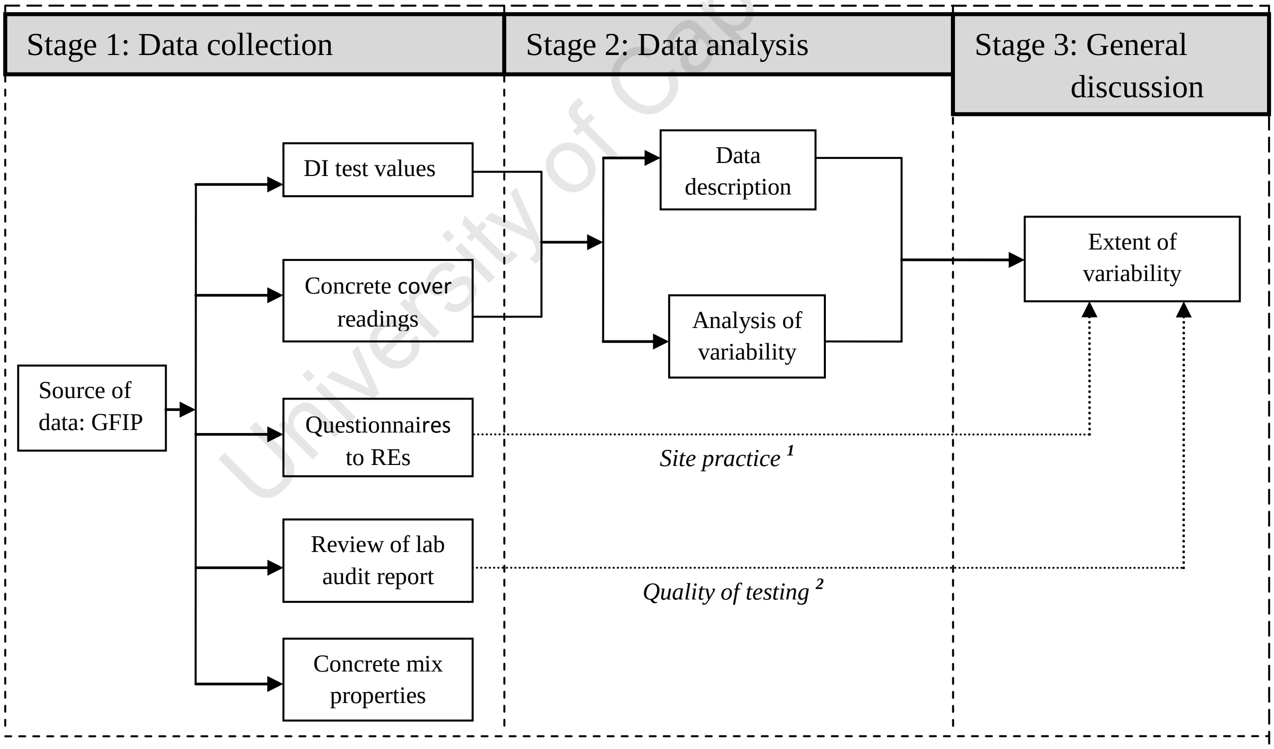
The main objective of the study was to evaluate the practicality of implementation of DI based performance specifications on site, in control of quality of RC structures and construction processes. The practicality of these specifications was evaluated by considering:-

(i) The extent (magnitude) of variability in DI test values and cover depth readings obtained from in-situ and precast elements and proportion of values that complied with limit values in specifications.

(ii) Application of tests on site and in laboratories in order to evaluate if the tests are correctly undertaken, which influences the reliability of results obtained, and to determine the practical difficulties encountered in application of the tests.

(iii) Perception of the implementation of DI-based performance specifications in practice by Resident Engineers to assess their opinion on effect of this approach, an additional quality control measure, on construction of concrete structures.

To attain the objectives outlined above, the research methodology that was adopted involved a three stage process illustrated in Figure 3.2.



[1] Site practice: Consideration of construction practices employed on site – methods of compaction and curing.

(Note: The site environmental conditions – relative humidity and temperature- have an influence on the DI test results. These aspects were however not considered as data on environmental conditions was not available.)

[2] Quality of testing: Consideration of execution of test in laboratories as per test method procedure (OPI).

Figure 3.2: Schematic representation of methodology adopted.

The ***first stage*** of the methodology involved the collection of data from various sources of the main case study, GFIP. The data collected includes:-

(i) DI test values (OPI and sorptivity values) obtained from nine projects involved in the GFIP; these values were obtained from testing of both in-situ (samples obtained from test panels or cubes) and precast elements.

(ii) Cover depth readings obtained from different structural elements of four bridges in one project, and precast elements used as median barriers.

(iii) Response to a questionnaire sent out to Resident Engineers in projects. The questionnaire sought to obtain information on construction processes used (compaction and curing methods), their perception on implementation of DI based specifications, and difficulties encountered in application of this approach.

(iv) The properties of constituents of concrete mixes used in the project obtained from the Ready Mix Concrete supplier. For the precast elements, the concrete mix properties were obtained from the precast manufacturer.

(v) Review of a report from a laboratory audit exercise carried out under the auspices of the Durability Index Focus Group[2]. The audit exercise was done with the aims of evaluating if proper testing was carried out in the labs and identifying any difficulties that were encountered in application of the tests.

The ***second stage*** involved statistical evaluation of data (OPI, sorptivity and cover depth values) in order to quantify the extent of variability. The initial stage of statistical analysis involved data description which was carried out by determining the measure of central tendency (mean values) and measures of dispersion (standard deviation and coefficient of variation). In addition to numerical summaries, graphical summaries were made by use of histograms and box plots. Scatter plots were also used to compare values of strength and DI test values (sorptivity and OPI) to determine if a relationship existed. Further statistical analysis involved hypothesis testing in order to determine if the extent of variability in results was ‘statistically significant’. The variability was based on consideration of: difference in mean values obtained from a project and the limit value in project specifications; difference in mean values among the projects considered through use of analysis of variance (ANOVA); difference in average values when data are classified on the basis of concrete

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[2] Cement and Concrete Institute Durability Index (DI) Focus Group
This is a committee comprised of people from cement and ready mix companies, the specifying agency (SANRAL), consultants, contractors and researchers (CSIR and universities) with the main objective of implementing South Africa’s DI approach through a critical evaluation of test methods in order to formalize them to SABS standards.

grade, source of sample (cube or test panel; in-situ or precast elements) (t-test). The hypothesis tests listed above are further described in subsequent sections.

The ***third stage*** involved a general discussion that considered the extent of variability in relation to execution of construction practices on site (curing and compaction) and execution of tests in laboratories. The main limitations that arose in this discussion were: firstly, the quality of testing in laboratories was based on review of a report from an audit exercise. This review only provided a background on quality of testing in laboratories; no information on the quality of testing of site-based elements (from samples obtained from cubes or test panels), on which analysis of data for this study was based, was available. Therefore, the review was used to infer (provide a background) on quality of testing in labs and the possible influence on test results. A second limitation that arose was that data on site practices used was obtained from questionnaires sent out to REs in projects (secondary data). No observations on the actual execution of these practices were available which limits information on quality of execution of these practices on different sites/projects.

The data collection process is further discussed in the subsequent section while the second and third stages that involve data analysis and a general discussion are presented in Chapter Four.

3.2 DATA COLLECTION

3.2.1 Durability Index tests

The GFIP is located in an inland region in South Africa; DI based specifications for reinforced concrete structures in this region are those that relate to carbonation-induced corrosion. The project specifications require verification of OPI, sorptivity and cover depth values of as-built structures (through use of test panels); limiting values for these parameters that were in use for the GFIP are summarized in Table 3.1.

Table 3.1: Limiting values used in DI based performance specifications and the reduced payments criteria applied (SANRAL, 2010)

Description of test	Oxygen Permeability Index		Concrete cover	
	OPI (log scale)	Percentage payment	Overall cover (mm)	Percentage payment
Full acceptance	> 9.70	100%	≥ 85% <(100%+15mm)	100%
Conditional acceptance (with reduced payment)	> 8.75 ≤ 9.70	80%	< 85% ≥ 75%	85%
Conditional acceptance (with remedial measures as approved by Engineer and reduced payment	-	-	< 75%	70%
Rejection	< 8.75	Not applicable	< 65%	Not applicable

For sorptivity, a limit value of 10 mm/√hr was established. However, a reduction in payment was not applied.

The limiting values in current project specifications have subsequently been modified; the revised values are based on consideration of exposure classification, cover depth and binder types (for CCI values). These values are presented in Table 6000/1 of SANRAL specifications, given in Appendix A.

Source of samples

Concrete structural elements (‘Class W’) are designated for both strength and durability testing; examples of such structures are bridge piers, abutments, concrete median barriers, retaining walls, culverts, parapets and bridge decks. The concrete mixes used for these elements need to be approved prior to casting; this is done by preparation of trial panels which are tested to ensure that the concrete producer meets the target requirements for OPI and sorptivity.

In the current project specifications, samples used for testing can be obtained from either: test panels which are constructed on site adjacent to the actual structure with the same concrete, shutter type, compaction and curing methods; or the actual structure e.g. for precast elements. The dimensions of test and trial panels are 0.4 m wide, 0.6 m high and 150 mm thick. Prior to a study carried out by Ronny and Everitt (2010), samples for DI testing were also obtained from cubes. However, from their study it was established that values from cubes were superior to those of the actual structure, and the use of test panels is preferable as they are more representative of the actual structure. The data collected and used in data analysis in this study consists of results obtained from cubes (obtained from an earlier period before specifications were revised), test panels and structural elements (precast units).

Additional information with regard to the frequency of testing, number of cores to be extracted and number of test panels required for durability testing are provided in Section B8100 Tables B8106/1 and B8106/2; this is given in an extract of the project specifications in Appendix A.

Samples obtained from site elements should be properly packaged during transport to the laboratory for testing to protect against adverse drying and damage. The test specimen on receipt by a laboratory should be kept under ambient conditions for a maximum of 3 days before preparation for DI tests. The OPI and sorptivity tests are carried out in accordance with test procedures outlined in the DI Test Manual (2009).

Recording of data

An essential part in undertaking a test is to ensure that proper recording of data is done. The recording of data and computation of test parameters (coefficient of permeability (k), OPI, sorptivity and porosity) is done using an Excel spreadsheet developed at the University of Cape Town (UCT).

In order to identify a test specimen a record of the following data should be provided:-

- Type of sample (test panel, cube or actual structure)
- Structural element (beam, pier, deck)
- Source of sample – geographical location
- Age of concrete at time of testing
- Curing history
- Concrete mix properties
- Any unusual specimen preparation e.g. removal of surface treatment
- Description and additional observations of sample e.g. presence of chipped edges

The input data for the OPI test recorded and used in computations are: the specimen dimensions; volume of permeability cell; and readings of pressure with time, which are imported into a spreadsheet from a pressure transducer using Observer II software. From the recorded data, a plot of ln (P_o/P) against time is made; a line of least squares is made through data points. The slope of this line (z) is used to determine coefficient of permeability. For a valid test determination, the R^2 of line of least squares should be more than 0.99. The test result (OPI value) is determined by obtaining the negative logarithm of the average of four test determinations (k-values), Equation 3.1.

$$\text{OPI} = -\log_{10}[\tfrac{1}{4}(k_1+k_2+k_3+k_4)] \tag{3.1}$$

The input data recorded for the water sorptivity test are: specimen dimension; mass of water gained over a period of 25 minutes, recorded at different time intervals; and saturated mass of specimen. A plot of mass gained against time is made and line of best fit made through the data points; slope of this line (F) is used to compute sorptivity (S) of a specimen by applying Equation 3.2. For a test determination to be valid, the R^2 should be equal to or more than 0.98. The porosity is also computed by considering the vacuum saturated mass and oven dry mass. A test result is obtained as the average of four test determinations.

$$S = \frac{Fd}{M_{sv} - M_{so}} \tag{3.2}$$

where: d – average thickness of specimen; M_{sv} – vacuum saturated mass; M_{so} – oven dried mass

The spreadsheets for computation of OPI and sorptivity values provide for checks on variability to enable the user to evaluate their performance of the tests. These checks are done through computation of:-

(i) *Repeatability* of test (CoV): based on four test determinations.

(ii) *Variability check*: based on comparing range in results with standard values – repeatability of test (Stanish et al., 2005).

From ASTM C 670 the maximum acceptable range of results is determined by applying a given multiplier to the repeatability. The multiplier used is based on number of test determinations. To evaluate the extent of variability, checks on maximum acceptable range are provided in the spreadsheets.

A test result has ‘high’ variability when the range in test determinations exceeds 4.6 times repeatability of the test. A second check indicating ‘caution’ is made by modifying the multiplier by two thirds (⅔*4.6 = 3.07). If the range does not meet the criteria described (i.e. variability is not ‘high’ or does not indicate ‘caution’), the variability check indicates that test result is ‘good’.

DI test values: data collection

DI test results were obtained from nine projects involved in the GFIP by way of the data being sent to UCT through an arrangement with SANRAL; this data was collated into one database for further analysis. A summary of the number of test results obtained from the projects is given in Table 3.2. Further analysis of these data was done in Chapter Four.

The data obtained were presented in either the UCT or SANRAL spreadsheets. Majority of the data obtained from the projects was recorded in SANRAL spreadsheets which provide a summary of data recorded in UCT spreadsheets. The details provided in these spreadsheets are:-

- Date of casting structure
- Structural element number and type
- Compressive strength measurements
- Durability index values – OPI, sorptivity and chloride conductivity values and if measured on cubes, structure or test panels
- Cover depth measurements

Table 3.2: Summary of number of DI test results (based on an average of four test determinations) obtained from GFIP project (raw data)

	DI test results	
Project	**OPI**	**Sorptivity**
1	185	165
2	99	80
3	23	22
4	127	110
5	14	14
6	91	91
7	43	43
8	18	18
9	136	136
Number of test results	736	679
Number of test determinations	2944	2716

Review of data recording

The following section provides a review of the observations made in data recording from the various sources of data collected; the review is based on data recording carried out using UCT spreadsheets.

The DI tests were developed to provide early age properties of concrete, at an age of 28 – 35 days. The recorded age in the spreadsheets indicates that tests were carried out, in some cases, on concrete samples at ages that range from 42 - 185 days. The testing of specimens at such late ages limits the effective application of the DI tests in quality control. Additionally, delayed testing of samples may result in higher values of OPI or lower values of sorptivity as the samples have been exposed to environmental moisture and undergone further hydration. The delay in carrying out the OPI test could be an indication of the lack of capacity in the laboratories to effectively carry out the tests within the required period.

The input data required to identify and characterize a test specimen includes the age of sample, curing method used and concrete mix properties; these aspects were however not always recorded in the spreadsheets. The reason for the missing data may be due to the fact that these aspects can only be reliably obtained from site. The age of casting a structural element, curing method applied on structures and details of the concrete mix should therefore be recorded on site, and such a record delivered to the laboratory with samples, for input in spreadsheets.

Another observation made from the spreadsheets is the predominance of missing or invalid sorptivity results. The summary of number of results in Table 3.2 indicates less sorptivity results in comparison to OPI results, yet the test is meant to be carried out on the same test specimen used in OPI. This observation indicates that difficulties are encountered during application of the test or it was omitted. The test results provided in the SANRAL spreadsheet format does not provide a record of porosity, thus analysis of these values is limited to those recorded in UCT spreadsheets.

The DI test values obtained from the nine projects are further analysed in Chapter Four using statistical methods (descriptive and inferential statistics) with the objective of determining the extent of variability in DI test values obtained from site elements.

3.2.2 Review of Laboratory audit report

The reliability of test results obtained from a laboratory is dependent on good practice which involves: the ability to follow test procedures correctly; knowledge and skill of operators in undertaking tests; maintenance of required test environmental conditions; proper documentation of test results; and calibration of test apparatus (Zimmerman, 2010). In addition, test methods used should also be clear and complete to avoid ambiguity and ensure correct interpretation of procedures (ASTM E -177).

The evaluation of precision of DI tests was previously undertaken through an inter-laboratory exercise reported by Stanish et al. (2006). From the exercise, variability (as indicated by repeatability and reproducibility) of OPI and sorptivity were observed to be within acceptable limits (reproducibility of 1.8% for OPI and 12.8% for sorptivity) while variability of chloride conductivity was high, which was attributed to apparatus used. The outcomes from the exercise were used to update and improve test statements at that time.

A submission of the test methods to the South African Bureau of Standards (SABS) for publication has been made; however, the publication was suspended due to the unresolved issue of test variability which still needs to be addressed and a requirement for accreditation of test laboratories to ensure their competence in execution of tests (Gouws, 2010; Gouws, 2011).

The Durability Index Focus Group resolved to address variability issues by carrying out an audit exercise, instead of a statistical validation process which is an expensive undertaking. The main aim of the audit was to identify variations in testing methods and equipment. To attain this objective the application of oxygen permeability index (OPI) test procedures in different laboratories was examined and evaluated to determine if tests are correctly executed, and the ambiguities and difficulties encountered in use of these procedures (Raath, 2011). In addition, experience of laboratories in application of tests was considered in order to evaluate best practices in different laboratories which can be employed in improvement of the test (Gouws, 2011). A review of the report from the audit exercise is given in the subsequent section.

3.2.2.1 Audit conducted on Durability Testing

The audit exercise was carried out in different participating laboratories (a total of 15) between the period of February 2011 and May 2011. Ceramic discs that had been developed for calibrating test apparatus were used to conduct the OPI test; these discs had been pre-tested for permeability by Contest Laboratories in Durban. The latest and official Durability Index Test method (2009) was provided to all participating laboratories, and the audit was with regard to test procedures in this method statement.

The procedure followed in carrying out the audit exercise, as observed by myself on 5 – 6th April 2011 during participation in the audit exercise in laboratories in Gauteng, involved the following stages:-

i. A seven day notification to the participating laboratory to ensure that ceramic discs are pre-conditioned for OPI test, by drying in oven at 50°C and cooling in desiccators for at least two hours prior to commencement of test, as per the test procedures.

ii. Visit by auditor to the laboratory where observations were made on the following aspects:-
    a) Execution of test procedures by operator using ceramic discs to assess approach used in: measuring specimen dimensions (diameter and thickness) and assembly of specimen in OPI

test apparatus (rubber collar, rigid sleeve, solid ring, cover) which is placed on top of the test chamber and tightened with the top screw.

b) Equipment used in: preparation of test specimen (core barrel, holding device, cutting saw); specimen pre-conditioning (ovens, desiccators); and for the OPI test (permeability cell, rubber collars, pressure gauge and transducers).

c) Handling of test specimens received from site to determine if they are properly identified on receipt, storage of samples prior to testing in laboratories, and if proper records for identification of specimen received (date of receipt, condition of samples when received) are kept by the laboratories.

iii. The last stage of the audit involved filling in a checklist of questions based on the Durability Index Testing Procedures Manual with regard to specimen preparation and OPI test. The auditor would ask the test operator questions in the checklist which required ‘yes’ or ‘no’ responses. Additional notes from personal observations made by the auditor during the audit exercise were also recorded against the questions in this checklist.

A copy of the audit checklist used is provided in Appendix B.

3.2.2.2 Part 1: Standard procedure for preparation of test specimen

Summary of Part 1

The test specimens are circular discs of 70 ± 2 mm diameter and 30 ± 2 mm thickness that are obtained from cubes or site elements 28 to 35 days after casting. The maximum nominal aggregate size of specimen should not exceed 26.5 mm. A water-cooled diamond tipped core barrel with a nominal inner diameter of 70 mm is used to obtain cores; direction of coring must be perpendicular to casting direction. For cubes, coring should be done the entire way through the cube ensuring that far side is reached.

A holding device that firmly holds the cube specimen in place should be used to ensure that the sides of the core obtained will be parallel. The first 5 mm from the cored face should be removed by cutting or grinding and discarded. Using a suitable water-cooled saw, cores are cut to required thickness. Damaged specimens e.g. where aggregates gets dislodged from test surface during the cutting and coring process should be discarded. Cores obtained from site elements should be properly packaged during transport to protect from conditions of adverse drying and damage due to rough handling. Specimens obtained from site should be kept at ambient conditions in the laboratory for a maximum of 3 days.

Observations from audit exercise on preparation of test specimen

The discs used in carrying out tests did not have aggregates larger than 26.5 mm as specified in test procedures, an aspect which all labs were aware of. The diameter and thickness of test specimen should be measured at four points evenly distributed; however, the diameter was in some cases not always measured at the greatest point while the thickness was not measured at four equidistant points on circumference. Test specimens with deviations in dimensions, as much as 4 mm in thickness, which exceeds acceptable tolerance in procedures were observed. In obtaining measurements, edges that were damaged during drilling were avoided.

The diameter of the cores differed in laboratories due to variations in diameter of the core barrel; a discrepancy was found to exist on whether the diameter of 70 mm related to inner or outer diameter of the core barrel; this resulted in differences in diameter of specimen obtained. For some laboratories, it was difficult to fit in the ceramic disc (diameter of 70 mm) as the collars used accommodate cores of smaller diameters (68 ± 2 mm) as provided in previous test procedures. The outer 5 mm of cores was discarded and test specimen marked on the edge of the disc.

The suitability of holding devices used to clamp cores for drilling varied; some devices allowed movement of core during drilling which would affect the attainment of parallel sides of drilled core. Site samples were mainly obtained from test panels. The coring of panels was not always done in a perpendicular direction to that of casting, an aspect that was pointed out to test operators with clarification of the importance on direction of coring. The use of a smooth diamond saw blade resulted in well trimmed discs with less chipped edges in comparison to the notched type.

The discs that were damaged during the coring and cutting process were seldom discarded as there is no guideline with regard to the extent of damage that is permissible and which would result in discarding of specimen. Two labs were found to wash test specimen after cutting to clean off the paste on the surface – an aspect that is not provided for in test procedures.

Limitations that were observed from use of these panels, the main source of test specimen, include:-

a) Variations in dimensions of panels e.g. thickness from one end to another which does not permit drilling of perpendicular cores, and in some cases failed to allow drilling of four cores.
b) Test panels from which samples were obtained were frequently transported to the laboratory for coring. However, no routine or set procedure was available for transport of panels to lab for testing; this resulted in delays in transport of panels and coring was done later than specified time period.
c) Lack of proper identification of test panels e.g. the casting date and elements of the structure which they represent were not always marked on panel.

The storage of panels in some laboratories was not properly done e.g. haphazard stacking of panels horizontally or vertically against each other without leaving a space between them.

3.2.2.3 Part 2: Standard procedure for OPI test

Summary of Part 2

The apparatus used in carrying out the test include:-

- Oven capable of maintaining temperature of 50 ± 2ºC which is used in pre-conditioning test specimen for a period of 7 days ± 4 hours.
- Permeability cell with a volume of 5L and tolerance of ±5%; this cell should be regularly tested for air-tightness using impermeable test specimen.
- Compressible rubber collars with Shore Hardness 39A that fit tightly around specimen ensuring a tight fit and eliminating leakage. These collars should be regularly checked and replaced when tears and cracks occur.
- Desiccators that are large enough to hold as many specimens that are tested simultaneously. Cooling of specimen in desiccators should be done over a period of no less than 2 hours and not more than 4 hours.

The conditions in the laboratory should be maintained at temperatures of 23 ± 2ºC and relative humidity of 60%.

The specimen is placed in a compressible collar within rigid sleeve, with test face (outer face of core) at the bottom and resting against the lip of the collar. The rubber collar is then fitted within the rigid sleeve and checks made to ensure that there are no gaps between the two. The sample, collar and rigid sleeve are then placed on top of test chamber to cover the top of permeability cell. A solid ring (optional) may be placed on top of the collar. The cover plate is then centered and the top screw tightened – first finger tight then one and a half revolutions using a spanner. The inlet and outlet valves in permeability cell should be open for a short period (approximately 5 seconds) to allow flow of oxygen gas which ensures purging of gases other than oxygen from chamber.

The test is commenced with a pressure of 100 ± 5kPa. The initial time and pressure should be recorded, and readings of pressure obtained after 5 minutes, where manual means of recording are used; in most cases automated readings are obtained where pressure readings are recorded after every 2 minutes using the Observer II software. Readings from this software are exported to an Excel spreadsheet developed in UCT for computations. The test is terminated when pressure has dropped to 50 ± 2.5kPa or after 6 hours ± 15 minutes, whichever happens first.

Aspects of test specimen that should be reported are those outlined in Section 3.2.1.

Observations from audit on standard procedure for OPI test

The volume of permeability cells had not been determined in any of the participating labs; they were advised to do so by filling the cells with water and measuring the mass. The permeability cells in the different laboratories were regularly checked for leaks using an impermeable disc. Laboratory temperature and humidity conditions were well recorded and controlled with the exception of site-based labs which found it difficult to control lab environment conditions. Majority of the laboratories did not have calibration records for pressure gauges due to low pressures required for the test which restricts calibration of the gauges by metrology laboratories.

The ovens used in laboratories were standard and had adequate temperature measurements. The preconditioning of samples was carried out as per test procedures; however, relevance of the tolerance of ± 4 hours for oven drying and maximum cooling period of 4 hours was questioned. These tolerance periods were found difficult to practically implement especially where they were many specimens to be tested. The specimens were correctly placed in the oven and records kept of when they were placed and taken out from the oven. In some cases, samples were left in the oven for periods longer than those specified due to shortage of OPI test apparatus. One laboratory deviated from the pre-conditioning procedure by drying specimen in oven and weighing discs until they reached a constant mass before carrying out the testing.

The placement of the disc with the outer face against lip of rubber collar was correctly done in most laboratories, with the exception of one.

Variations and difficulties observed with assembly of test apparatus (placing sample in rubber collar, use of rigid sleeve and solid ring) in different laboratories include:-

a) Use of a plastic rigid sleeve, which was shorter than the rubber collar; such a sleeve would offer a different degree of restraint in comparison to one made from steel.
b) In some laboratories a rubber O-ring was placed in the groove and sealed against the rubber collar, in other labs this ring was not used.
c) Different diameter of perforated steel disc, which is placed above rubber collar as a cover plate, where in some cases it was smaller than the rigid sleeve which enabled it to be placed directly on top of rubber collar, while in other cases the steel disc had a larger diameter and a wider solid ring had to be placed on rubber collar to transmit load. Some laboratories did not have a solid ring as part of the apparatus and were unaware of its purpose.
d) In some cases insertion of rubber collar into the rigid sleeve or test specimen into the rubber collar was extremely difficult. To resolve this some laboratories applied grease to the interface; they were however advised against doing this.
e) The hardness of the rubber collars differed which results in a variety of collars that have different resistance under applied loads.
f) Variations were observed in tightening of the top screw where in some labs it was done by hand while in others with the use of a spanner; this difference in tightening would result in variation in applied force.

The record of pressure change was done at intervals of 15 minutes in one laboratory; with the rapid pressure drop encountered with ceramic discs it resulted in few measurements being made to enable computations to be carried out. In another lab, measurements were taken at intervals of 30-seconds which resulted in too many measurements. Differences were observed in most laboratories between readings in automatic pressure transducer and pressure gauge.

The details on specimen tested such as concrete properties (binder type, water/cement content), curing history were not known and hence not reported by the laboratories; these aspects can only be reliably obtained from sites from which samples are obtained. Attempts by laboratories to obtain these data after several enquires to site supervisory staff were however unfruitful. Some laboratories did not carry out any testing on unidentified test panels while others did.

General observations

The range of OPI results obtained from testing of ceramic discs in different laboratories was 8.94 – 9.24, with an average of 9.125. These results compared favourably with those previously obtained from testing the ceramic disc in Contest laboratories where a range of 9.012 – 9.247 was obtained with an average of 9.095; a summary of these values is given in Table 3.3. The values observed had a low variation in comparison to OPI values from pre-tested ceramic discs. This indicates that low variability in test results can be obtained despite the variations observed in test method application and equipment.

Table 3.3: OPI values obtained from testing ceramic discs (Raath, 2011)

	OPI (log scale)		
	Average	Min	Max
Pre-test values	9.095	9.012	9.247
Audit exercise	9.125	8.940	9.240

The test panels or cubes used in obtaining test specimen were observed to be poorly made and this was attributed to be the main cause of variability in results of site-based samples. There was a prevalence of missing information with regard to test panels such as: lack of proper identification with regards to structural elements to which panel relates to; failure to provide casting dates of panels, details on concrete mix used and curing methods.

The laboratories generally expressed reluctance in investing in more equipment for carrying out the OPI tests until demand for this test increases. In addition, poor response by site supervisory staff to results discouraged lab staff as the purpose for undertaking the test was not perceived. To address the variability issues currently encountered a standardized approach in manufacture and proper identification of test panels should be developed. Further training of the supervisory site staff is required to increase awareness on importance of undertaking the durability tests and the underlying concepts of the approach.

3.2.3 Questionnaires to Resident Engineers

To obtain the perception on the implementation of durability index based specifications on site, a questionnaire was sent out to resident engineers (REs) involved in projects of the GFIP; responses were obtained from seven REs.

The questionnaire was sent out to REs with the objective of obtaining information on:-

- Construction processes used (compaction and curing process)
- Source of samples used for testing (cubes, test panels or actual structure)
- A record of sample age
- Checks carried out to ensure correct placing of concrete
- Problems encountered in implementation of the DI-based specifications
- Improvements that have taken place in construction processes due to implementation of DI based specifications

The questionnaire used is provided in Appendix B.

The construction processes employed on the sites were: compaction through the use of vibrators; curing through retention of formwork and on removal, application of curing compounds. Samples used for testing were obtained from either cubes (for samples obtained in earlier periods before revision of project specifications) or test panels, and for precast samples from the actual precast units.

The responses from all the REs indicate that checks had been carried out on cover depth provisions on all sites, before placing concrete by ensuring that concrete spacers are properly placed and after placing of concrete through cover surveys. One resident engineer indicated difficulties encountered in use of cover meters which result in many readings in one spot, and difficulties in interpretation of these readings; he highlighted the need to standardize cover meters to ensure fair and accurate assessment.

The practical difficulties that were encountered in application of the DI-based specifications on site included:-

(a) Lack of a standardized procedure in the manufacture of test panels, which may have occurred due to introduction of use of test panels as projects were ongoing. There was also an observation on failure to properly handle the panels on site which may lead to poor results being obtained.

(b) Difficulty was encountered in transportation of test panels from site to the laboratory for testing due to their large size, and also poor communication between the contractor and laboratory on when to collect panels for testing.

(c) The amount of testing for quality control has increased, where in addition to strength measurements samples are obtained for DI tests; this increased amount of testing was found to be difficult to handle, especially due to different equipment and properly trained staff required for DI tests.

Three out of the seven respondents felt that implementation of DI-based specifications had no effect on construction processes which generally remained the same; the changes identified were in the concrete mix from concrete producer where the cement content was high leading to higher strength values than that specified. However, three respondents felt that the specifications had resulted in strict controls in construction processes and more care taken in handling of concrete – the placing, compaction and finishing.

Some confusion on how to carry out the tests was observed where one respondent stated that durability tests are carried out on concrete that fails to attain specified strength values.

3.2.4 Concrete mix properties

The penetrability of concrete cover is influenced by the material constituents of the concrete mix – the water/cement ratio, binder type and contents. The concrete supplied to some of the projects involved in the GFIP was obtained from a reputable Ready Mix Concrete producer in Gauteng. A summary of the range of concrete mix properties - water/cement ratio and binder content obtained from testing laboratory samples is given in Table 3.4; additional details on the concrete mix constituents are given in Table B1, Appendix B.

Table 3.4: Summary of the range of concrete mix properties from four plants of Readymix concrete producer

Binder content (kg/m^3)				Water content (L/m^3)	Water/binder ratio
OPC	Fly ash	GGBS	Total		
383 - 403	68 - 71	-	451 - 474	184 - 208	0.41 - 0.44
360 - 373	-	90 - 93	450 - 466		0.44 -0.45

The mix properties of concrete used in precast elements were obtained from precast manufacturer are summarized in Table 3.5. Additional properties of this concrete mix are given in Table B2, Appendix B.

Table 3.5: Summary of mix proportions of concrete used in production of precast elements

Mix constituents	Proportion (kg/m^3)
Cement	410
Fly ash	176
Total binder content	586
Water content (L/m^3)	220
Water/binder ratio	0.38

From the summary of mix proportions of concrete used for site and precast elements the following observations are made:-

(i) Low water/cement ratio was used in concrete mixes with a maximum ratio of 0.45 for the Ready Mixed concrete and the lowest ratio of 0.38 for concrete used by precast manufacturers. A low water/cement ratio is preferable for durability as it leads to a lower volume of voids in the cement paste and a reduction in penetrability of concrete (Neville, 2007).

(ii) The binder content used in both concrete mixes is high, with as much as 586 kg/m^3 for concrete mix from the precast manufacturer. From the SANRAL Project specifications, binder content of as much as 400 kg/m^3 were permitted to meet durability criteria. When binder content exceeded 400 kg/m^3 but were below 450 kg/m^3 adjustments in payments for binder content were made; no payments were made for binder contents exceeding 450 kg/m^3.

3.2.5 Concrete cover

The concrete cover is defined as the distance between the face of the concrete surface and of the outermost reinforcement. The extensive deterioration of reinforced concrete structures currently observed in concrete infrastructure worldwide can be largely attributed to inadequate cover depths. Many cover depth survey studies done on structural elements (bridges and buildings) have shown that the majority of the structures had inadequate cover depths and that the extent of variability was significant (Marosszeky and Chew, 1990; Clark et al., 1997; Sharp, 1997; Ronne, 2005). Inadequate cover depth is therefore a chronic problem that needs to be addressed.

It is difficult to assign a specific cause for lack of adequate cover depth, but some of the reasons are:-

(i) *Design faults* e.g. inadequate specification of cover depth where a designer fails to fully grasp the severity of exposure conditions and makes provisions for a cover depth that is less than that which would ensure adequate protection to reinforcement from corrosion (Neville, 1998).

(ii) *Poor construction practice* which leads to failure in attaining specified cover depth on site, for example due to: inadequate provision of cover spacers to ensure that concrete placed attains the specified depth; faults in formwork e.g. movement during placing of concrete or incorrect placing of formwork (Marosszeky and Chew, 1990) ; lack of care by operatives e.g. standing on reinforcement cages which may cause misalignment of steel bars (Neville, 1998); failure to properly fix reinforcement by steel fixers (Clark et al., 1997).

(iii) *Poor structural detailing* where the reinforcement provisions by the structural engineers are not practical for application on site e.g. provision of too much reinforcement in an element which may lead to congestion in the member and difficulty in placing and compaction of concrete (Clark et al., 1997; Neville, 1998). In addition, the tolerance provided for in some cases has been described as very onerous and difficult to practically apply in construction (Sharp, 1997).

To address durability concerns, some designers may opt to increase the cover depth of a member. This however does not alleviate durability problems as with increased depths there is a greater incidence of cracking in concrete, which increases the penetrability of concrete, in addition to increased costs due to larger member size (Gouthaman and Menon, 2001).

The recommended approach however to address the variability and incidences of shortfall below specified cover depth values is by carrying out strict controls on quality where cover depths in as-built structures are verified for compliance. This observation was made through a quantitative analysis that highlighted the lower variability of cover depth readings in bridges, where controls on concrete cover were undertaken, in comparison to buildings (Marosszeky and Chew, 1990). The controls that may be undertaken to ensure that the specified depths are met involve verification of cover depth: (a) before placing of concrete through inspection of adequacy of cover spacers that should be of the correct depth and properly placed; (b) after placing of concrete through a cover survey using cover meters.

Cover depth provisions

The concrete cover used in structural design and detailing is the nominal cover depth (C_{nom}) which is obtained by adding the minimum allowable deviation (Δ_c) to minimum cover depth provisions (C_{min}). The minimum cover depth is provided to ensure protection to reinforcement for durability, safe transmission of bond forces and fire resistance (BS EN 1992-1). In addition, a maximum allowable deviation (Δ_{plus}) and minimum allowable deviation (Δ_{min}) on nominal cover is provided which depends on the structural member depth. These values for cover depth provisions are illustrated in Figure 3.3.

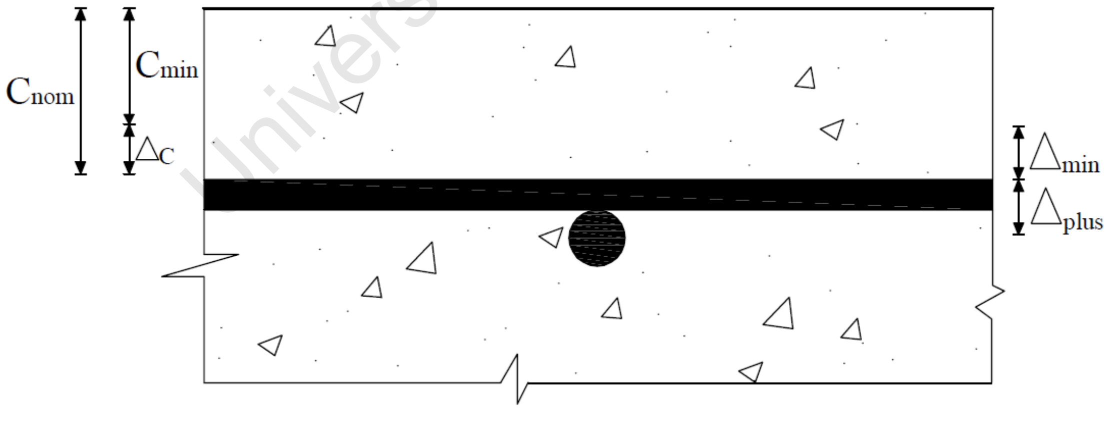


Figure 3.3: Schematic representation of cover depth (EN 13670-1).

The allowable deviations (tolerance) for cover depth provided in standards, ACI 318M-08 and EN 13670-1, are summarized in Table 3.6. There have been proposals to increase the tolerance so as to deal with failure to attain specified cover depth e.g. Sharp (1997) evaluated the effect of increasing tolerance from 10 mm to 15 mm and observed that the shortfall in measurements below the minimum value specified reduced from 10% to 5%. Clark et al. (1997) however argue against such an increase in tolerance as this may result in a greater proportion of defects that are unassignable and fail to be detected. It may also create the perception of a relaxation on expected performance.

Table 3.6: Tolerance levels for cover depth (ACI 318M-08 and EN 13670-1)

ACI 318M-08		EN 13670-1
Minimum cover tolerance Δ_{min}	$d^a \le 200$ mm : 10 mm $d \ge 200$ mm : 13 mm	10 mm
Maximum cover tolerance Δ_{plus}	-	$h^b \le 150$ mm : 10 mm $h^b = 400$ mm : 15 mm

[a] d: effective depth of structural element; [b] h: height of cross-section

Cover meters use electromagnetic methods to determine the location and cover to reinforcement embedded in concrete. Luco et al. (2005) carried out a comparative study on cover meters to evaluate the factors that influence accuracy in readings of cover depth. From the study, it was observed that cover meter readings are not influenced by temperature, moisture condition or water/cement ratio of concrete but are dependent on the cover depth of a structural member; with an increase in depth, the accuracy of readings obtained decreased.

Cover depth readings: GFIP

The project specifications in GFIP require the provision of cover spacer blocks before placing of concrete, and cover depth measurements of concrete cover of all reinforced concrete structural elements (those designated as Class 'W' durability concrete and normal reinforced concrete structures). The limiting value for cover depth for durability requirements in the project is 40 mm for in situ and precast elements; acceptable ranges of cover depth, allowing for tolerance, are summarized in Table 3.7. The minimum cover allowable in the specifications is low e.g. 28 mm on the individual bar. This low cover depth would have an implication on effective service life of a structure which is related to cover thickness; Equation 2.5. A reduction in cover reduces the service life (t) of a structure.

Table 3.7: Acceptance range for concrete cover (SANRAL, 2010)

	Minimum		Maximum	
Specified cover	Individual bar	Overall	Individual bar	Overall
40 mm	70 % specified	0.85*(Specified cover – 5mm)	Specified cover + 25 mm	Specified cover + 15 mm
Acceptable values	28 mm	30 mm	65 mm	55 mm

The cover depth surveys were carried out using Hilti Ferroscan PS 200 cover meter, illustrated in Figure 3.4. The meter contains a scanner that is moved across the surveyed area. Readings obtained from the scanner are then transferred to the monitor from which location, depth and size of reinforcement can be determined.

Figure 3.4: Cover depth meter Ferro scan PS 200 (Hilti, 2011)

The survey was carried out on a scan area of approximately 1 m^2, randomly distributed over the entire structure. The scanned area should represent at least 5% of the total surface area of the structure. An individual bar reading was obtained as a minimum of three readings along a single bar spaced at 100 mm intervals. An average cover per scan area was obtained as the average of at least 16 individual readings. The overall cover depth for a structure was then obtained as the average of cover readings obtained from the scanned areas of a structure.

The scans obtained with the cover meter could either be:-

(i) Quick scan which can be carried out over a large section of a structure and covers lengths of up to 30 m; the scan is carried out at right angles to rebar. Figure 3.5 (a) illustrates a quick scan carried out over a length of 2.325 m. The spaces in the figure represent distance between reinforcement bars while the vertical lines indicate distance of a bar from the concrete surface.

(ii) Image/block scan which is carried out over a grid area, 600×600 mm and provides an image of reinforcement. Figure 3.5(b) illustrates a block scan made along outermost reinforcement bar; four readings are taken on each bar and a total of four bars are considered in this grid making a total of 16 readings over a scan area of 1.2m^2. The numbers in the figure (1 – 16) indicate the position of obtaining a cover depth measurement.

It is recommended that the block scan is carried out to verify cover depths where more than 10 % of readings obtained with the quick scan are below the specified lower limit. Additional information on cover depth provisions are provided in extract of project specifications in Appendix A.

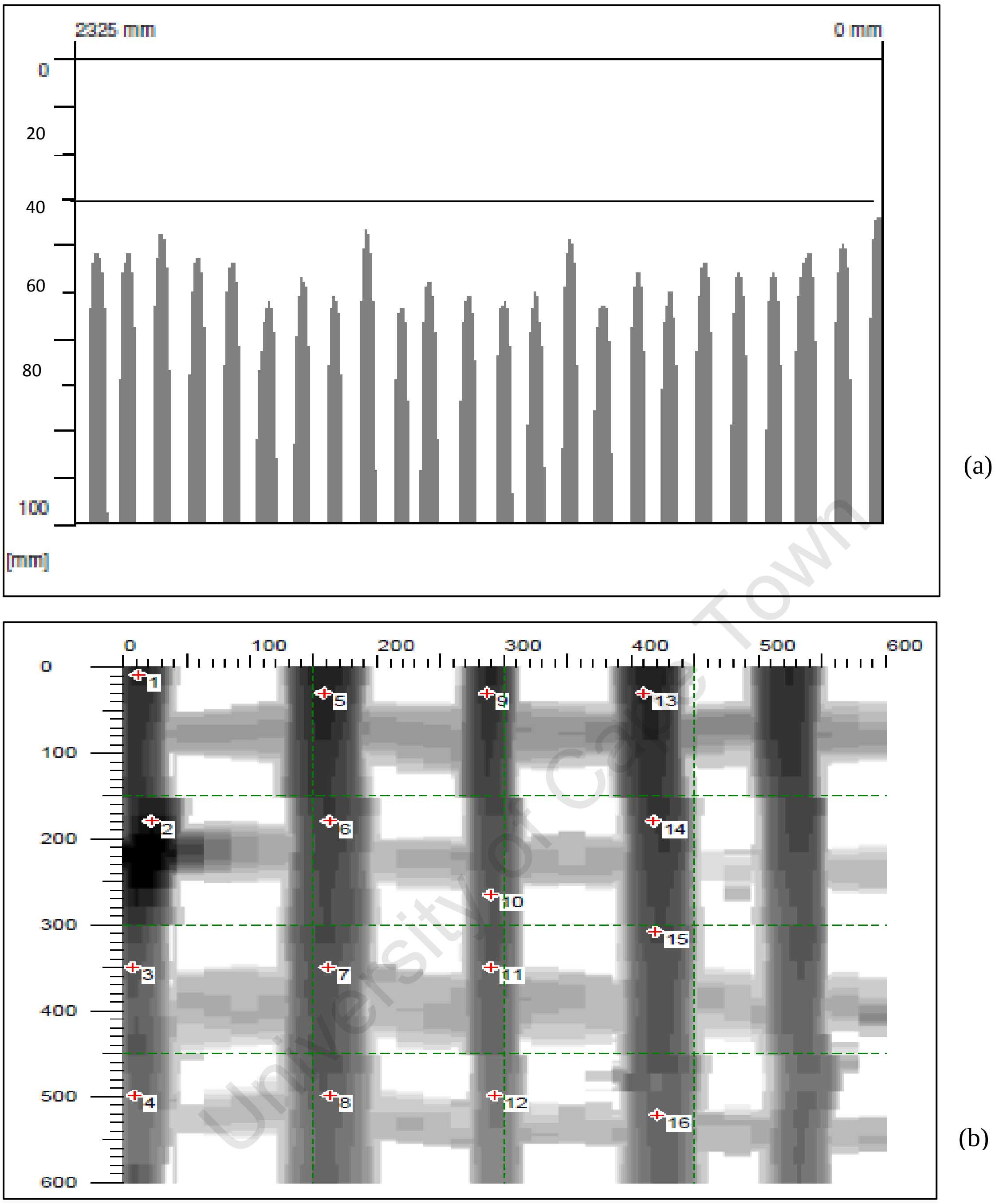


Figure 3.5: Schematic representation of scans obtained with cover meter (a) Quick scan (b) Image scan.

Cover depth readings obtained for the study were obtained from two sources:-

(i) In-situ structures from a project in the GFIP; structures on which surveys were done included bridges, culverts, retaining walls, gantries.

(ii) Precast concrete barrier elements which are used as median barriers on the freeways.

For the in-situ structures, the cover survey procedure is that provided in the project specifications and briefly outlined above. The cover survey procedure used for precast elements (illustrated in Figure B1, Appendix B) was however different; it involved obtaining three readings each from the top of the element, from the front – top, and from the front – bottom, making up a total of nine readings from which an average was obtained. These locations were selected as they would be more exposed to environmental conditions. A schematic of obtaining cover depth measurements for these elements is given in Figure 3.6.

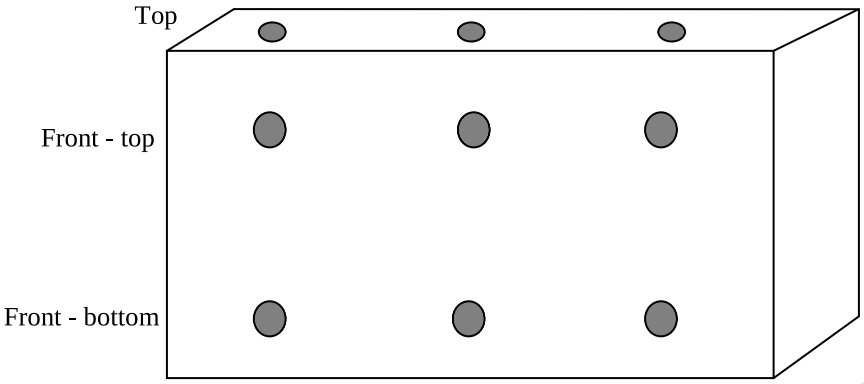


Figure 3.6: Schematic of procedure used to obtain cover depth measurement of precast elements (shaded circular regions indicate where cover depth measurements were made).

A summary of number of readings obtained and used for data analysis of cover depth values is given in Table 3.8.

Table 3.8: Summary of number of individual cover depth readings (raw data)

Source	Number of readings
In situ structural elements	2533
Precast elements	1493

3.3 STATISTICAL METHODS USED IN DATA ANALYSIS

Statistics involves the collection, organization, summarization and analysis of data in order to draw scientifically objective conclusions (Peck and Devore, 2008). Data collected for statistical analysis is obtained from a sample of the population; a sample should be properly selected to ensure it is representative of the population under study. Descriptive statistical methods are used to organize and summarize data with the use of graphical and numerical methods; data description provides insight on important characteristics of data. To draw conclusions from a sample, inferential statistics is applied where hypothesis tests based on a sample are used to learn more or draw conclusions about certain parameters of a population.

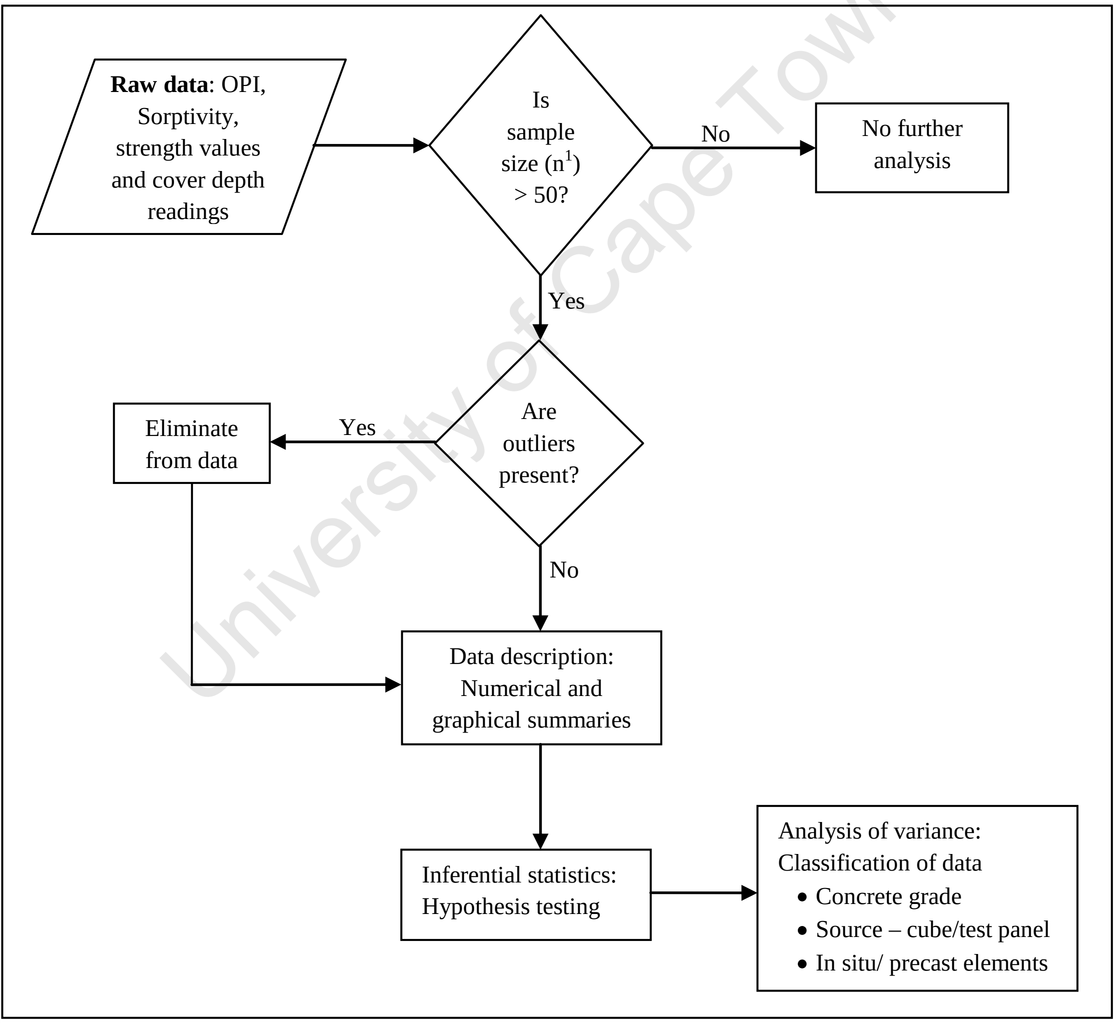
The analysis of data (DI test values, cover depth readings and strength values) was carried out by applying the process illustrated in Figure 3.7; this process is further described in subsequent sections.

3.3.1 Selection of sample size

The initial step in the data analysis process involved selection of a suitable sample size to use for data analysis. The size of data samples ranged from 14 to 185 values for DI test values (based on an average of four test determinations). A large sample size is more representative of a population; therefore the sample size selected and used for data analysis was based on 50 or more readings. A summary of the projects and the corresponding sample size (n) used for analysis is provided in Table 3.9.

Table 3.9: Summary of projects with selected sample size used in data analysis

	Sample size (n)	
Project Identity	OPI values	Sorptivity
1	185	165
2	99	80
4	127	110
6	91	91
9	136	136



^{1}n – number of test results (as provided in Table 3.2 for DI values and Table 3.8 for cover depth readings)

Figure 3.7: Schematic representation of data analysis process for DI test values.

The projects which had a sample size of less than 50, and were not used in data analysis are summarized in Table 3.10.

Table 3.10: Summary of projects with sample size less than 50

Project Identity	Sample size (n)	
	OPI values	Sorptivity
3	23	22
5	14	14
7	43	43
8	18	18

3.3.2 Elimination of outliers

Outliers are defined as observations that deviate excessively from others (either too large or small); they are not due to random variation in data. They may arise from errors in recording of data (transcription errors), or failure to properly carry out test procedure which leads to wrong results. The reason for an outlying result should be identified before it is eliminated from analysis, where this is possible.

The use of *box plots* was applied to identify and eliminate outlying results; Figure 3.8 illustrates the features of a box plot. A box plot presents the median (middle value), first and third quartiles of a sample of data. The difference between the first and third quartile is defined as the interquartile range (IQR) and provides an indication of variability in data. A line extending from the first and third quartile to the most extreme value that is not an outlier is known as a whisker. Any value that is more than 1.5 IQR above the third quartile or more than 1.5 IQR below the first quartile is considered an outlier (Navidi, 2008). An extreme outlier is a point that is more than 3 IQR from the first or third quartile.

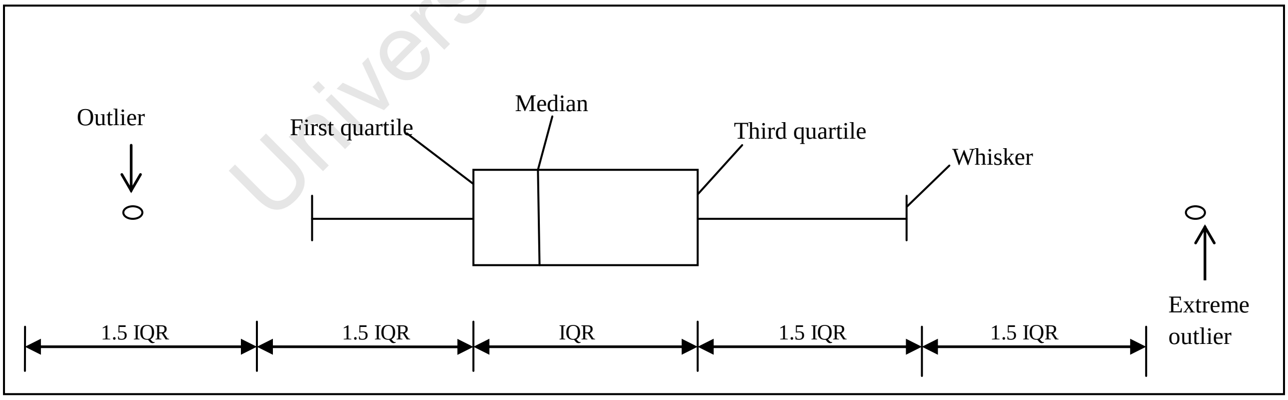


Figure 3.8: Schematic representation of main features of a box plot (Navidi, 2008).

The identification and elimination of outliers in analysis of data using box plots was carried out using the computer program MATLAB.

Box plots provide a visual display on the amount of variability within a sample of data in addition to comparison of different samples to evaluate if differences between or among samples are significant, by identification of those that have higher spread (Sheskin, 2007); illustration of a comparative box plot is provided in Figure 3.9 where OPI values of different strength grades from Project 6 are considered. The dotted line in the figure represents the limit value in specifications of 9.7.

3.3.3 Data description (exploratory data analysis)

The description of data involves computation of numerical measures or graphical summaries in order to present and summarize data. It allows the essential features of data to be presented clearly and concisely for immediate use of an analyst and provides guidance in selection of suitable methods for further analysis (Sheskin, 2007; Peck and Devore, 2008; Ross, 2009).

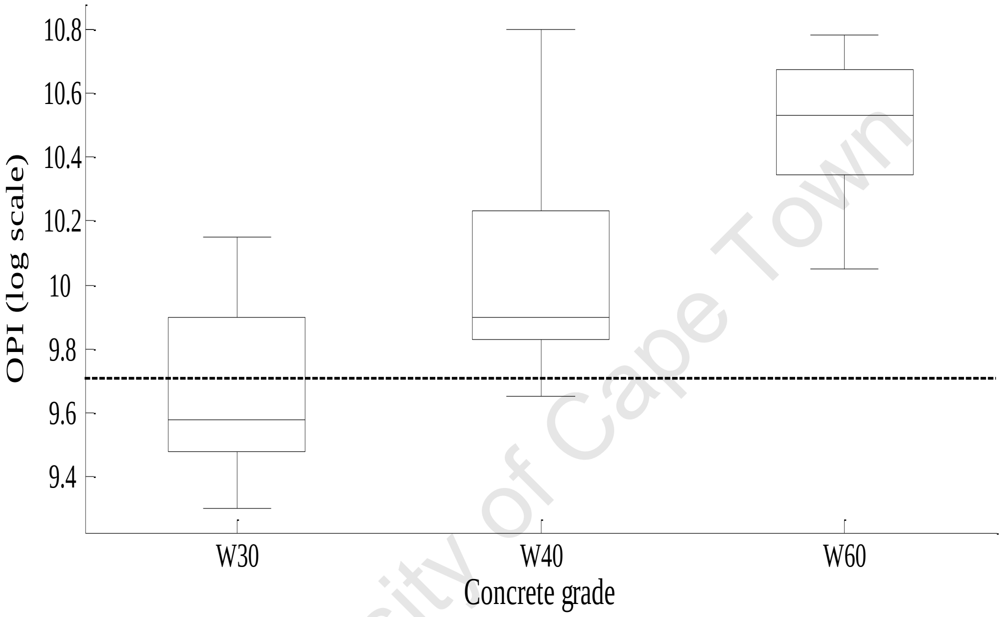


Figure 3.9: Comparative box plot of OPI values for different concrete grades (Project 6).

Numerical methods of describing data were computed through a measure of central tendency which involves determining the average value ($\bar{x}$) and indicates centre of the data, computed using Equation 3.3.

$$\bar{x} = \frac{\sum_{i=1}^{n} X_i}{n} \tag{3.3}$$

Where n is the sample size

A measure of dispersion provides an indication of the degree of spread or variability in data. This measure can be provided by computing:-

(i) Range which is the difference between the largest and smallest value in data.

(ii) Standard deviation (s) which measures amount of spread by considering the difference in sample values from the sample mean; it is computed using Equation 3.4.

$$s = \sqrt{\frac{\sum_{i=1}^{n} (x_i - \bar{x})^2}{n-1}} \qquad (3.4)$$

(iii) Coefficient of variation (CoV) determined by dividing the mean value with standard deviation and expressed as a percentage, Equation 3.5. This value is useful in comparing data sets with different sample sizes.

$$CoV = \frac{s}{\bar{x}} \times 100 \qquad (3.5)$$

Graphical methods applied in data summaries provide a visual display of the patterns and trends of data – its variability, distribution and can also be used to evaluate the relationship between two variables. Graphical methods used include histograms, box plots and scatter plots.

A *histogram* presents a graphical display of a sample and is obtained by grouping data into intervals (bins/class) and determining the frequency in each interval. The number of intervals used is approximately determined as,

$$\text{Number of intervals/bins} \approx \sqrt{n} \qquad (3.6)$$

where, n is number of observations

On the vertical scale for the histograms plotted, the density was used which is determined by dividing relative frequency with class width (Equation 3.7).

$$\text{Density} = \frac{\text{Relative Frequency}}{\text{Class width}} \qquad (3.7)$$

Histograms provide a visual display of distribution of data and indicate the amount of spread (scatter) in values (Montgomery and Runger, 2007). The distribution of data as indicated by a histogram may be symmetric where values are equally distributed about the central value - this indicates normal distribution of data; or skewed which may be positively skewed where the upper tail stretches farther than the lower tail or negatively skewed where the lower tail stretches farther than the upper tail. To determine the distribution model of a sample of data, a distribution curve was fitted onto the histogram using the *dffittool* in MATLAB; this is illustrated in Figure 3.10.

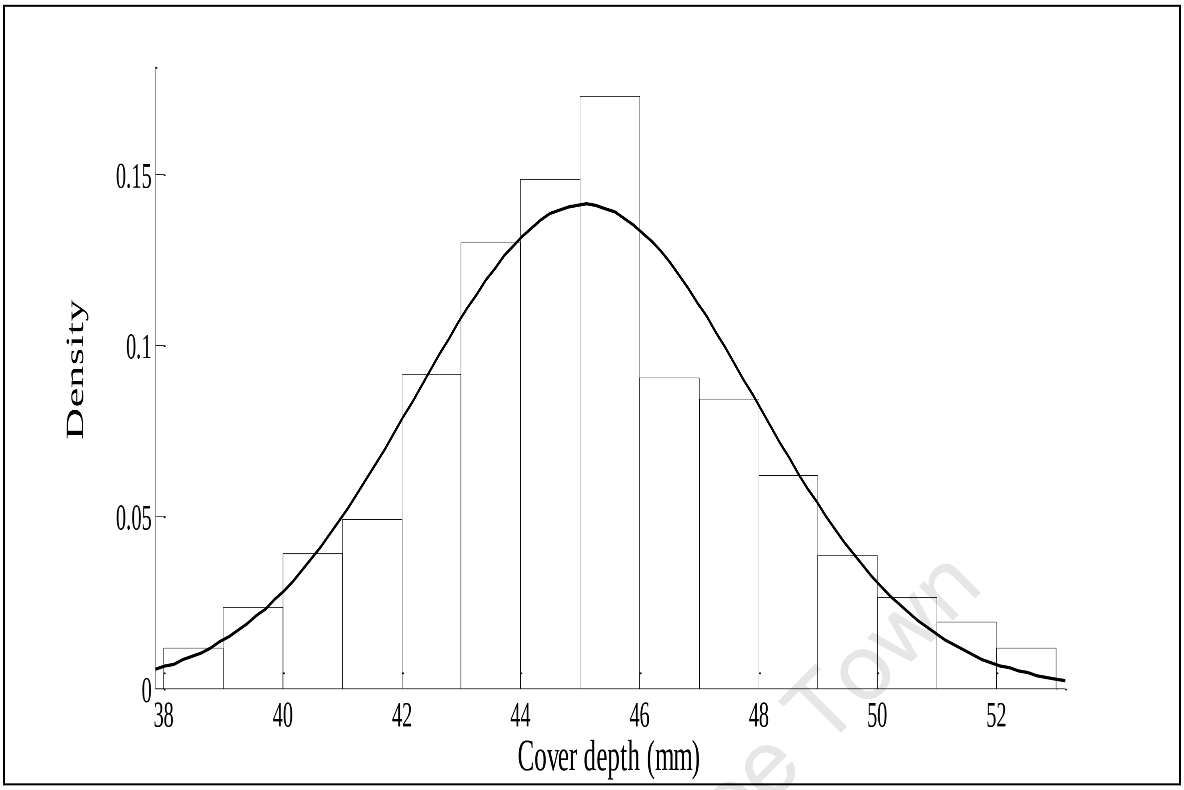


Figure 3.10: Histogram of precast cover depth readings.

A scatter plot is used in the comparison of two sets of data to determine if a relationship exists (Rice, 2007). The scatter plot was used in the comparison of strength and DI test values (OPI and sorptivity) as illustrated in Figure 3.11.

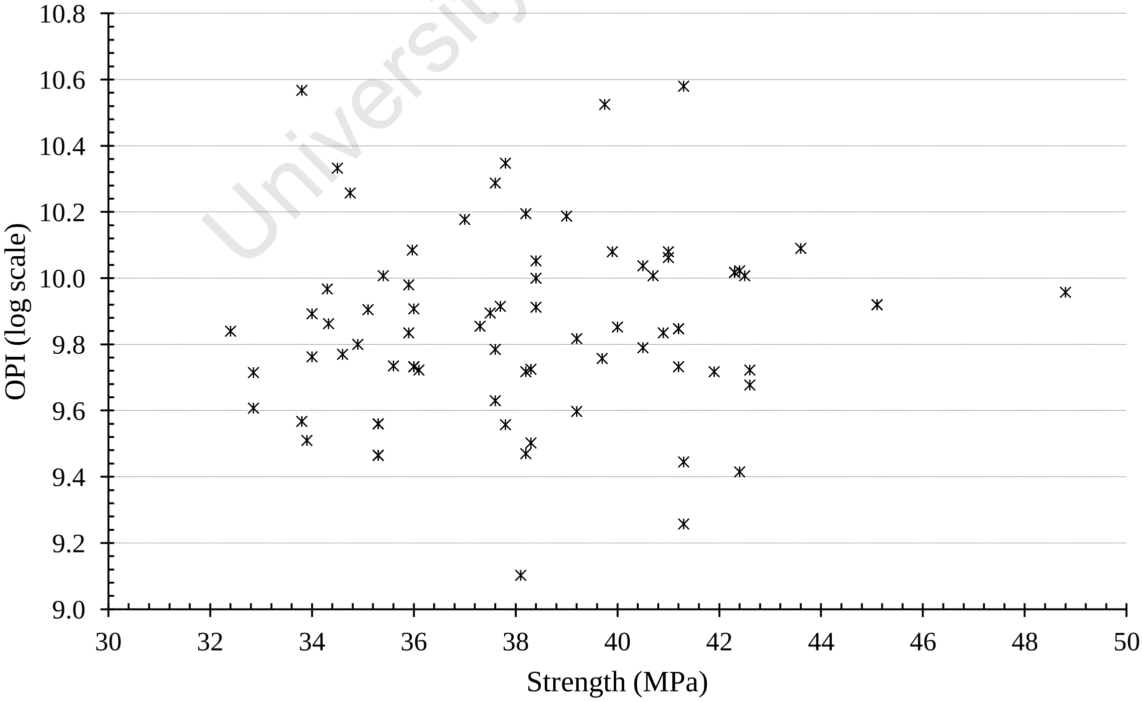


Figure 3.11: Scatter plot for comparison of strength with OPI values for Project 4.

3.3.4 Inferential statistics

This involves the use of a sample to infer or draw conclusions about certain parameters of interest from a population by hypothesis testing. A statistical hypothesis is a statement about the parameters of one or more populations (Montgomery and Runger, 2007). A null hypothesis (H_o) is a statement about a population parameter that is initially assumed to be true while an alternative hypothesis (H_1) is the competing claim.

In carrying out a hypothesis test using a sample, there is always an element of risk as the sample is not fully representative of the population; this may lead to a wrong conclusion. The probability of making a Type I error (defined as rejecting the H_o when it is true) is known as the significance level (α). This level of significance is usually pre-defined depending on permissible degree of uncertainty or risk. The observed significance level or smallest value of α at which data are significant is known as the P-value. This value provides more information about the data as it enables decision makers to determine how significant data is without imposing a pre-selected level of significance.

The P-value provides a quantitative measure of the probability of drawing a sample whose mean value is less than that specified. It is computed by determining the area under the curve that fails to comply with the limit value (in some cases, minimum value). Figure 3.12 illustrates the concept of P-value for OPI values. The value of 9.75 is the average OPI obtained from the Project. From the figure below, the P-value of 0.00689 can be interpreted as the probability of obtaining values less than 9.7 to be 0.689%. A smaller P-value illustrates that there is greater the evidence against a null hypothesis (H_0).

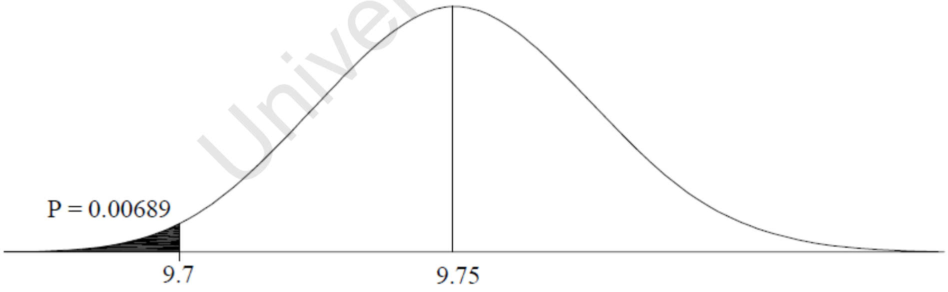


Figure 3.12: Schematic illustration of P-value used in hypothesis tests.

The null hypothesis is rejected when $P \leq \alpha$. The data is ‘statistically significant’ when the null hypothesis is rejected. Statistical significance can be interpreted as an event that is highly unlikely to happen by chance e.g. Moroney (1965) defines a highly significant event as one that has a 1 in 1,000 chance of happening. Careful interpretation of a hypothesis test should be done as an event that is ‘statistically significant’ may not be of practical (‘engineering’) significance e.g. from a hypothesis test a difference of one in a measurement may be statistically significant but in practical terms would have little or no effect on the aspect considered.

The application of hypothesis tests in data analysis is further described below.

3.3.4.1 Comparison of mean value in Projects with limit value

A hypothesis test was carried out to determine if the mean values (of OPI, sorptivity and cover depth) in projects considered for analysis differed significantly from the specified value. The aim of the hypothesis test was to determine if the data is significant at the given level of significance. The significance level selected for use was 10% which is higher than that used for strength (5%) as durability is related to serviceability of concrete structures, unlike strength which is an ultimate limit state criterion (Stanish and Alexander, 2005). Additional description of the process used in this hypothesis test is provided in the attached CD.

3.3.4.2 Application of a classification system

Further hypothesis tests were carried out to determine the effect of different variables on the mean values obtained. The classification system used was based on consideration of:-

- Projects: a total of five considered for DI test values.
- Cover depth readings: four locations were considered.
- Concrete grade
- Source of sample – test panel/ cube
- Type of structure - in situ/ precast elements

(a) Comparison of two independent samples

The difference in mean values of samples obtained from two independent sources is considered; hypothesis test used was the *t*-test (further description provided in attached CD). The hypothesis test is based on determining the difference in means $\mu_1 - \mu_2$ as stated in Equation 3.8, where Δ_o is taken to be zero.

$$H_o: \mu_1 - \mu_2 = \Delta_o$$
$$H_1: \mu_1 - \mu_2 \neq \Delta_o \qquad (3.8)$$

The *t*-test was used, for example to evaluate the difference in OPI values on considering source of sample, test panels/cube and precast/in situ elements. The hypothesis for these tests are given in Equation 3.9 and 3.10.

$$H_o: \mu_{\text{Test panel}} = \mu_{\text{cube}}$$
$$H_1: \mu_{\text{Test panel}} \neq \mu_{\text{cube}} \qquad (3.9)$$

$$H_o: \mu_{\text{Precast}} = \mu_{\text{in situ}}$$
$$H_1: \mu_{\text{Precast}} \neq \mu_{\text{in situ}} \qquad (3.10)$$

(b) Comparison of more than two samples (ANOVA)

The single factor analysis of variance was applied to compare mean values of DI test values of different projects (five considered) and for cover depth readings bridge structures in different locations (four considered) to determine if there is a significant difference. The hypothesis tests on which this test is based on is given in Equation 3.15.

$$H_o: \mu_1 = \mu_2 = \mu_4 = \mu_6 = \mu_9$$

$$H_1: \mu_1 \neq \mu_2 \neq \mu_4 \neq \mu_6 \neq \mu_9 \qquad (3.11)$$

Where, 1,2,4,6 and 9 are the five projects considered.

Further description of the ANOVA approach on which repeatability and reproducibility of a test is based on is provided in Appendix C. For this study, repeatability and reproducibility were not determined. This is because one of the requirements for carrying out an inter-laboratory exercise is for material used to be ‘as nearly identical as possible’ (ASTM C 803; ISO 5725:2, 1994); the values obtained and used in data analysis were however obtained from different sites where in most cases the material properties were unknown.

3.3.4.3 The goodness-of-fit test

To confirm the distribution model of data in a sample, in addition to fitting a distribution curve onto the histogram as illustrated in Figure 3.12, the goodness-of-fit test based on the chi-square distribution was done. The aim of carrying out this test is to determine if the distribution of data is normal, log-normal or gamma. The distribution model influences the statistical methods used in analysis where for non-normal data, different statistical methods are applied. The goodness-of-fit test was carried out using an Excel program developed by Muigai (2008); worked examples are provided in the attached CD. From the tests, it was observed that not all data had a normal distribution. In spite of the different distribution models, the descriptive statistical methods described in the preceding section of determining mean values and variability were applied for all the data considered.

3.4 SUMMARY

The Chapter provides a review of the methodology applied to evaluate the objectives of the research; the main aspect considered was the data collection process. The data obtained was mainly from the GFIP, a SANRAL project that aims at improvement of the road network in Gauteng Province. This data includes: durability index values (OPI and sorptivity results), cover depth readings, and for some projects, strength values. In addition, a review was made on: report of an audit exercise on laboratories that carry out the DI tests; response to questionnaires sent out to REs in GFIP projects; the concrete mix properties from the Ready Mix Concrete supplier and precast units manufacturer. The statistical methods applied for data analysis are also described. Descriptive statistics which involves determination of measure of central tendency (mean values) and measure of variability are outlined. The application of inferential statistics through hypothesis testing is also reviewed and its use in determining if significant differences are present on application of a classification system that considers concrete grade, type of structure (precast/insitu) and source of sample (test panel/cube). Further statistical analysis of the quantitative data is provided in Chapter Four.

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Chapter Four

Data analysis

4.1 INTRODUCTION

The Durability Index-based (DI) performance approach provides for the control of concrete quality, with regard to durability, through measures of concrete penetrability (obtained through OPI and sorptivity values) and cover thickness (determined from cover surveys). This chapter provides a statistical analysis of test results obtained from DI tests, strength measurements and cover depth surveys carried out on a mega-project (the GFIP) to evaluate extent or magnitude of variability in test values, which influences practicality of application of this approach in control of concrete quality.

4.1.1 Control of quality in concrete structures

Current approach

Quality control is defined as those methods that are used to measure, control, monitor and improve the quality of a product (Montgomery and Runger, 2007). The control of concrete quality, for both durability and strength requirements, provided in current standards e.g. in BS EN 206-1 (2000), ACI 318M (2008) is presently based on a measure of strength using either cubes or cylindrical specimen.

The quality of concrete structures as measured through strength varies. Sources of variability may be due to: materials used in production of concrete; quality of workmanship and execution of construction practices on site; and quality of testing in laboratories which is influenced by apparatus used and skill of operators in execution of tests (Popovics, 1982; ASTM E177-06b). A measure of the extent of variability and control of quality are important during construction as it provides information on the uniformity of construction (Bungey and Millard, 1996).

The application of statistical methods in quality control of concrete was suggested in 1955 by Walker who provided four categories of standard of control based on a measure of coefficient of variation for strength measurements; these values are provided in Table 4.1.

Table 4.1: Classification of different standards of control based on coefficient of variation (Walker, 1955)

Standard of control	Coefficient of variation (%)
Near laboratory precision	10
Excellent	12
Good	15
Fair minus	20

Further studies on the application of statistical methods in control of concrete quality have been undertaken e.g. Soroka (1971) provided three categories of control for strength that were established for local conditions on the basis of extensive field studies. The maximum values, based on coefficient of variation, were 12%, 20% and 30% for “good”, “fair” and “poor” ratings respectively.

DI-based Performance approach

The DI-based performance approach requires the verification of cover penetrability (using DI tests) and cover depth, in addition to strength measurements.

DI tests have been widely applied in laboratory-based studies to characterize concrete and determine the influence of different concrete grades, curing methods and binder types (Ballim, 1993; Mackechnie, 1996; Scott, 2004), in addition to an interlaboratory exercise reported in Stanish et al. (2005) carried out to determine precision of the tests. Some site-based studies have also been undertaken, for example a pilot study on application of these tests on site by Bouwer (1998) where it was observed that valid test results, comparable to those obtained from laboratory prepared samples, can be attained on site.

The DI-performance based approach has been implemented in project specifications by SANRAL in the GFIP. The main objective of this Chapter in evaluating practicality of application of the DI-based performance approach on a large scale was to carry out statistical analysis of data (number of test results summarized in Chapter 3, Table 3.2 and 3.8, page 86 and 101) in order to determine:-

(i) Average values obtained from different projects, and if these values comply with the limiting values that are provided in project specifications.

(ii) The extent or magnitude of variation in test values which indicates the degree of control in application of the tests on site.

(iii) Proportion of values that fail to conform to limiting values.

In addition to the above statistical quantification of distribution and variability of data, a classification system was applied to evaluate the effect of the different variables on test values.

(a) For DI test values the basis of classification was:-

- Projects (five considered)
- Source of samples (test panel/cube)
- Type of structure (precast/in-situ)
- Concrete grade

(b) For cover depth readings classification was made on the basis of:

- Type of structure (precast or in situ)
- Four bridge structures in Project 2 located in different locations

An analysis of strength measurements was also made to determine average values and amount of variability. Comparison of strength values with DI test values was also done to determine if the two parameters have a relationship.

4.2 DURABILITY INDEX TEST VALUES

The analysis of data in the subsequent sections is with regard to OPI (and k-values), sorptivity (and porosity) values from five projects. A brief description of the projects, with regard to source of sample (cube, test panel or actual structure), type of structure (precast or in situ) and concrete grades, is presented in Table 4.2.

Table 4.2: Description of data used for analysis

Project ID	Description		
	Source of sample	Type of structures	Concrete grade (MPa)
1	Cubes, test panels	In situ	25, 30, 40, 50
2	Cubes	In situ	Not provided
4	Cubes, test panels	In situ	30, 40
6	Cubes, test panels	In situ	30, 40, 60
9	Structure	Precast	30

Data relating to in-situ structures was obtained from: bridges e.g. piers, abutments, wing walls; retaining walls; culverts; toll gantries. Data from Project 9 was obtained from precast median barriers used on highways. These barriers are either single sided or double sided; an illustration of sections of the two types of barriers is provided in Figure B1, Appendix B.

4.2.1 DI test values of concrete mixes from RMC

The concrete supplied to some of the projects in the GFIP was obtained from four RMC plants in Gauteng. Concrete mixes were tested to determine their durability properties; these values represent the ‘material potential’ of the concrete (Alexander et al., 2006). The testing was done on samples obtained from 100 mm wet cured cubes, and testing conducted in accordance with the Durability Index Test Manual (2009). Table 4.3 presents a summary of these values; additional details on DI values of concrete mixes are provided in Table B3, Appendix B.

Table 4.3: Summary of range of DI values of concrete mixes from RMC

OPI log scale	Sorptivity mm/√hr	Porosity %
9.86 - 10.62	6.70 - 8.50	7.30 - 12.70

The limit values provided in the specifications are: for OPI, values should be more than or equal to 9.70 (on a log scale) while for sorptivity, values should be less than or equal to 10 mm/√hr. From the values in Table 4.3, all concrete mixes comply with these limit values indicating that concrete mix has the ‘potential’ to meet durability requirements.

4.2.2 Oxygen permeability Index (OPI)

The section below provides a statistical analysis of OPI values from the five projects considered.

4.2.2.1 Descriptive statistics

The initial step in analysis involved the determination of the distribution model of the data. This was done by plotting of histograms and fitting a suitable distribution curve using the *dfittool* in MATLAB, in addition to the goodness-of-fit tests. Histograms for the projects considered are illustrated in Figure 4.1; these histograms contain all data from a project without classification i.e. with regard to concrete grade or source of sample (cube/test panel). The histograms were all plotted on the same horizontal axis, with values ranging from 8.8 to 11.0 (log scale), for the purpose of comparing the amount of spread (variability) among the projects. The limit value of 9.7 is indicated by the dotted line in the figures.

From the histogram plots in Figure 4.1 the following observations are made:-

i. The amount of spread in values varies among the projects; it is lowest for Project 9 and highest for Project 6 where values range from 8.8 to 11.2.
ii. The proportion of values that fail to comply with the limit value (as indicated by area bonded under the curve and to the left of the dotted line) is highest in Project 1 while in Project 9, all values exceed this limit value.

Numerical summaries of the data were then computed by determining mean values and measures of variability (standard deviation (s) and coefficient of variation (CoV)). These values for the five projects considered are presented in Table 4.4.

Table 4.4: Summary statistics of OPI values

		OPI (log scale)				
Project ID	n	Mean	Max	Min	s	CoV (%)
1	172	9.75	10.41	9.07	0.28	2.84
2	94	9.91	10.42	9.37	0.22	2.24
4	116	9.87	10.40	9.39	0.23	2.33
6	91	10.06	11.10	8.83	0.46	4.60
9	132	10.25	10.70	9.85	0.18	1.75

The observations made from the computed numerical summaries in the table above are:-

i. The mean values for all the projects considered exceeded the limit value provided in project specifications. The highest mean value was obtained from precast elements in Project 9 while for in situ elements, the highest mean value was from Project 6.
ii. The highest maximum value obtained was 11.10 which was much greater than limit value of 9.7 (considering that these values are on a log scale). The lowest minimum value of 8.83 was also obtained from the same project, which indicates a high variability in values measured from this project.

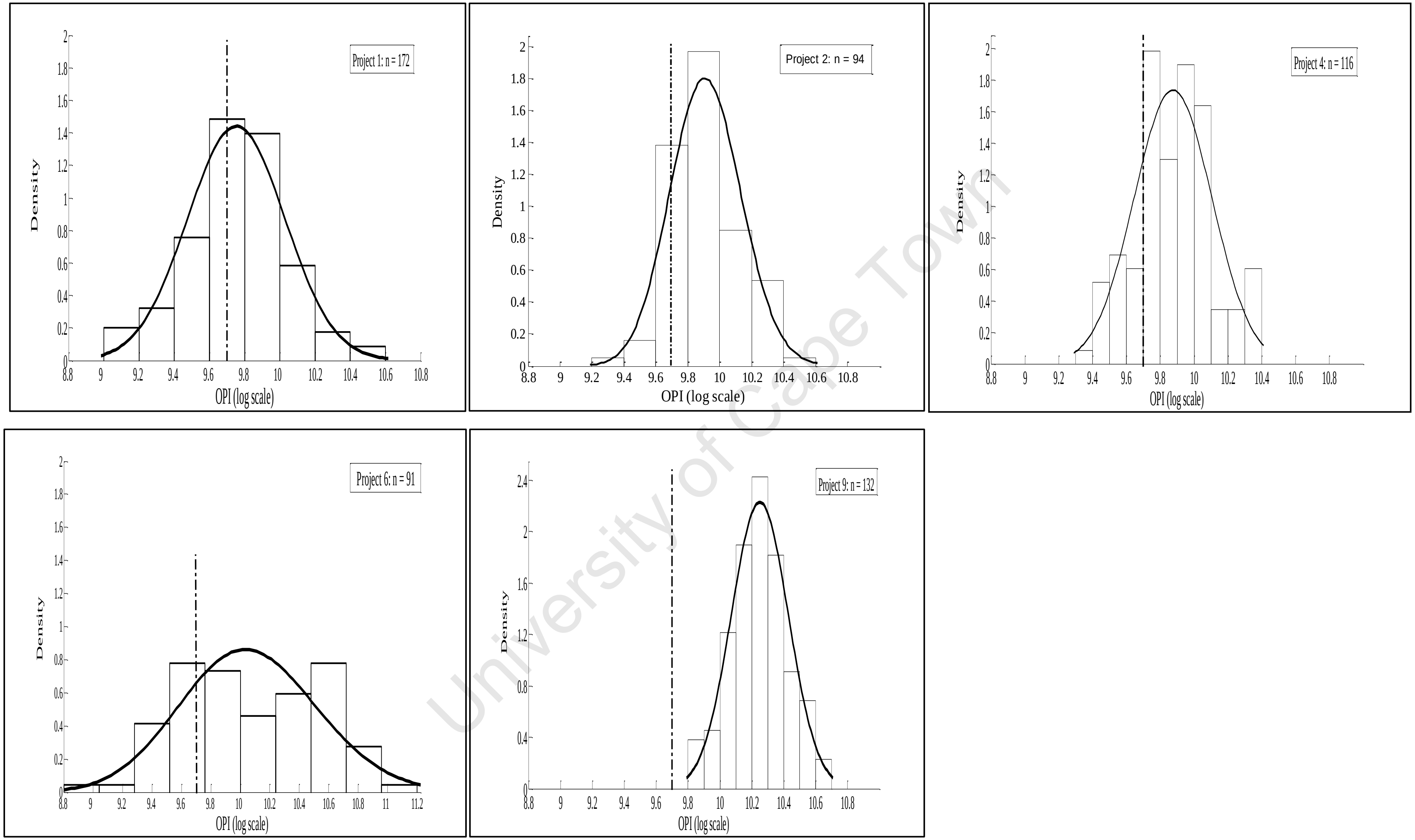

Figure 4.1: Histogram plots of OPI values from five projects of GFIP.

iii. The variability of values was within the same range (1.75 – 2.84%) with an exceptionally high value of 4.60% for Project 6. From the interlaboratory exercise reported in Stanish et al. (2006), the reproducibility of the OPI test was determined to be 1.8%. When compared to this value, the variability of OPI values obtained from site elements is low. A higher measured value of variability would be expected as these samples were obtained from site elements where the degree of control in construction is lower in comparison to that of laboratory prepared samples.

The number (n) and percentage of values within each project that failed to comply with limiting values is provided in Table 4.5; the values considered are those that are less than 9.70 but greater than 8.75, which would result in a reduction in payment of 80% as provided in Table 3.1 (page 83).

Table 4.5: Proportion of values that fail to comply with the limiting OPI values

	OPI (log scale)	
Project ID	n	(%)
1	69	40.1
2	13	13.8
4	21	18.1
6	24	26.4
9	0	0

The proportion of values that fail to comply with the limit value of 9.70 but were greater than 8.75 was high with as much as 40.1% from Project 1. From analysis of data, no values were found to be below 8.75 for all projects considered. The proportion of values in the other projects is also high with the exception of Project 9 where all values complied with the limit value. In as much as the mean value is attained for all projects, the proportion of values that fail to comply with this value is high and needs to be considered in making payments i.e. a high proportion of non-compliance as observed in Project 1 would require a reduction in payments made to contractor by 80%. The essential question to ask in implementation of these specifications is, are such reductions in payments enforced?

4.2.2.2 Data analysis on k-values

The OPI value, a test result is determined by obtaining the negative logarithm of the average of coefficient of permeability (k) values (four test determinations). This section will consider an analysis of k-values. The limit k-value, based on obtaining the antilog of limit value of 9.7 from project specifications is 2.00 E-10 m/s. Histogram plots of the k-values are illustrated in Figure 4.2.

The distribution of values as illustrated by the histograms in the Figure is skewed to the left, indicating a log-normal distribution. This indicates that majority of the values comply with the limiting value of 2.00E-10 m/s, with the exception of Project 1. The amount of spread in values, as indicated by range of value on the horizontal axis, is highest for Project 1 and 6, and lowest for Project 9; this is similar to what is observed for OPI values, Figure 4.1.

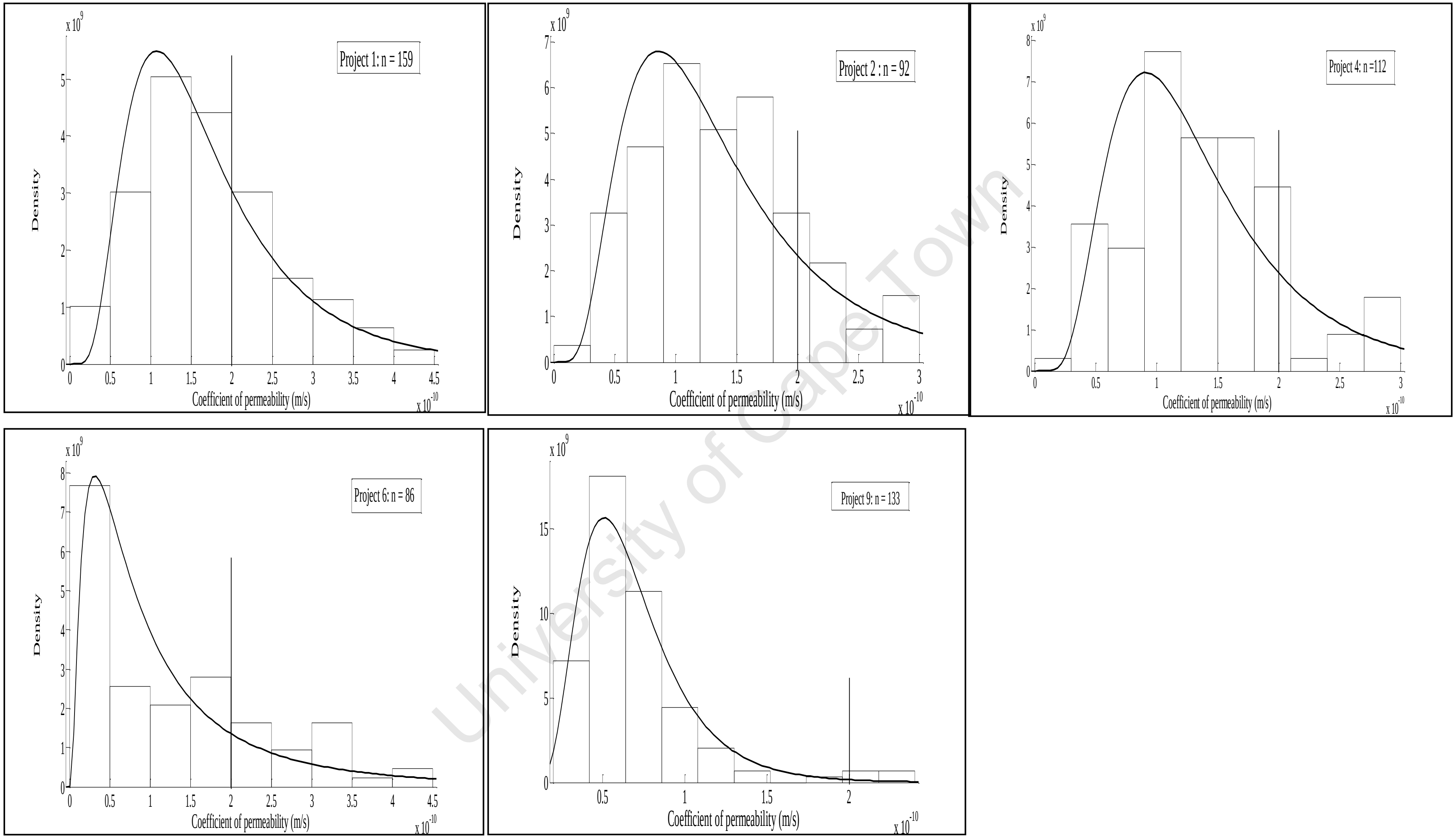

Figure 4.2: Histogram plots for k-values.

The numerical summaries computed for k-values are presented in Table 4.6.

Table 4.6: Summary statistics of k-values

Project ID	n	Coefficient of permeability (E-10 m/s) Mean	Max	Min	s	CoV (%)
1	159	1.72	4.10	0.267	0.879	51
2	92	1.37	2.89	0.062	0.628	46
4	112	1.36	2.97	0.252	0.625	46
6	86	1.27	4.01	0.085	1.08	85
9	133	0.695	2.27	0.227	0.369	53

From the computed numerical summaries in the table above, the following observations were made:-

i. All mean values in the different projects were below the limit value of 2.00E-10m/s. A lower value of permeability indicates low permeability of concrete; the samples from Project 9 therefore had the lowest permeability on average.

ii. The maximum values for Project 1 and 6 were approximately double that of the limit value which indicates test samples with high permeability properties. Low permeability values were also observed, with minimum values as low as 0.062E-10 m/s.

iii. The variability of k-values in the Projects was within the same range of 46 – 51%, with the exception of Project 6 that had the highest variability of 85%. From the interlaboratory exercise reported in Stanish et al. (2006), the CoV was found to range from 23.7 to 53.9%. The variability of k-values obtained from the projects, when compared to those obtained from the interlaboratory exercise, are therefore within an acceptable range with the exception of Project 6 where the variability is high.

There is a difference in variability of the k-value and OPI. The k-value represents the actual value of material permeability while the OPI is based on a transformed value which is much less in magnitude.

The proportion of values greater than the limit value of 2.00E-10 m/s was also determined and is summarized in the table below.

Table 4.7: Proportion of values that fail to comply with limit k-value

Project ID	k-value (m/s) n	%
1	52	32.7
2	16	17.4
4	13	11.6
6	21	24.4
9	4	3

The highest proportion of values that fail to comply with the limit value is from Project 1 with 32.7%; a similar observation was made with OPI values for this project. However, the proportion of values higher than the limit value is less than that observed for OPI values. In addition, Project 9 has 3% of non-complying values in comparison to that observed for OPI where all the values complied.

4.2.2.3 Inferential statistics

The average value obtained from the five projects considered is greater than the limit value of 9.70. A hypothesis test to determine if the average OPI values in the projects is more than that specified, as indicated in Equation 4.1, at a 10 % level of significance is therefore not necessary.

$$H_o: \mu \leq 9.70$$
$$H_1: \mu > 9.70 \tag{4.1}$$

The use of inferential statistics was applied in analysis based on classification of data and is given in the following sections.

C***lassification of data***

The data used was obtained from different projects (a total of five considered), concrete grade, source of sample (precast or in-situ) and cube/test panel; a brief description of data is given in Table 4.2. To determine the influence of the variables on OPI values, hypothesis testing was carried out and is outlined in sections below.

(a) Comparison of projects

The OPI values from the five projects considered were compared and analyzed using Analysis of Variance (ANOVA). A random sample of 50 test results was obtained from the five projects considered. An equal sample size of 50 test results was used to enable comparison of data and for the ANOVA computation. A comparative box plot was made with these data, as illustrated in Figure 4.3. These values were obtained from different concrete grades and source (test panel or cube). The summary of values used and computations are presented in Excel spreadsheets, in the attached CD.

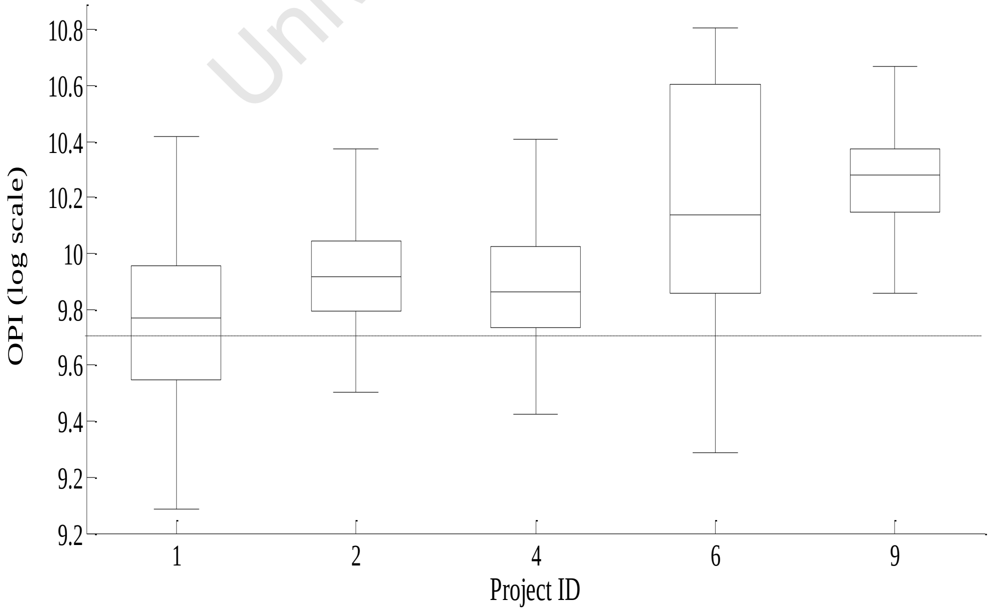

Figure 4.3: Comparative box plots of OPI values in the GFIP projects.

From Figure 4.3 it is observed that variability in Project 6 is the highest (this is also indicated by numerical summaries and histograms) while that of Project 9 has the lowest variability. OPI values from Project 9 are higher than those from other projects. The average value from Project 9 which represents precast elements is also higher than that for the other projects where data was obtained from in situ elements.

The ANOVA test is based on the hypothesis given in Equation 4.2 to determine if there is a difference in mean values.

$$H_o: \mu_1 = \mu_2 = \mu_4 = \mu_6 = \mu_9$$

$$H_1: \mu_1 \neq \mu_2 \neq \mu_4 \neq \mu_6 \neq \mu_9 \tag{4.2}$$

P-value obtained from this test was 1.90E-17 which is lower than α of 0.1; therefore the null hypothesis is rejected and it is concluded that mean OPI values in all the projects considered differ. The difference in mean may be attributed to the different concrete grades considered, which may have an influence on the OPI values obtained.

From Figure 4.3, the mean values as illustrated by the box plots are similar for Projects 2 and 4. A t-test was therefore done to compare the mean values from the two projects and it was observed that the values do not differ significantly at a 10% level of significance. This indicates that for in situ elements, similar average values can be obtained from different projects i.e. if similar controls in quality and construction practices are exercised for different projects, the variability in values can be reduced such that mean values do not differ significantly.

(b) Comparison of concrete grades

To explore the effect of concrete grade on OPI values obtained an analysis of the effect of concrete grade on OPI values was carried out on data from Project 1, 4 and 6. This analysis is provided in the data analysis spreadsheet contained in the attached CD. For Project 1 and 4, it was observed that mean values of OPI do not differ at a 10% level of significance for different concrete grades. However, for Project 6 the mean OPI values from three concrete grades considered was observed to differ. Comparative box plots of this project are presented in Figure 4.4. The observations made from the figure are that with a higher concrete grade, higher values of OPI are obtained. For the Grade 30 concrete there is a high proportion of values that fail to comply with the limit value of 9.7, as indicated by values in box plot below the dotted line.

From the box plots in Figure 4.4, it is observed that with a higher grade of concrete the OPI value is higher. Thus, the high variability determined for Project 6 of 4.60% (as presented in Table 4.4, page 117) may be due to different grades of concrete considered.

A hypothesis test based on ANOVA was done to determine if the mean values from the three concrete grades differs significantly. The hypothesis used for this test is given in Equation 4.3.

$$H_0: \mu_{W30} = \mu_{W40} = \mu_{W60}$$

$$H_1: \mu_{W30} \neq \mu_{W40} \neq \mu_{W60} \tag{4.3}$$

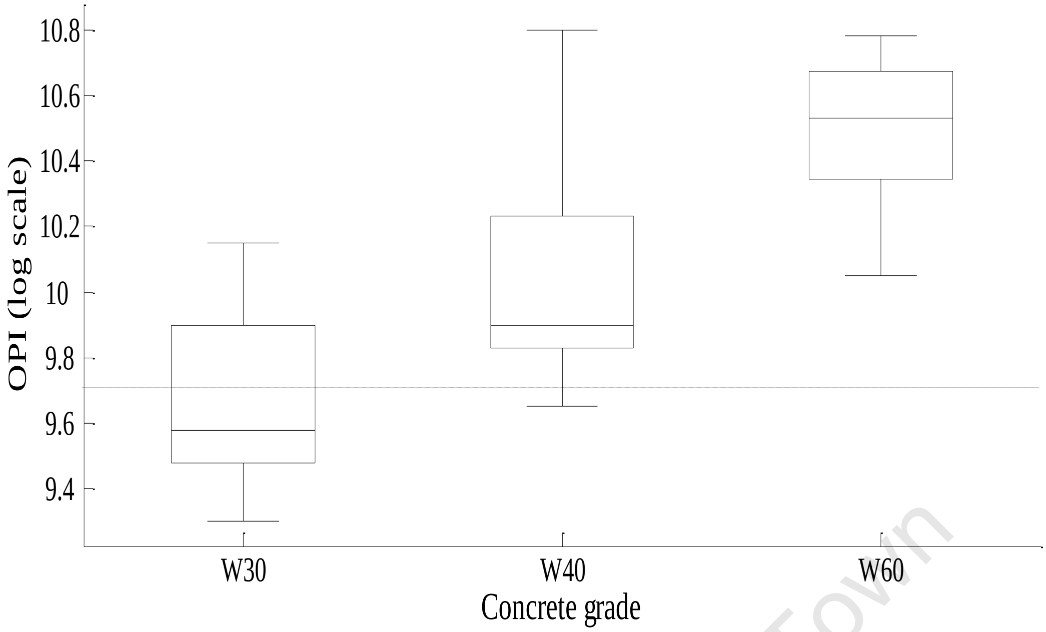

Figure 4.4: Comparative box plots for three different concrete grades (Project 6).

From this test, the P-value obtained is 1.37 E-09 which is less than α of 0.1. The H_o was therefore rejected and it was concluded that the mean OPI values for the three concrete grades differ. It is concluded that the concrete grade may have an effect on the average OPI value obtained.

Based on this effect of concrete grade on OPI values, a second analysis of variance of OPI values was done; only elements of Grade 30 concrete were considered in this case. A random sample of 15 values was used in the analysis; Figure 4.5 illustrates the comparative box plot for these values.

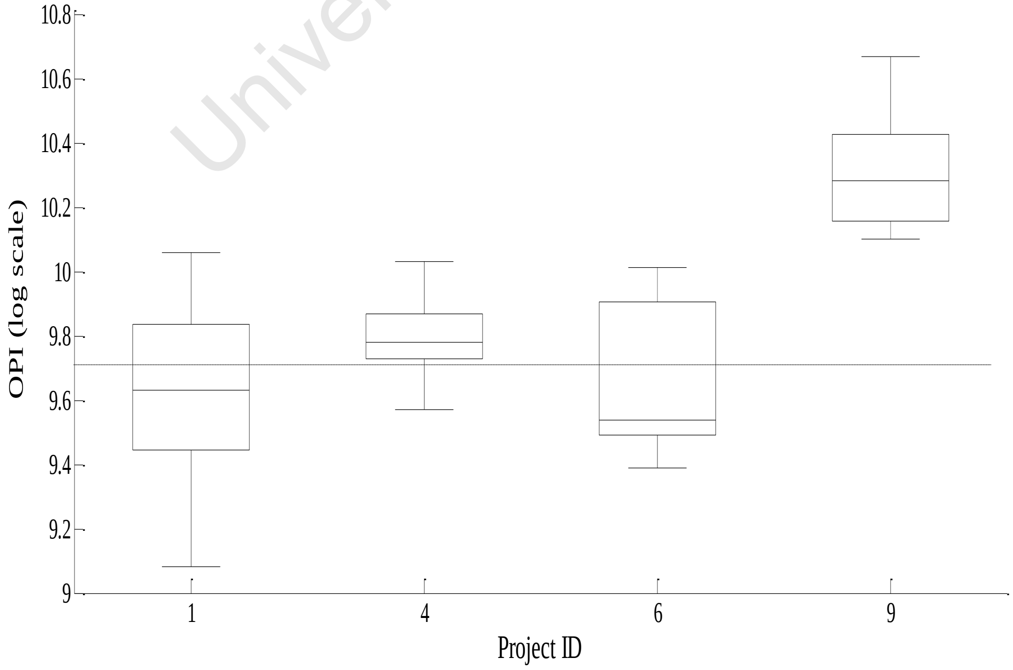

Figure 4.5: Comparative box plot for work packages (Grade 30 concrete)

From the figure above, the variability of values in Projects 1 and 6 is high for Grade 30 concrete; Project 4 has the lowest variability in OPI values. From the analysis of variance, the P-value obtained was 2.12E-12 which is lower than level of significance of 0.1. Therefore, the mean OPI value differs significantly in the projects even when the same grade of concrete is considered. This observation illustrates that the variation of OPI values is attributed to more than strength values. The difference in values may be attributed to the construction practices exercised and the degree of control in their execution.

(c) Comparison of Type of structure (in situ and precast elements)

The source of OPI values from precast and in situ structures was considered. The random sample used for analysis was obtained from Project 1, 4 and 6 for in situ elements and Project 9. All values considered were for Grade 30 concrete. A comparative box plot of these values is presented in Figure 4.6. The values from in situ elements have a higher variability, as indicated by the larger spread, in comparison to precast elements. The mean values for precast elements are also much higher than those of in situ elements.

The t-test was applied to determine if the difference in mean values from the two sources is statistically significant; hypothesis used for this test is given in Equation 4.4.

$$H_0: \mu_{precast} = \mu_{in\ situ}$$
$$H_1: \mu_{precast} \neq \mu_{in\ situ} \qquad (4.4)$$

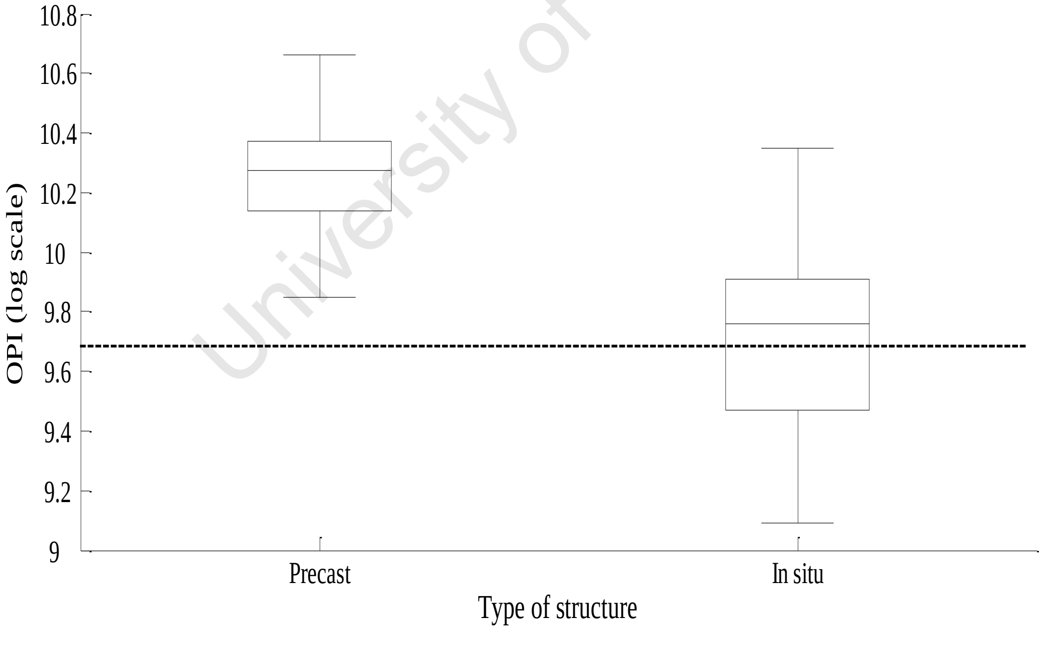

Figure 4.6: Comparative box plot for precast elements (Project 9) and in situ elements (Project 1, 4 and 6).

The P-value from this test was 2.28E-18, which is lower than α of 0.1. Therefore, the null hypothesis (H_0) was rejected and it was concluded that mean values from precast and in situ elements differ significantly at level of significance of 10%. This indicates that a higher degree of control is exercised in the production of precast elements which leads to the higher OPI values, in comparison to that of in situ elements.

(d) Comparison of Source of sample (test panel and cube)

From the earlier provisions in specifications, samples used for carrying out DI tests could be obtained from either cubes (water cured) or test panels. A comparison of the two sources of samples obtained from Project 4 is presented in Figure 4.7. It is observed that variability of values obtained from cubes is higher than that from test panels. This may indicate that there was less control in the preparation and use of cubes to determine durability properties, in comparison to that exercised for panels for this project.

A t-test to evaluate if the mean values from the two sources differ significantly was done; hypothesis used for this test is given in Equation 4.5.

$$H_0: \mu_{cube} = \mu_{panel}$$
$$H_1: \mu_{cube} \neq \mu_{panel} \qquad (4.5)$$

The P-value from this test is 0.24 which is higher than α of 0.1. The null hypothesis (H_0) is therefore not rejected and its concluded that mean values from the two sources do not differ. This conclusion is however contrary to that determined from a study undertaken and reported in Ronny and Everitt (2010). As there was limited data from the GFIP projects that classified data on basis of test panels or cubes, further statistical analysis could not be done to further explore and validate this aspect. It is therefore concluded that cubes do not always give higher values of test results in comparison to test panels, and the values obtained are dependent on the extent of control undertaken in a project on preparation of these samples.

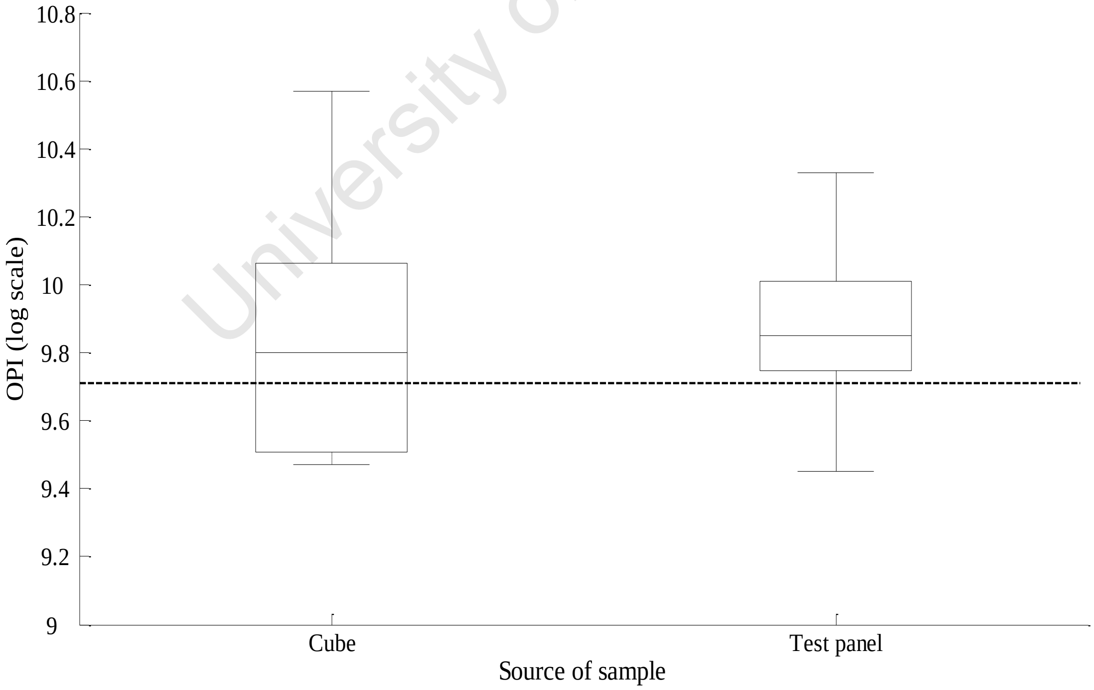

Figure 4.7: Comparative box plots for source of sample (Project 4).

4.2.2.4 Discussion

The preceding sections provide an analysis of OPI values obtained from five GFIP projects. The aspects observed from this analysis are discussed below.

(i) *Extent of variability in values*: The average OPI value for all projects complied with the limit value of 9.70. The variability, as computed using CoV was also observed to be acceptable when compared to past studies undertaken (Stanish et al., 2005; Bouwer, 1998). However, in as much as these values comply with the specified value and the observed variability is acceptable, a high proportion of values fails to comply; as much as 40.1% for Project 1. From the project specifications (SANRAL, 2010) the payment criteria provides for 100% payment when values are greater than 9.70 and a reduction in payment of 80% if this is not met; additional details are provided in Table B8212/1 contained in extract of specifications. The specifications do not however provide for a minimum percentage of OPI values for elements considered for a structure that are permitted to fail to comply with limiting values, which is an aspect that should be considered.

(ii) *Coefficient of permeability (k-value) analysis*: The average values of k were all observed to be below the limit value of 2.00E-10m/s. The variability of these values was also similar to that obtained from an interlaboratory exercise reported in Stanish et al. (2005). In RILEM TC 116-PCD (1998), a comparative study was undertaken on permeability tests. It was observed that the variability of permeability was found to be in the range of 30% for concrete samples tested in laboratories. The high variability in k-values was attributed to high sensitivity of the tests to differences in material properties. The variability of k-values for this study (with the exception of that from Project 6 of 85%) was therefore found to be within an acceptable range in comparison to that obtained and reported in Stanish et al. (2005).

(iii) *Classification of data*: The OPI values from five projects considered (without classification of values in concrete grades) were compared using the ANOVA and it was observed that mean values differed. The extent of variability in some projects e.g. Project 6 was high which indicates the level of control was lower when compared to Project 9 where variability in OPI values was less. This comparison of projects was done without considering the effects of different concrete grades or source of sample (test panel or cube).

The OPI values were then classified by considering concrete grades; from this comparison it was observed that with higher concrete grades, a higher value of OPI can be obtained. For two out of the three projects considered, the mean values from different concrete grade did not differ which indicates that though a higher concrete grade may result in a higher OPI value, it does not have a significant effect on mean OPI values.

From the comparison of type of structure, precast or in situ elements, it was observed that the mean values between these two elements differ significantly. The extent of variability for precast elements was lower than that of in situ elements which indicates that a higher degree of control is exercised in construction of these elements. For the comparison of test panels and cubes, it was observed that the mean values of OPI from the two sources do not differ significantly. However in as much as no significant difference was observed between the two, it is preferable to use test panels as they are more representative of the actual structure in comparison to water cured cube samples.

4.2.3 Water Sorptivity Index (WSI)

Statistical analysis of sorptivity values (and porosity) are provided in the section below. Histogram plots of sorptivity values from the five projects considered are presented in Figure 4.8. For the purpose of comparison, the histograms plotted have the same range of values on the horizontal axis of 5 to 15 mm/hr$^{0.5}$. A dotted line is plotted through the limiting value of 10 mm/hr$^{0.5}$. The observations made from histogram plots in Figure 4.8 are:-

i The distribution of data as indicated by the plots varies, with that from Project 4 and 6 having a positive skew. The spread of values is highest for Project 9 with values ranging from 5 to 15 mm/hr$^{0.5}$.

ii The proportion of values that fails to comply with limit values, indicated by values to the right of the dotted line, is low for Projects 4 and 6 and higher for Projects 1, 2 and 9.

Numerical summaries were determined by computing mean values and measures of spread (standard deviation (s) and coefficient of variation (CoV) are presented in Table 4.8.

Table 4.8: Summary statistics of sorptivity values

		Sorptivity (mm/√hr)				
Project ID	n	Mean	Max	Min	s	CoV (%)
1	154	9.67	13.73	6.88	1.49	15.4
2	74	9.36	13.21	6.04	1.76	18.8
4	110	6.29	11.37	3.48	1.92	30. 6
5	88	6.97	10.35	4.28	1.50	21.5
9	136	7.80	14.43	1.00	3.8	49.0

The observations made from computation of the numerical summaries given in the table above are:-

i. The average values for all the projects were below the limit value of 10 mm/hr$^{0.5}$.

ii. The variability of values was high, with as much as 49.0% for Project 9. The variability is described as high when compared to CoV values from an interlaboratory exercise which ranged from 10 – 26% (Stanish et al., 2006).

The data for Project 4, 6 and 9 indicated extremely low sorptivity values, as low as 1.0 mm/hr$^{0.5}$ for Project 9. From laboratory based studies, the lowest observed values for sorptivity were in the range of 5 – 6 mm/hr$^{0.5}$ (e.g. in Mackechnie, 1996). Thus, the extremely low values less than 5 mm/hr$^{0.5}$ in Project 4, 6 and 9 were eliminated. This low values are an indicator that proper testing was not carried out in the laboratories.

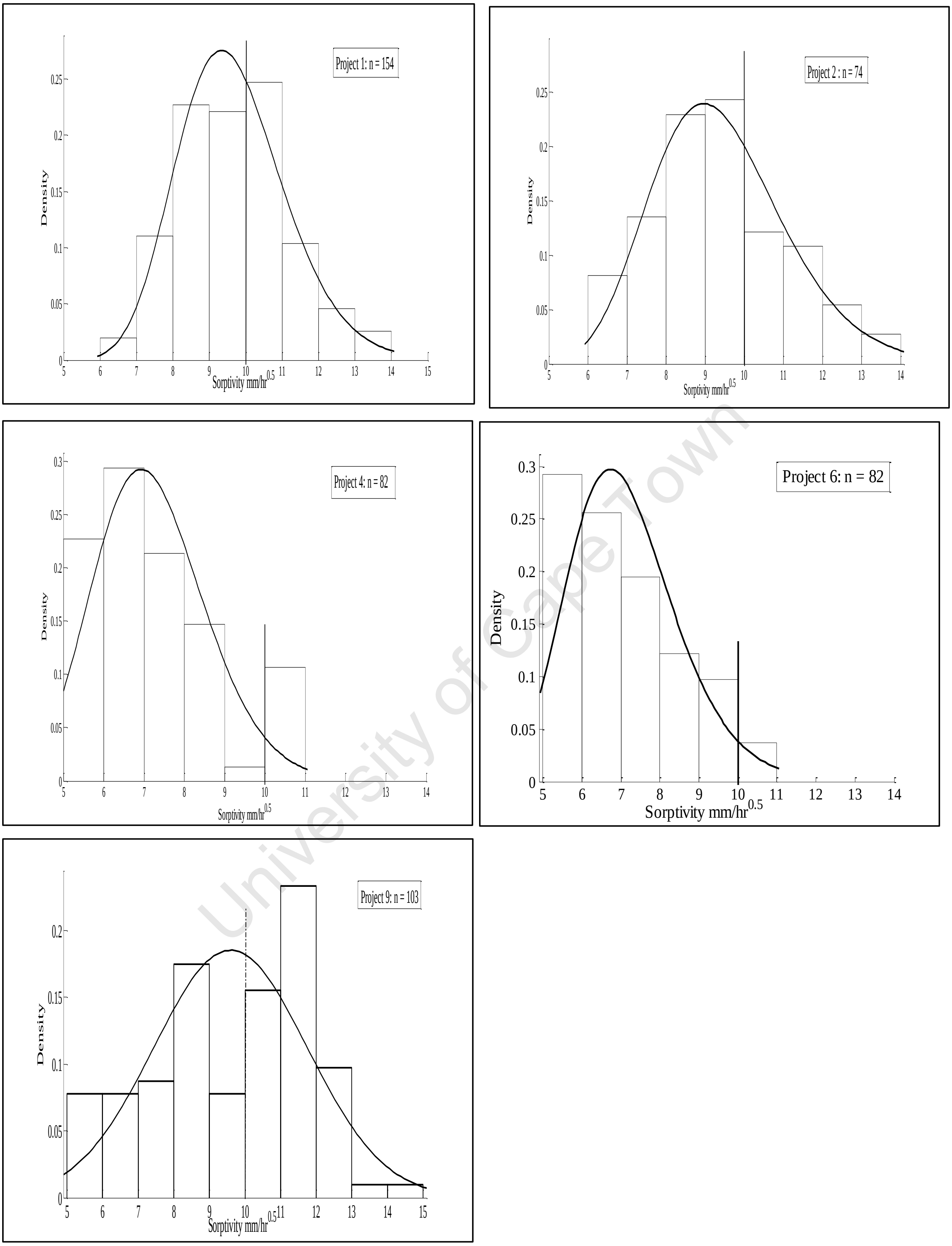

Figure 4.8: Histograms of sorptivity values from five projects considered.

The re-computed numerical summaries for sorptivity values of these projects are presented in Table 4.9.

Table 4.9: Summary statistics of sorptivity values after elimination of extreme low values

		Sorptivity (mm/hr$^{0.5}$)				
Project ID	n	Mean	Max	Min	s	CoV (%)
4	75	7.20	10.33	5.15	1.4	19.9
6	82	7.13	10.35	5.20	1.4	20.0
9	103	9.59	14.43	5.08	2.2	22.5

The effect of elimination of these values is an increase in mean sorptivity values and a reduction in variability e.g. for Project 9 an increase from 7.8 mm/hr$^{0.5}$ to 9.56 mm/hr$^{0.5}$ while variability reduced from 49.0% to 22.5%. From re-computed values, the variability obtained is comparable to that reported in Stanish et al. (2005) of 10 – 26%.

The proportion of values that fail to comply with limiting values is presented in Table 4.10. For Project 9, this proportion was high with up to 50% of values being higher than the limit value. The proportion of values failing to comply with limit values in Project 1 and 2 was also high.

Table 4.10: Proportion of values that fail to comply with limiting values

Project ID	Proportion	
	n	%
1	65	42.2
2	23	30.7
4	8	10.7
6	3	3.7
9	52	50.5

In addition to the sorptivity values, numerical summaries for porosity values for Project 1 and 2 were also determined, these are presented in Table 4.11; for the other three Projects the spreadsheets did not provide a record of the porosity values.

Table 4.11: Numerical summaries of porosity values

Project ID	n	Mean	Max	Min	s	CoV
1	160	13.0	17.1	8.7	1.7	12.7
2	83	11.5	15.5	8.1	1.6	13.7

Porosity provides a measure of the volume of pores within a concrete sample that can be occupied with water. It is a ‘bulk property’ while sorptivity measures the rate of absorption of water as a ‘near surface property’. The average porosity values and variability obtained for the two projects are within an acceptable range when compared to those obtained from an interlaboratory exercise which ranged from 10.2 – 13.5% for porosity and 5.7 – 15.4% for the CoV (Stanish et al., 2006).

(b) Inferential statistics

The average sorptivity values from the projects considered are all lower than the limit value of 10 mm/√hr. A hypothesis test, given in Equation 4.6, to determine if the data is statistically significant at a 10% level of significance is therefore not required.

$$H_o: \mu \geq 10 \text{ mm/hr}^{0.5}$$
$$H_1: \mu < 10 \text{ mm/hr}^{0.5} \quad (4.6)$$

The inferential statistics was therefore applied in a comparison of the sorptivity values; Figure 4.9 illustrates comparative box plots for these Projects. The variability of values in Project 9 is observed as the highest; sorptivity values from Projects 4 and 6 are lower in comparison to the three other projects.

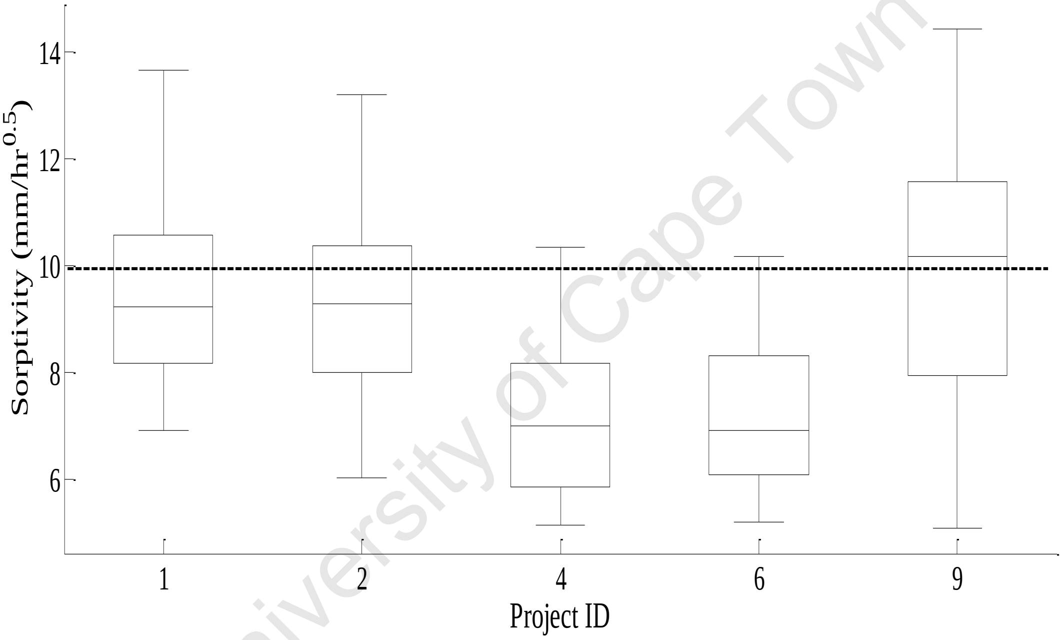

Figure 4.9: Comparative box plot for sorptivity values.

A random sample of 50 sorptivity values was obtained from the five projects considered. Analysis of variance (ANOVA) was done to determine if the in mean values for the five projects differed; hypothesis used is given in Equation 4.7.

$$H_o: \mu_1 = \mu_2 = \mu_4 = \mu_6 = \mu_9$$
$$H_1: \mu_1 \neq \mu_2 \neq \mu_4 \neq \mu_6 \neq \mu_9 \quad (4.7)$$

From the analysis, the P-value obtained is 5.05 E-19 which is less than α of 0.1. Therefore, the H_o was rejected and it was concluded that the average sorptivity values for the five projects differs. This indicates that with different concrete mixes used and methods of construction the sorptivity values for different projects differ.

For Project 4 and 6 the range of values is close. A t-test was done to determine if the difference in mean is significantly different. From the test, the P-value (0.47) was found to be greater than significance level of 0.1.

It was therefore concluded that the mean value from the two projects is not significantly different; this computation is provided in the attached CD.

Discussion

The observations made from the data analysis in the preceding section are discussed below:

(i) The average sorptivity values from all projects comply and are all below the limit value provided in specifications of 10 mm/hr$^{0.5}$. The sorptivity test is meant to be carried out on the same test specimen used in the OPI test. It is expected that with a high OPI value, a low sorptivity value would be obtained. This is however not always the case e.g. for the OPI values, the precast elements had the highest value of 10.25 while it has a high sorptivity value, in comparison to the other projects that had lower OPI values.

(ii) *Extent of variability of sorptivity values*: The variability of sorptivity is high ranging from 15.4% - 22.5%. A high variability in sorptivity was also observed from an interlaboratory exercise reported in Stanish et al. (2005). This high variability from the study was attributed to failure to achieve the same degree of saturation in labs due to difficulties encountered in saturation of samples; this would affect porosity measured which is used in computation of sorptivity.

For this study, there was no information with regard to execution of sorptivity tests in labs. Therefore, the spreadsheets provided from the GFIP were observed to examine the trends in porosity values. Most values ranged from 8 to 16%; however, an extremely low value of 4.8% was observed which may indicate failure to completely saturate a specimen. The saturation of samples is therefore an important aspect to carefully consider in execution of the sorptivity test.

The average values of sorptivity from two projects (Project 4 and 6) were low and values compare favourably with those of samples of concrete from RMC tested (Table 4.3, page 116). This indicates that with proper control and construction practices (more so curing), suitable values of sorptivity can be obtained.

(iii) *Proportion of values that fail to comply with limit value*: This proportion was observed to be high with as much as 50.5% values from Project 9. This high proportion of values that fail to comply is in contrast to what was observed for the OPI values for this project (all values complied with limit value). This observation indicates that a high OPI value is not conclusive in evaluating the penetrability properties of the concrete cover. The sorptivity test is also important as it provides an indication of the effectiveness of curing applied and the resulting sorptivity properties of the cover. Thus, this observation indicates that though high permeability of samples tested is obtained for Project 9, the curing applied may not have been effectively done which led to high sorptivity values.

The specifications provide a limit value of 10 mm/√hr. However, there are no reduced payments for values that do not comply with this value, unlike the provisions made for OPI. A preferred approach to establishing a limit value would be to set a value that is site or project specific that would be determined by pre-testing concrete mixes to be supplied to a project (Alexander, 2011).

4.2.4 Comparison of DI test values from RMC supplier and in situ elements

The concrete mix from RMC supplier was supplied to Projects 1 and 2, from different plants as presented in Table B1, in Appendix B. This section considers the difference in DI values from concrete supplier which indicates the 'material potential' and average values from the two projects. Table 4.12 illustrates this comparison.

Table 4.12: Comparison of 'material potential' values and 'as-built quality' values

	OPI (log scale)	Sorptivity (mm/√hr)
Project 1	9.75	9.67
RMC supplier	10.07	7.60
Project 2	9.91	9.36
RMC supplier	10.17	7.70

The trend observed from the table above is that the mean value for site-based elements is lower than that obtained from RMC supplied. This indicates that in as much as concrete supplied may comply and exceed the durability requirements, the execution of construction practices has a significant effect on the resulting quality of a structure.

4.3 STRENGTH VALUES

The traditional approach in control of concrete quality is through a measure of strength. The data from the GFIP projects was recorded in two formats; using either UCT or SANRAL spreadsheets. Data recorded in SANRAL spreadsheets provided a record of compressive strength (based on an average of three test determinations) while that in UCT spreadsheets does not provide strength data. The section below provides an analysis of strength values from Projects 4, 6 and 9.

Histograms were plotted using strength values from Project 4 and 9 (selected based on a sample size of more than 50 values); these plots are illustrated in Figure 4.10 (a) and (b). The specified strength for these projects was 30 MPa. From these plots, the limit value was attained and exceeded in the two Projects. Some of the strength values in Project 9 were extremely high, with values of more than 75 MPa observed which represents values that are more than twice the specified value.

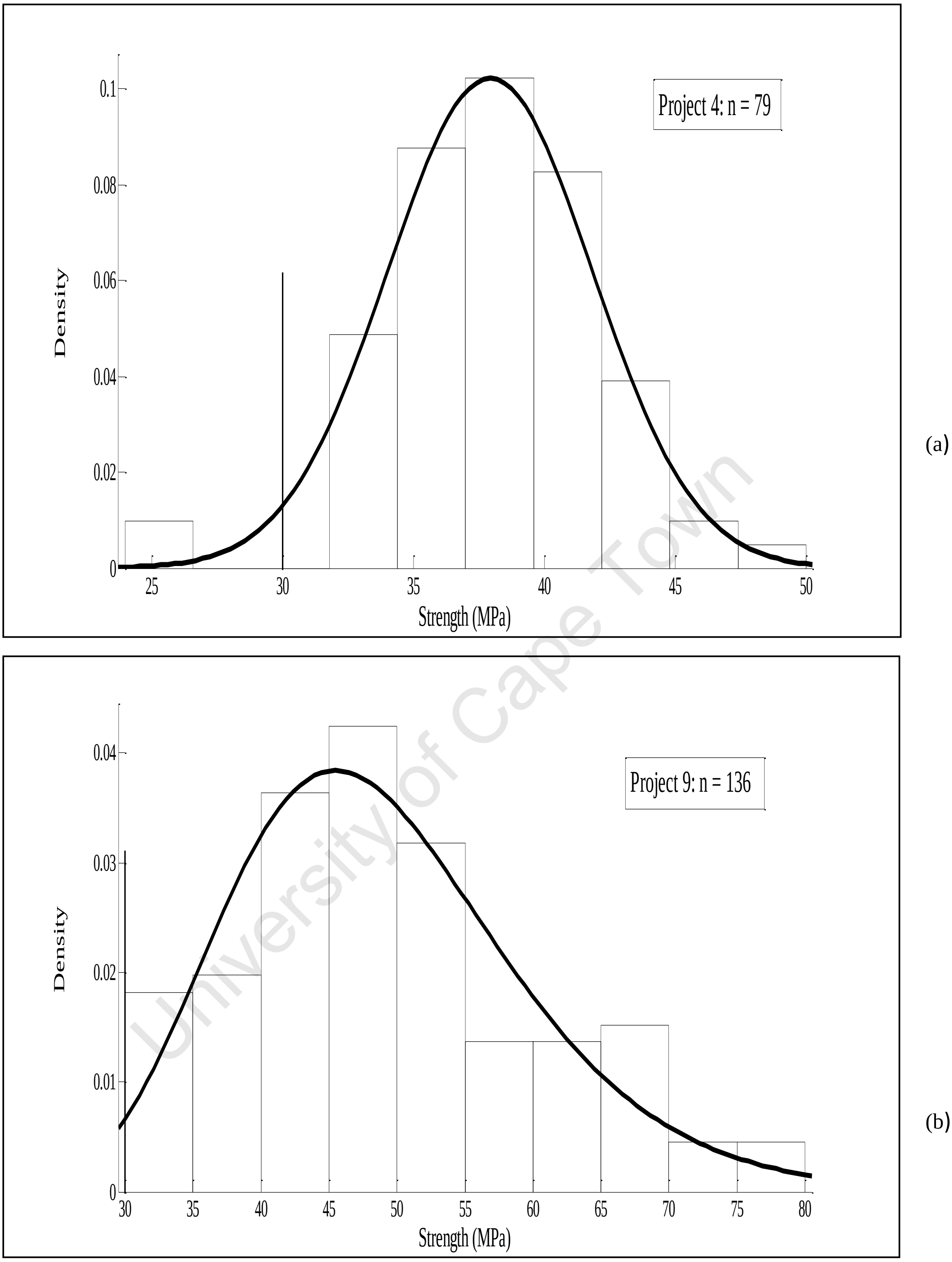

Figure 4.10: Histogram plots for (a) Project 4 and (b) Project 9.

The ***numerical summaries*** of strength values were computed by determining the mean values, standard deviation (s) and coefficient of variation (CoV) are given in Table 4.13.

Table 4.13: Summary statistics of strength values

Project		Strength (MPa)					
ID	n	Specified	Mean	Max	Min	s	CoV (%)
4	79	30	37.9	48.8	25.0	3.9	10.3
	23	40	43.5	49.2	39.9	2.4	5.5
6	22	30	48.2	61.4	37.3	6.7	13.9
	45	40	56.1	71.5	42.8	6.4	11.4
	19	60	79.0	84.6	69.2	3.9	5.0
9	136	30	49.4	77.0	30.0	11.2	22.7

The observations made from computed numerical summaries in the table above are:-

(i) The mean strength values for the three projects exceed the specified values in all three projects. The maximum values observed were quite high with as much as 77 MPa for a 30 MPa specified concrete for Project 9. The reason for such high strength values are due to high binder content used of up to 586 kg/m^3 i.e. Project 9 relates to precast manufacturer, concrete mix properties summarized in Table 3.5 (page 97) where the cement content was 410 kg/m^3 and fly ash content was 176 kg/m^3.

(ii) The variability of values was generally low. From consideration of the classification of degree of control for strength (Walker, 1955; Soroka, 1971) summarized in Table 41, there was “good” control for Projects 4 and 6 and poor control for Project 9. The high variability in this project may have arisen from the wide range of values observed.

(iii) The proportion of values that failed to comply with limit value specified was 1.5% for Grade 30 concrete and 12.2% for Grade 40 concrete in Project 4 which exceeds the allowable proportion of defectives of 5%. For the other projects, all values complied with the specified value.

4.3.1 Comparison of strength values with DI test values

The purpose of the scatter plots in the section below is to compare strength and OPI/ sorptivity values to determine if a relationship exists, i.e. do high strength values lead to high values of the DI tests (OPI and sorptivity).

From the scatter plots in Figures 4.11 to 4.13, a random scatter is observed in all cases indicating that no apparent relationship exists between strength and OPI (sorptivity) values. A higher strength therefore does not always lead to higher DI values. The specified values can therefore be attained with strength that is within the specified range.

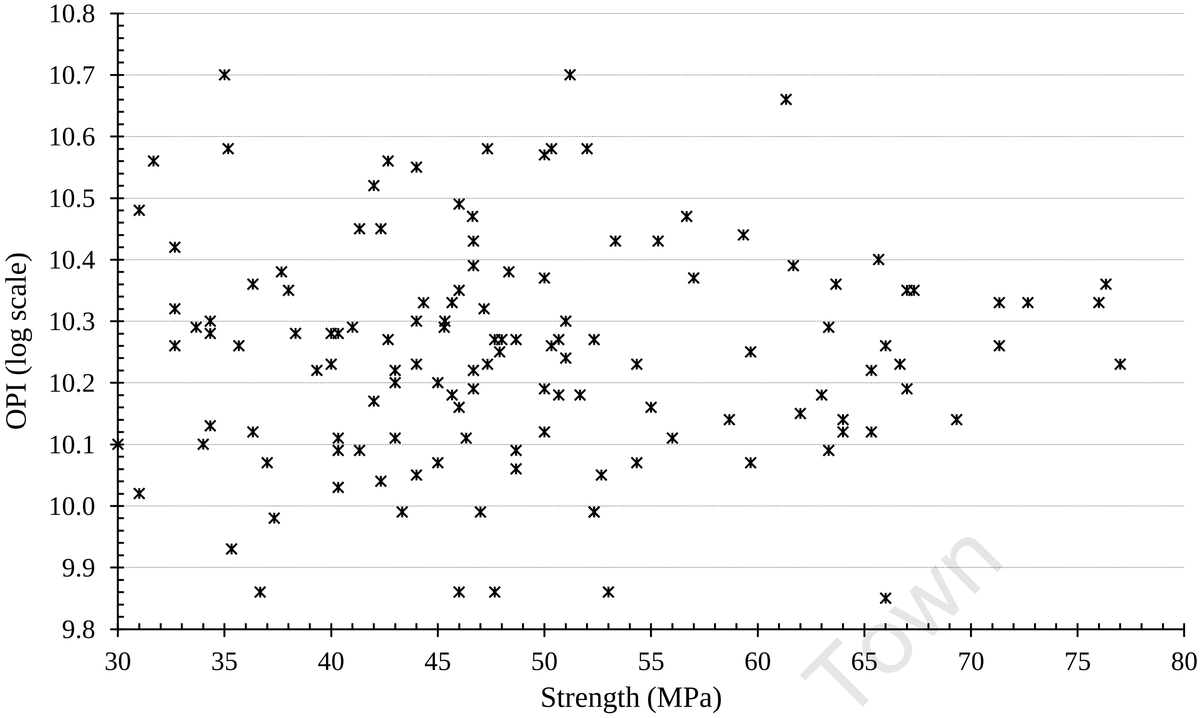

Figure 4.11: Comparison of strength and OPI values for Project 9.

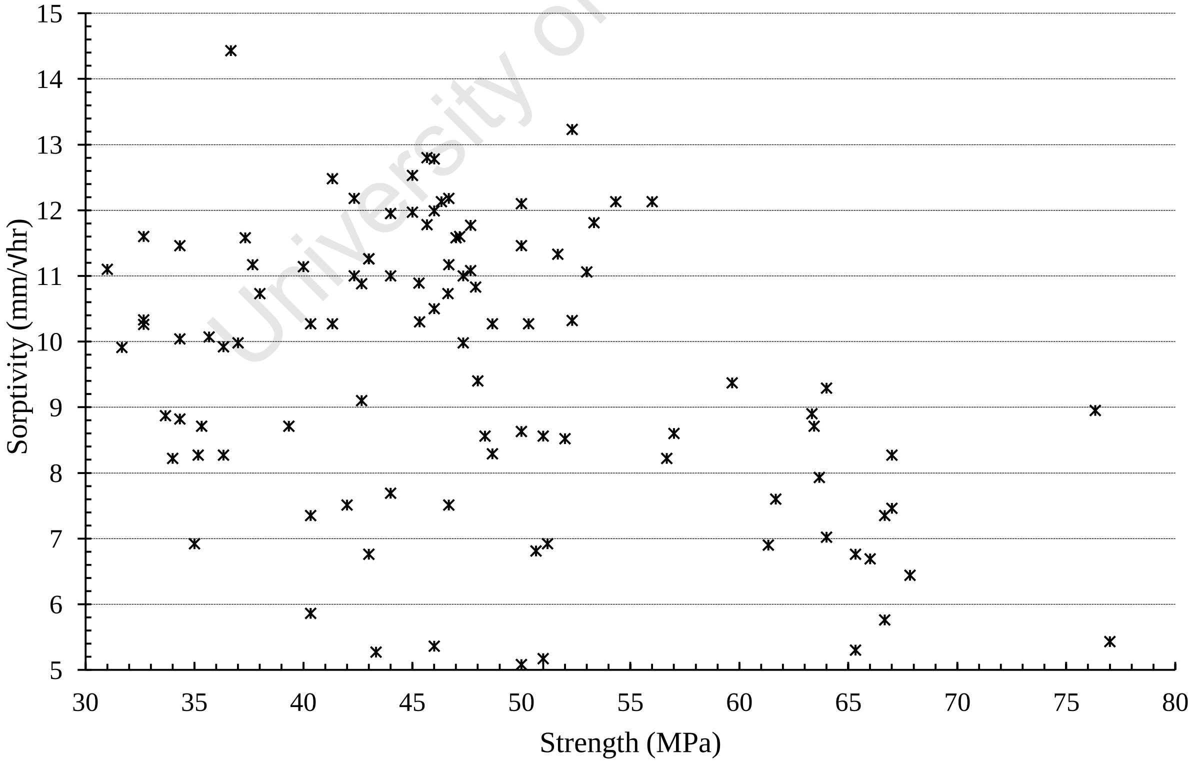

Figure 4.12: Comparison of strength and sorptivity values for Project 9.

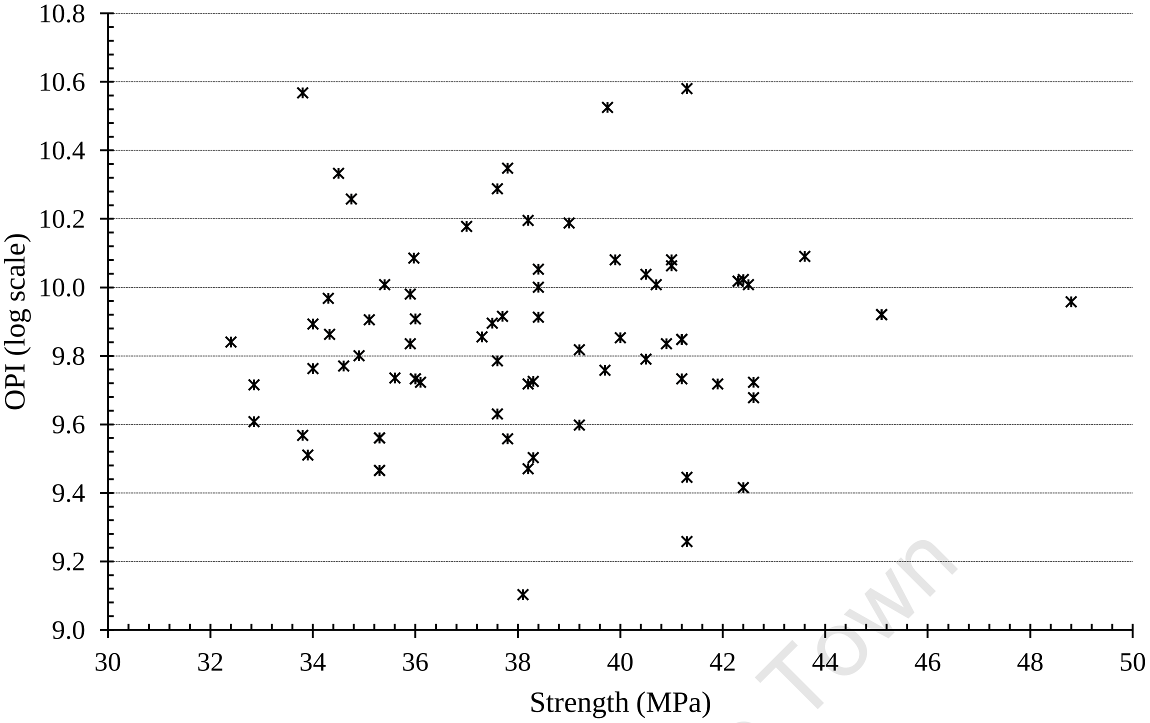

Figure 4.13: Comparison of strength and OPI values for Project 4.

4.4 COVER DEPTH VALUES

In addition to DI test and strength measurements, the DI-based specifications require measurement of cover depth of finished structures. The limiting values provided in the specifications are 40 mm with a minimum value of 30 mm and maximum of 55 mm. A minimum cover depth provision is given for durability requirements while a maximum limit is set to ensure that cracking does not occur in a section, which would increase penetrability. The section below provides analysis of cover depth readings for: in-situ structures where four bridges in different locations of Project 2 are considered; and precast elements in Project 9.

4.4.1 In-situ structures

The initial step in analysis of cover depth readings involved plotting of *histograms* to determine the distribution model of the data and provide an indication on amount of spread in values; these plots are illustrated in Figure 4.14. The histograms are plotted on the same horizontal axis ranging from 20 – 110 mm.

From Figure 4.14, the spread of values is highest in bridges in Location C and D. For the bridge in Location C values range from 20 mm to as much as 100 mm.

The proportion of values that fail to comply with the minimum cover depth provision of 30 mm is low for all bridges in the different locations; however, a high amount of values exceeding the maximum value of 55 mm is observed in bridges in Locations C and D.

For computation of ***numerical summaries*** of cover depth readings, five structural elements from a bridge were considered. The average value and measures of variability (standard deviation (s) and coefficient of variation (CoV)) of these elements were determined. To compute numerical summaries of overall cover depth readings (which are representative of the bridge structure considered), the cover depth readings from the different structural elements were combined. The numerical summaries computed are presented in Table 4.14.

Table 4.14: Numerical summaries of cover depth readings of in-situ structures (Project 2)

			Cover depth (mm)		
Location	Element	n	Average	s	CoV (%)
A	Abutment	152	49.0	16.1	32.9
	Parapet	103	51.7	9.8	19.0
	Parapet	98	48.7	8.6	17.7
	Parapet	105	44.4	8.5	19.1
	Ear wall	119	46.4	3.9	8.3
	All	***547***	***46.6***	***8.9***	***19.0***
B	Abutment	235	54.2	5.4	10.0
	Beam	71	41.7	6.5	15.7
	Beam	75	45.8	9.8	21.3
	Beam	66	44.2	4.7	10.7
	Pier	219	53.3	4.8	9.0
	All	***664***	***50.7***	***7.5***	***14.7***
C	Deck face	186	79.2	13.6	17.2
	Parapet	61	48.2	5.7	11.9
	Parapet	72	47.9	8.3	17.2
	Parapet	68	46.9	5.8	12.3
	Piers	192	50.5	12.2	24.1
	All	***571***	***58.0***	***17.1***	***29.4***
D	Abutment	133	69.0	6.9	10.0
	Abutment	182	49.2	7.2	14.6
	Piers	200	62.4	4.8	7.8
	Deck beam	72	39.7	5.9	14.8
	Deck beam	74	41.8	3.7	8.8
	All	***661***	***55.3***	***12.1***	***21.9***

From the numerical summaries in the table above the following observations were made:-

i. The overall average values in the four locations and average of different structural elements within these locations complied with the limit values provided in project specifications of 40 mm. Mean values from Location C and D exceed the maximum permissible value of 55 mm.

ii. Average values of structural elements complied with limiting values provided in specifications; some cover depth readings were observed to exceed the maximum limit e.g. for bridge in Location C, the deck face has a value of 79.2 mm while for bridge in Location D, readings of 69.0 and 62.4 mm are observed for the abutment and pier.

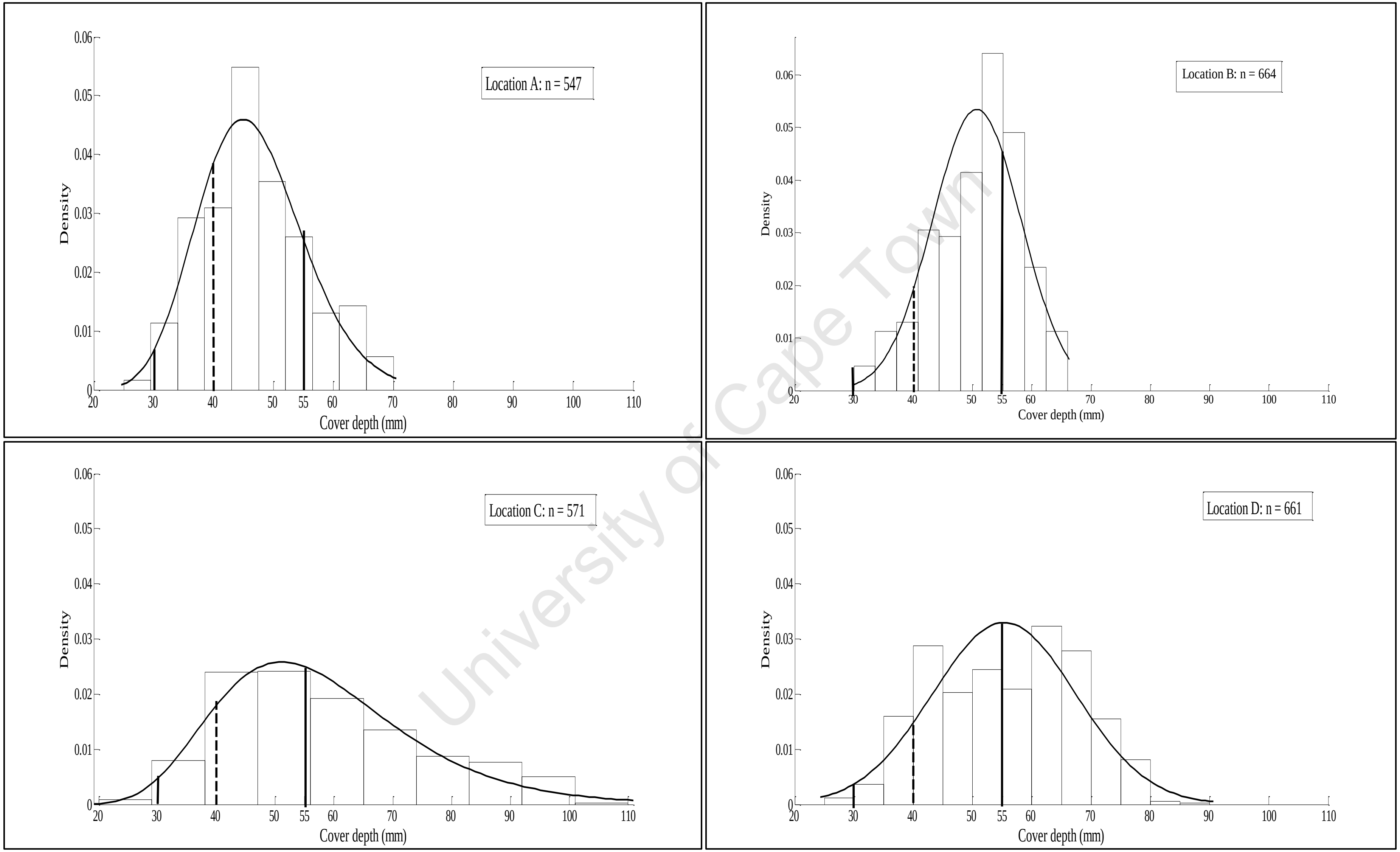

Figure 4.14: Histogram of cover depth readings for bridges in four locations.

iii. The variability of different structural elements, as determined using CoV, was observed to range from values as low as 7.8% (pier, Location D) to as high as 32.9% (abutment, Location A).

iv. The CoV for the overall readings was observed to range from 14.7% to 29.4%.

v. The percentage of cover depth readings that fail to comply with the minimum limit value is low, with the highest proportion being 1.1%. The proportion of values that fails to comply with specified value of 40 mm is highest for Location A with 19.6%. The proportion of values that exceeds the maximum value is observed to be high e.g. in Location D 51.4% of readings exceed this value. A summary of the percentage of values that fail to comply with limit values is given in Table 4.15.

Table 4.15: Proportion of cover depth readings (x) that fail to comply with limit values

	Percentage of values		
Location	< 30 mm	30 < x < 40	> 55 mm
A	0.7	19.6	16.0
B	0	8.7	30.1
C	1.1	10.9	48.9
D	0.6	9.8	51.4

The incidence of cover depths below the required limit value of 40 mm is highest for Project A (20.3%). The implication of such a high proportion of values below the specified limit value is that there is reduced cover depth provision for the structure which may compromise the durability properties of a structure and effectively reduce its service life. From the project specifications, the failure to meet minimum cover depth provisions would result in a reduction in payment of 85% (the criteria for payments is provided in Table 3.1, page 83). For the other locations considered, the cover depth values that fail to meet requirements are less than 15% therefore no reduction in payment would result.

4.4.2 Precast elements

Analysis of cover depth readings from precast elements in Project 9 are presented in the subsequent section. A histogram plot of the cover depth readings and distribution curve is illustrated in Figure 4.15. The range of values is lower in comparison to that of in situ structures (Figure 4.14). Majority of the values comply with the limit value of 40 mm. The proportion of values that failed to comply with this limit value was 3.5%. No values were found to be below the minimum (30 mm) or exceeding the maximum (55 mm) value provided.

Numerical summaries of the cover depth readings were computed and are presented in Table 4.16.

Table 4.16: Summary statistics of cover depth readings of precast elements

Mean (mm)	45.1
Standard Deviation (mm)	2.8
Coefficient of variation (%)	6.3
Sample size (n)	1399

The mean values comply and exceed the limit value provided in specifications. The variability as measured using CoV of 6.3% is low in comparison to that observed for insitu structures.

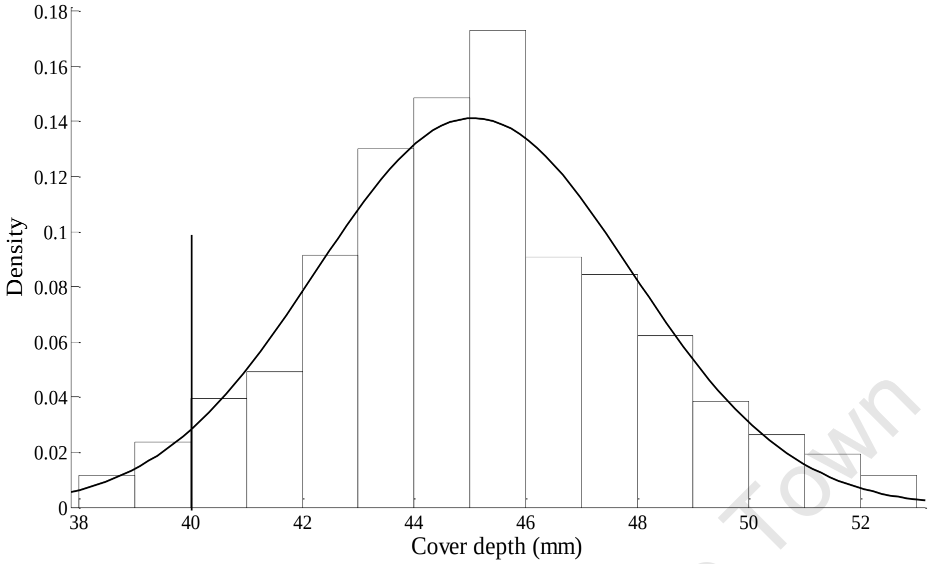

Figure 4.15: Histogram of cover depth readings for precast elements.

4.4.3 Inferential statistics

The use of inferential statistics was applied in the comparison of: cover depths in the four locations considered; insitu and precast elements. The analysis is given in the ensuing sections.

(a) Comparison from four locations

The cover depth readings of in situ elements in four locations were compared. Figure 4.16 presents a comparative box plot of values from these locations. The box plots were plotted using a random sample of 80 cover depth readings from the four locations. The extent of variability in Locations C and D is observed to be higher than that in Location A and B. The dotted lines in the figure represent the upper limit of 55 mm and lower limit of 30 mm.

The analysis of variance was applied to determine if the mean values from four locations differ significantly; hypothesis used for this test is given in Equation 4.9.

$$H_0: \mu_A = \mu_B = \mu_C = \mu_D$$
$$H_1: \mu_A \neq \mu_B \neq \mu_C \neq \mu_D \qquad (4.9)$$

The P-value from the test was 2.38E-05 which is less than α of 0.1. The H_0 was therefore rejected and it was concluded that mean values of cover depth in locations considered differs.

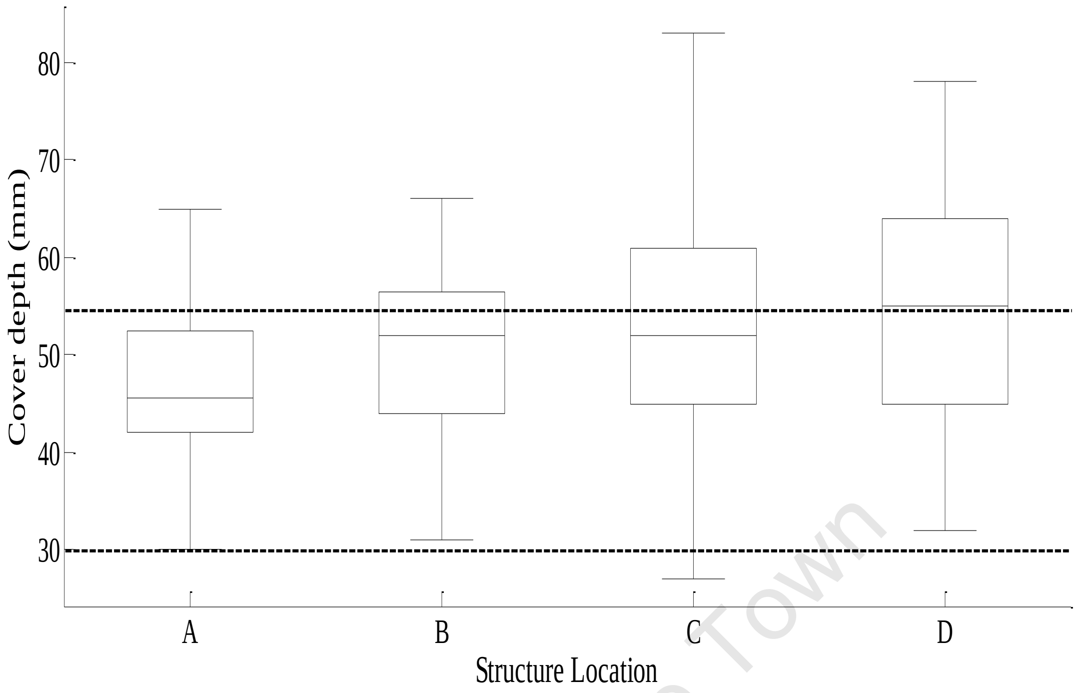

Figure 4.16: Comparative box plot for cover depth readings in four locations.

(b) Comparison of precast and in situ structures

The cover depth readings for the precast and in situ structure are compared; this is illustrated in Figure 4.17 To make the comparison, a random sample of 180 cover depth readings was obtained from precast elements and insitu structures. From the figure, the variability in precast elements is considerably lower than that of insitu structures, indicating a higher level of control is exercised for these elements.

A hypothesis test (t-test) was used to compare if the mean values from these two sources differs. Hypothesis used for this test is given in Equation 4.10.

$$H_0: \mu_{precast} = \mu_{in\ situ}$$
$$H_1: \mu_{precast} \neq \mu_{in\ situ} \quad (4.10)$$

From the test, the P-value obtained was 1.72E-11 which is less than α of 0.1. Therefore H_0 is rejected and it was concluded that mean values from precast and in situ elements differ significantly. This observation indicates that a higher degree of control is undertaken in the placing of concrete to achieve the required cover depth for precast elements in comparison to insitu elements. Therefore, with a high degree of control in execution of the construction practices, variability in cover depth readings can be reduced.

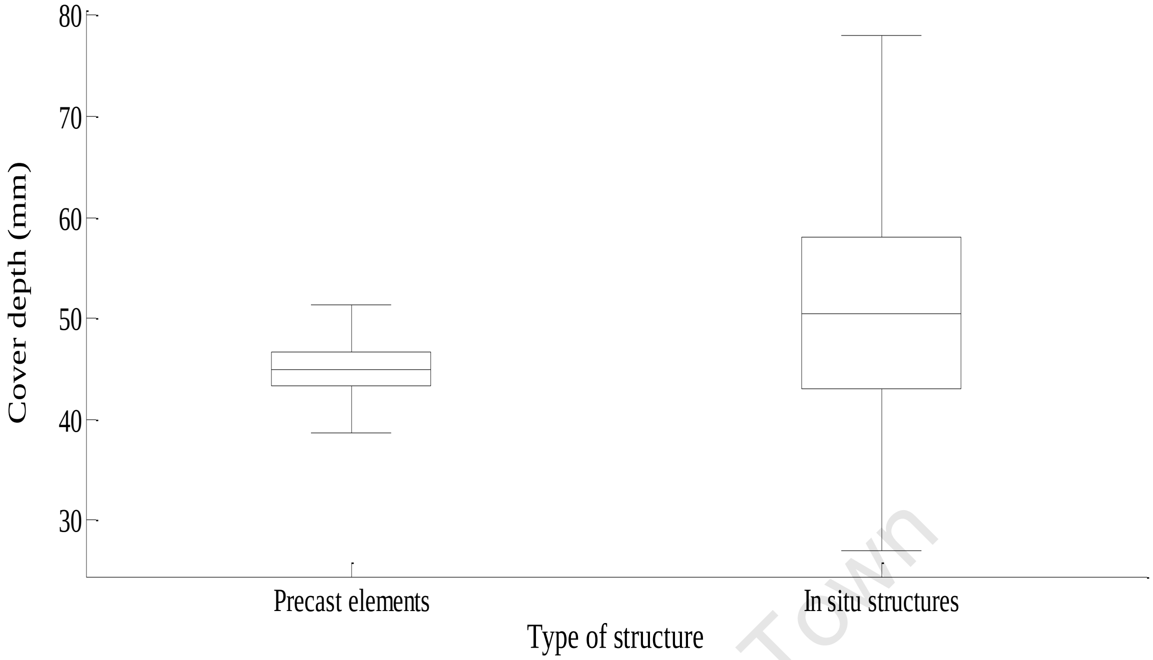

Figure 4.17: Comparative boxplot for cover depth readings from precast and in situ structures.

4.4.4 Discussion

From the analysis of cover depth readings, the following observations were made:-

(i) *Extent of variability in cover depth values*: All the average cover depths for in situ and precast structures considered comply with the limit value in project specifications. The magnitude of variability for in-situ structures varies extensively with CoV as high as 29.4 % obtained from one bridge. The variability of cover depth readings for precast elements is however low, 6.3%, which indicates that a high degree of control is exercised in construction of these precast elements.

(ii) *Proportion of values complying with limit*: The incidence of cover depths below the minimum value is low while values exceeding the maximum permissible value are high, with as much as 51.4% obtained from bridges in Location D. For the purpose of computing the average cover depth for Projects to be used in making payments, a ‘capped’ value of 55 mm was applied for any readings that exceed the maximum value (SANRAL, 2010).

The incidence of values below the limit value of 40 mm was generally low for all the bridge structures considered with the exception of that in Location A. The low incidence of cover depth values below the minimum value indicates that majority of the structures considered comply with durability requirements, in terms of attaining the specified cover depth.

The high proportion of values above the upper limit set of 55 mm may be attributed to cover meters used. Evans (2011) observed that cover depth measurements in excess of that specified were obtained from structures, yet when cores from this structure were taken the required cover had been attained. The cover meters used were observed to give erroneous readings, which is an aspect that should be carefully considered when obtaining and interpreting measurements from these meters. In addition, high variability of cover depth values may indicate poor construction practices e.g. standing on reinforcement by operatives which may lead to misplacement of reinforcement bars.

4.5 GENERAL DISCUSSION

The magnitude of variability in test results is influenced by three main aspects: material properties of sample execution of the test; and quality of workmanship in execution of construction practices (compaction and curing) which influences the final quality of sample tested. The preceding sections have presented statistical analysis of DI test values, cover depth readings and strength values to evaluate the extent of variability. The OPI test was generally observed to have an acceptable level of variability (when compared to past studies) while sorptivity and cover depth values have high variability in results.

This section will provide a general discussion of test results where the three aspects that influence variability – material properties, execution of test and quality of workmanship - will be considered.

4.5.1 Concrete mix properties

The concrete mix properties from the Ready mix concrete supplier and for the precast manufacturer are summarized in Tables 3.4 and 3.5 (page 95). The binder content of these mixes was high, with as much as 586 kg/m^3 for the precast elements. The high binder contents are similar to those used in high performance concrete which range from 400 to 550 kg/m^3. The use of high binder content is aimed at producing concrete of high strength and with low penetrability, thus enhanced durability properties (Neville and Aitcin, 1998). Provisions in the previous Project specifications indicated that to meet durability requirements, high binder contents should be used. With the use of high binder content, high strength values were obtained e.g. for the RMC supplier for a concrete mix with specified strength of 30 MPa, the measured strength obtained was 60 MPa while for precast elements, values exceeding 75 MPa were observed.

From the scatter plots presented in Figures 4.11 to 4.13, it was observed that high strength values do not lead to higher OPI values (or lower sorptivity values). The use of high cement contents is however detrimental as it may increase incidences of thermal deformations (cracks) with an increase in penetrability of concrete, and is also uneconomical (high costs involved with high binder contents) (Long et al., 2001). Therefore, high binder contents need not be used for concrete mixes as high strength does not guarantee lower penetrability and an improvement in durability properties.

4.5.2 Execution of test

From the review of a report on an audit exercise on the OPI test carried out in laboratories (Raath, 2011) it was observed that the test was carried out correctly in most cases with a few deviations in test procedures. The equipment used in undertaking the tests was also observed to vary e.g. differences in hardness of rubber collars used. Despite the variations observed in equipment and execution of test by operators, the variability in results (tests undertaken on ceramic discs) was observed to be low. This indicates that the OPI test is robust and that valid results can be obtained despite variation in test procedures, lab environmental conditions or equipment used. From results obtained, the results of OPI have an acceptable level of variability with the highest observed of 4.60%. Thus, it can be inferred from observations made in laboratory audit exercise that results obtained from testing of site-based samples from the projects are valid and variability indicated by test results arises from inherent differences in materials of samples tested or execution of construction practices, and not due to execution of the test.

The sorptivity test was however observed to have high variability. The incidence of high variability in this test was similar to that observed in the interlaboratory exercise reported in Stanish et al. (2006), though the CoVs from this study were higher. A higher variability would be expected as testing was carried out on site-based samples. The reason for high variability in values may arise due to porosity values measured; this measure is influenced by vacuum saturation of samples, an aspect that was highlighted in Stanish et al. (2006).

4.5.3 Quality of workmanship

The execution of construction practices, more so the curing of concrete structures influences durability properties. From responses to questionnaires sent out to Resident engineers (REs) in GFIP projects, similar curing practices were applied in all projects considered; curing method employed was retention of formwork and application of curing compound on removal of formwork, both for in situ and precast elements. The quality of execution of these practices however varied, which indicates the degree of control on different sites/projects. From Figure 4.1, which presents histograms that illustrate spread of OPI values, it was observed that the values from precast elements (Project 9) had a lower spread in values. This indicates that strict control was applied in execution of construction practices (fixing of reinforcement, compaction and curing) in comparison to Project 6 (in situ elements) where the observed spread in values is high. The response of the RE from this project also indicated that strict controls were undertaken in construction of these elements. This illustrates that with strict control in execution of construction practices, variability in DI test results can be reduced with an increase in quality of RC structures (in relation to durability).

For the sorptivity values the amount of variability for both in situ and precast structures was high and ranged from 15.4% to 22.5%. The precast elements had the highest variability in sorptivity values which is in contrast to what was observed for OPI values where these elements had the lowest variability due to strict controls in execution of construction practices. This indicates that for sorptivity values, variability in values mainly arises from execution of the test in laboratories where it was observed (from review of spreadsheets) that very low porosity values were obtained due to failure to properly saturate specimens.

From the REs response with regards to the effect of implementation of DI tests on construction practices, some respondents perceived that strict controls on quality of workmanship had taken place while others observed that the only change that had taken place was with regards to concrete mix used where cement producers had resorted to use of high cement contents to achieve durability requirements. The observations made by REs are confirmed by high binder content and strength values obtained from concrete mix properties of RMC supplier and precast manufacturer.

4.6 SUMMARY

The Chapter presents statistical analysis of OPI (k-values), water sorptivity (and porosity), strength values and cover depth readings obtained from Projects in the GFIP. The objective of the statistical analysis presented was: to determine the average values from different projects, and if these values comply with the limit values provided; evaluate the extent of magnitude in DI test values, strength and cover depth readings; and to determine the proportion of values that fail to comply with limit values specified. For all the five projects considered, the average value for OPI and sorptivity computed were greater than the specified value. However, the proportion of values that failed to comply with specifications was high with as much as 40.1% for OPI values in Project 1 and 50.5% variability for sorptivity values. This high proportion of values failing to comply with limit value is an aspect that requires to be addressed and provided for in specifications. For the strength values, all the Projects considered complied with the limit value. High values of strength, as much as twice the specified values, were observed. The high strength values observed was attributed to the use of high binder content so as to attain concrete with low penetrability. The strength does not however always relate with penetrability (as measured by DI test values); higher strength does not guarantee high OPI (or lower sorptivity). Majority of cover depth readings complied with limit value with the exception of one bridge (in situ cover depth readings) in which values below limit value of 40 mm was 20.2%. The high cover depth readings were attributed to the cover meter used which in some cases have been found to give erroneous readings.

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Chapter Five

Conclusions and Recommendations

5.1 INTRODUCTION

The durability of reinforced concrete (RC) structures is dependent on penetrability and thickness of the concrete cover. Penetrability properties of the cover are influenced by material properties (water/binder ratio and minimum cement content) and proper execution of construction practices – the placing, compaction and curing of concrete. The provisions in current specifications, which are described as 'prescriptive', are based on the premise that concrete will be 'deemed-to-satisfy' durability requirements on proper selection of exposure class, on attaining limiting values of material constituents and proper execution of construction practices. Limitations have however been encountered with this approach such as difficult to verify limiting values and an inadequate compliance criterion based on strength. To address these limitations, performance-based approaches to durability design have been developed and implemented internationally, and locally using the Durability Index (DI) tests.

The durability index tests were developed to characterize near surface concrete properties with regard to resistance to ingress of aggressive ions that initiate corrosion. The tests were developed to be simple with little demand on operator skill, requiring minimum sample preparation, having low variability and executable within 28 -35 days which would make them applicable for quality control (Alexander and Mackechnie, 2001; Alexander, 2004). These tests have been extensively applied in laboratory studies to characterize concrete mixes, with a few site-based studies. The DI tests have been developed for application in the design and construction of structures for durability through the DI-based performance approach (Alexander, 2008).

The DI-based performance specifications have been implemented on a large scale e.g. in the SANRAL project, GFIP. The main objective of this study was to validate the practicality of the DI-based performance approach in the control of concrete cover quality. To evaluate the practicality of this approach the following aspects were considered:-

a) The extent or magnitude of variability in DI test results (OPI and sorptivity) and cover depth values.
b) Applicability of the DI tests on site (obtaining test specimens from structures and test panels) and in laboratories with regards to execution of the tests by operators.
c) Perception of the DI based approach in practice by Resident engineers.

5.2 CONCLUSIONS

From the collection and analysis of data from various projects and sources of the GFIP, the following conclusions with regard to objectives of the study were made.

5.2.1 Extent/magnitude of variability

The practicality of a test is largely influenced by the ability to obtain reliable results with a low (acceptable) variability. The variability of the OPI test values, sorptivity, cover depth and strength values were considered in this analysis. In addition, the average values were computed and compared with the limiting values provided in specifications to determine average values and if they comply with limit values in specifications. A summary of the range of statistical values for the projects considered (mean values and CoV) and the proportion of values failing to comply with limit value is provided in Table 5.1.

Table 5.1: Summary of range of statistical values determined

Parameter	Average	CoV (%)	Proportion of defectives (%)
a) OPI (log scale)	9.8 - 10.3	1.8 - 4.6	0 - 40.1
b) k-value (E-10 m/s)	0.7 - 1.7	46.0 – 85.0	3.0 – 32.7
c) Sorptivity (mm/√hr)	7.1 - 9.7	15.4 - 22.5	3.7 - 50.5
d) Porosity (%)	11.5- 13.0	12.7 - 13.7	-
e) Cover depth (mm)	46.6 - 58.0	14.7 - 29.4	8.7 – 19.6

From the summary of values in the table above, it is observed that for OPI, sorptivity and cover depth, the average values comply with the limiting value in project specifications. The variability for OPI is low, when compared to reproducibility value of 1.8% obtained from an interlaboratory exercise (Stanish et al., 2005). The sorptivity values have a high variability; the reason for such a high variance may have arisen from failure to properly saturate samples which leads to misleading porosity values. The proportion of values that fails to comply with limiting values is observed to be high for OPI and sorptivity. For the cover depth, the proportion of values considered in the table above are those with values that exceed the limit value of 40 mm. This proportion is observed to be high for one bridge structure which had as much as 19.6% values below the limit value. Such a high incidence of non-complying values would have an influence on durability of a RC structure.

It was observed from the analysis of strength values that the specified strength was exceeded e.g. for Project 9 strength value of 77 MPa was obtained for a specified strength of 30 MPa. The reason attributed for such high strength values was the use of high binder contents. It was previously assumed that high strength would result in low durability values. From a comparison of strength and DI test values (OPI and sorptivity) it was observed that no relation exists between the two. Durability is therefore not dependent on strength.

The compliance of average values obtained from analysis of DI test values and cover depth readings with limit values indicate that the performance specifications are applicable on site. With strict control in execution of structures, low variability can be obtained e.g. the CoV of 1.8% for OPI from one project. However, the proportion of values that fail to comply with limit values is high and is an aspect that needs to be carefully considered in the specifications and in making payments.

5.2.2 Applicability of DI-based performance specifications

The DI tests were developed as simple tests with little demand on test operators and applicable both on site and in laboratories. The objective of evaluating applicability of the test was to determine if they are practical on site and in the laboratory.

To evaluate the practicality (in terms of applicability) of the use of DI-based specifications on site the following aspects were evaluated: if samples are obtained from site elements within the required period of 28 days; if proper recording of sample age is done; and any difficulties that were encountered in obtaining samples from site elements. The main difficulty observed in application of the test on site was with regards to transport of these samples (test panels) to laboratories for testing. They were found to be bulky (size of a panel is 600 mm x 400 mm x 150 mm thick) making it difficult to transport them. In addition, there was lack of proper communication between site and laboratory operators on when to collect samples, which in some cases resulted in delays in testing. It was also observed that test panels were not properly handled on site and their quality varied e.g. variations in dimensions of panels. This would have an effect on the reliability of test results obtained. From these observations it is concluded that the test is applicable on site but improvements are required in the use of test panels. This is further outlined in the recommendations.

The evaluation of applicability of the tests in the laboratories was done to determine: if the labs had the capacity to undertake the tests within the required period; if proper testing as per test manual was undertaken and difficulties encountered in application of the OPI test. It was observed that there was proper application of the OPI test, with a few deviations from test procedures (from review of a report on audit exercise on laboratories). The observed variability from testing ceramic discs was low indicating that the test method is robust, and valid results can be obtained despite variations in application of test procedures by operators and equipment used. The main difficulty encountered in most laboratories audited was lack of proper identification of test samples obtained from site. The laboratories were also reluctant to invest in more equipment to conduct OPI tests until demand for the tests increases; a consequence of the limited test equipment is extended periods (more than 28-35 days) of storage of samples in laboratories, which limits the effective application of the DI tests in quality control.

5.2.3 Perception of DI-based performance specifications

The effective application of DI-based performance specifications is dependent on how this approach is perceived by engineers in practice who are required to provide specifications in projects and takes measures to ensure effective compliance with provisions made. The resident engineers (REs) perception of the test was evaluated from a review of responses to questionnaires sent out to REs involved in GFIP projects. The general perception was that the tests had not had an effect on construction practices and resulted in an increased amount of testing for quality. This increased amount of testing was in some cases difficult to handle due to different equipment and training required for DI tests. However, some REs found that with implementation of these specifications, stricter controls were placed on construction practices (compaction and curing) in order to meet durability requirements. Implementation of the DI-based performance approach can therefore result in improvements in construction practices and more controls in their execution, which is essential in construction of durable concrete structures.

From the above consideration of magnitude of variability, applicability of tests on site and in laboratories, and perception of REs on DI tests, the DI-based performance approach is validated as a practical in the control of concrete structures quality. However some limitations have been identified with the implementation of the approach and recommendations outlined in the subsequent section are proposed to make improvements.

5.3 RECOMMENDATIONS

5.3.1 Improvements in test procedures

From the review on the audit report and analysis of the data, some limitations in the execution of DI tests were observed and the following recommendations are proposed to make improvements:-

(a) A provision in the test procedures is made to discard test specimen that are damaged e.g. due to chipped edges. However, no criterion is given as to what constitutes a level of damage of test specimen that would result in its being discarded. The aspect of level of damage in test specimen should therefore be established.

(b) From the review of audit report, it was observed that laboratories had not determined the volume of permeability cells. This volume should be established in each laboratory where the permeability cell can be filled with water and volume determined. The determination of this volume is important in computation of the k-values (OPI).

(c) The laboratories commented on the tolerance periods provided for oven drying (± 4 hours) and cooling periods (± 2 hours). The main limitation identified with these periods is that it may restrict testing especially where the numbers of specimens are many. It is proposed to reduce the tolerance periods after exploring the effect of shorter periods on testing.

(d) The spreadsheets in UCT should be amended to include a record of strength of test specimen. This would be useful in making comparisons between the parameters and to enable capturing dual aspects of quality control (strength and durability) in one spreadsheet.

5.3.2 Improvements in data recording

The main shortcoming observed in obtaining site-based samples was the prevalence of missing information with regards to:

(a) Casting date of panel – which is required to enable the determination of age of sample
(b) Curing history - curing method used
(c) Proper identifications of sample tested e.g. the structural element it represents, geographical location
(d) Concrete mix properties of sample – binder type, water/cement ratio

This information can only be reliably obtained from site. To ensure its provision, a form of recording should be implemented on site that requires that the above aspects are fully recorded and samples are only accepted for testing in laboratories on provision of this information.

5.3.3 Standardization of test panels manufacture

From REs responses to questionnaire and the review of report on audit exercise, it was observed that difficulties arose with use of test panels. Difficulties identified were with regards to:-

- Lack of proper handling of panels on site which could lead to high variability in results
- Difficulties in the transport of test panels to laboratories due to their large size
- Lack of proper communication between site and laboratory staff on when to collect samples for testing

To address these difficulties, a standardized method of production of test panels should be instituted and provided in the project specifications. Contractors should also ensure proper handling of test panels on site. The same methods of compaction and curing used on actual structure should be executed on these panels to ensure that they are fully representative. The coring of test panels can be altered to take place on site instead of transporting panels to laboratories which would eliminate difficulties encountered in transportation; however, proper packaging of cored samples should be ensured to prevent damage.

5.3.4 Increased awareness on DI-based performance approach

The control of quality of concrete structures using the DI-based performance approach is an important aspect in construction of durable structures. From the audit report, most of the engineers were observed to respond poorly to DI test results which discouraged laboratories from undertaking the tests as the relevance of carrying out these tests were not perceived. The importance and underlying concepts of this approach should be communicated to practicing engineers and test operators to raise their awareness on its relevance, an additional quality control measure to the determination of strength. Proper communication channels should also be established between contractors, engineers and laboratory operators to ensure that at all stages of construction, the durability requirements and their relevance are well understood by the different parties. A common school of thought in industry is that with increase in cover depth, durability issues are addressed. However, the actual depth of cover of finished structure and properties of this cover with regards to its penetrability need to be verified to effectively address durability concerns in concrete structures.

5.3.5 Standardization of cover meters

The cover readings were observed to have high variability with one RE highlighting difficulties in the use of cover meters where many readings were obtained in one location. High values of cover depth, as much as 110 mm were observed in a bridge. These high readings could be attributed to cover meters used. To address the high variability concerns and difficulties in use of cover meters on site, a standardized approach should be developed that ensures proper calibration of the meters, and a guide on proper interpretation of results.

5.3.6 Development of acceptance criteria

From the analysis of data, as provided in Chapter 4 it was observed that the proportion of values that fail to comply with the limit value is high e.g. as much as 40.1% OPI values from Project 1 were defective. The provisions in the specifications are with regards to adjustment of payments when values are less than the limit value of 9.70 for a structural member. An additional provision should be made in specifications on the total allowable proportion of values that fail to comply with limit values for a project. This percentage will be based on the allowable risk for durability requirements and should be practical i.e. it should not be too stringent which would result in penalties that are unfair to contractors.

5.3.7 Application of DI test results in service life prediction

The durability index test results from the nine sources of the GFIP projects can be collated to form a database that represents early age test results from site elements. This database of results can be applied in long term studies where comparisons of early age DI test results with conventional durability tests (e.g. determination of carbonation depth) on the same structural elements after 5, 10 years to 15 years are made, to determine if correlations exist between short term durability index values with long term durability properties of structures, which will further validate the practicality of DI tests.

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APPENDICES

APPENDIX A:

EXTRACT OF SANRAL SPECIFICATIONS

EXTRACT OF SANRAL PROJECT SPECIFICATIONS

B6404 CONCRETE QUALITY

a) General

"When structural concrete prefixed "W" is shown on the drawings, it shall, in addition to the strength requirement, comply with the durability requirements specified in sub-clause 6404(h). "W" class concrete shall not apply to minor structural elements such as side drains and catch pits except in very severe environmental conditions of exposure. Requirements for concrete quality (including any durability requirements) for concrete pavements are found in Section 7100 of the specifications."

b) Strength Concrete

"Where concrete is designated by the prefix "W", e.g. class W30/19, such designations shall denote concrete achieving the durability criteria specified in the relevant tables under subclause B6404(h)."

Concrete Durability

(i) General

Concrete designated by the prefix "W" shall, in addition to the requirements of subclause 6404 (b) comply with the durability parameters described below.

Durability is influenced by the materials used in the concrete, their mix proportions, transporting, placing, compacting and, in particular, curing of the finished cover concrete (concrete layer between the outermost layer of steel reinforcement and the exposed outer surface of the concrete element). The tests required to prove durability performance of the placed concrete are given under subclause B8106 (h). The numbers of tests shown under pay item 81.02 are the minimum requirements that the Engineer considers necessary to achieve the desired quality of concrete. It is the Engineer's responsibility to approve the component materials and their mix properties; however it is the Contractor's responsibility to utilize acceptable component materials and to achieve mix properties complying to the specifications. It is the Contractors responsibility to design and blend materials to produce concrete of the specified quality.

(ii) Durability Parameters

Water sorptivity:	Sorptivity is sensitive to surface effects and may be used to assess the effectiveness of initial curing.
Oxygen permeability :	Permeability is sensitive to changes in the coarse pore fraction and thus a means of assessing compaction of concrete. It is used to quantify the microstructure of the concrete and sensitive to macro-defects such as voids and cracking.

Chloride conductivity: Chloride conductivity provides a method of characterization of concretes in the marine environment and is used to assess the chloride resistance of concrete.

Cover concrete: Cover concrete is the outer concrete layer that protects reinforcing steel. Concrete cover is a requirement for al concrete whether specified as durability concrete (Class “W”) or normal reinforced concrete.

Individual Cover Depth Measurement (CDM): Individual cover depth measurement determined by an electromagnetic cover meter, complying with BS 1881, Part 204.

Average Cover: The average of at least 30 individual CDMs per m^2 determined on a clearly identified area.

Overall Cover: The mean average cover determined for the scanned area per structure

Scan Area: Areas of approximately 1 m^2, randomly distributed over the entire structure, representing at least 5% of total surface area for that structure

Individual bar reading: A minimum of 3 linear CDMs, spaced at 100 mm intervals, representing a single bar of reinforcement

Capped Value: A value in mm, assigned to a cover reading where the raw reading exceeds the specified cover, plus a value (mm) specified by the Engineer.

Quick/Linear Scan: For evaluation of cover depth measurements taken perpendicular to closest rebar in a line covering required area to be scanned.

Image/Block/Grid Scan: Provides an overview of rebar layout. Measurements taken over a square meter clearly indicating position of first and second layer of rebar.

Notes:

1. Water sorptivity and Oxygen Permeability tests are required to assess carbonation resistance.
2. Water sorptivity, permeability and chloride conductivity tests are required to assess chloride resistance

Concrete cover: Concrete cover is a dimensional indicator of cover concrete depth and it varies according to the requirements of the different environmental exposure classes.

When tested in accordance with the test protocols described in B8106 for each potential durability parameter, the concrete shall meet the limits listed in tables B6404/4, and B6404/5.

(iv) Environmental Classes of Exposure

For this project, the environmental classes for carbonation and chloride exposure for the different structural elements are as shown below in Table B6404/3.

Table B6404/3

ENVIRONMENTAL CLASSES OF EXPOSURE FOR ELEMENTS OF STRUCTURE

Element	Carbonation Environment (OPI)	Chloride Environment (Chloride Conductivity
Foundations	n/a	XS1
Substructures	XC3	XS1
Superstructures	XC3	XS1

(Note to compiler: Insert relevant environmental classes, from Table 6000/1 – Concrete Durability Specification Targets (Civil Engineering Structures only) – Update 12 August 2009).

(v) Acceptance Ranges

Table B6404/4

DURABILITY PARAMETERS ACCEPTANCE RANGES

Acceptance Category	**Test No./ Description/ Unit**				
-	**B8106(g)(i) Water Sorptivity (mm/√h)**	**B8106(g)(ii) Oxygen Permeability (log scale)**			
		Parapets	**Sub-structures**	**Super-structure**	**Etc for other members**
Concrete made, cured and tested in the laboratory using Trial Panels	<10.0[1]	> a*	> c*	> e*	> g*
Full acceptance of in-situ using Test Panels	<10.0[1]	> a*	> c*	> e*	> g*
Conditional acceptance of in-situ concrete based on results of Test panels	Not applicable[2]	b* – a*	d* – c*	f* – e*	h* – g*
Rejection based on results of Test Panels	Not applicable[2]	< b*	< d*	< f*	< h*

Note:

1. A value has been given, but the value to be adopted shall be based on the results from design mixes.
2. Although no value has been given due to ongoing research, values above 12 are regarded as poor quality concrete.

* *Note to compiler: The limiting values for OPI to be inserted in Table B6404/4 i.e. values a, c, e and g shall be the "Recommended" values and values b, d, f and h shall be the "Minimum" values obtained from the Table 6000/1, discussed above. The values to be used are based on the cover requirements and therefore vary for the different cover ranges as well as environmental categories chosen.*

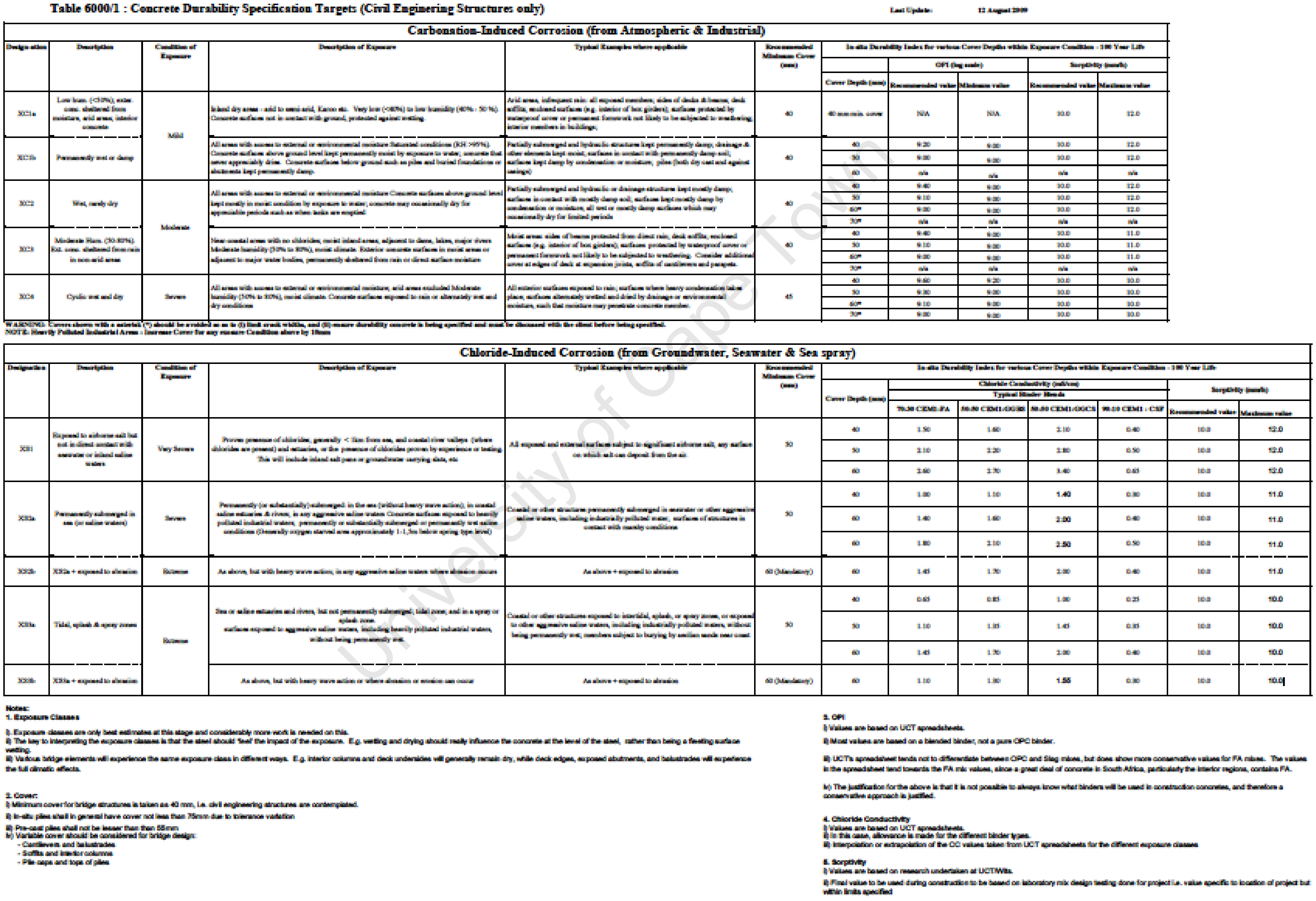


Table 6000/1 : Concrete Durability Specification Targets (Civil Engineering Structures only)

Last Update: 12 August 2009

Carbonation-Induced Corrosion (from Atmospheric & Industrial)

Chloride-Induced Corrosion (from Groundwater, Seawater & Sea spray)

Notes:

1. Exposure Classes

i). Exposure classes are only best estimates at this stage and considerably more work is needed on this.

ii) The key to interpreting the exposure classes is that the steel should 'feel' the impact of the exposure. E.g. wetting and drying should really influence the concrete at the level of the steel, rather than being a fleeting surface wetting.

iii) Various bridge elements will experience the same exposure class in different ways. E.g. interior columns and deck undersides will generally remain dry, while deck edges, exposed abutments, and balustrades will experience the full climatic effects.

2. Cover:

i) Minimum cover for bridge structures is taken as 40 mm, i.e. civil engineering structures are contemplated.

ii) In-situ piles shall in general have cover not less than 75mm due to tolerance variation

iii) Pre-cast piles shall not be lesser than then 55mm

iv) Variable cover should be considered for bridge design:

- Cantilevers and balustrades
- Soffits and interior columns
- Pile caps and tops of piles

3. OPI

i) Values are based on UCT spreadsheets.

ii) Most values are based on a blended binder, not a pure OPC binder.

iii) UCT's spreadsheet tends not to differentiate between OPC and Slag mixes, but does show more conservative values for FA mixes. The values in the spreadsheet tend towards the FA mix values, since a great deal of concrete in South Africa, particularly the interior regions, contains FA.

iv) The justification for the above is that it is not possible to always know what binders will be used in construction concretes, and therefore a conservative approach is justified.

4. Chloride Conductivity

i) Values are based on UCT spreadsheets.

ii) In this case, allowance is made for the different binder types.

iii) Interpolation or extrapolation of the CC values taken from UCT spreadsheets for the different exposure classes.

5. Sorptivity

i) Values are based on research undertaken at UCT/Wits.

ii) Final value to be used during construction to be based on laboratory mix design testing done for project i.e. value specific to location of project but within limits specified

(v) Site Testing

To ensure that the concrete has been placed, compacted and cured correctly, a number of test shall be carried out on the concrete by an approved laboratory.

(ii) Non-compliance with specified criteria

The Contractor should also note that there is specific provision made for curing of concrete under payment item B64.07 of the project specification. The amount priced under this item will be subject to reduced payments should the durability tests indicated under B8106 (h) fail to meet the required targets. Similarly, failure to achieve the required durability test results will be sufficient cause to apply partial payment factors for all the pay items of the elements of the structure under sections 6300 and 6400 of the standard and project specifications or in some cases the removal of the rejected concrete.

B6404 (i) Mix design approval procedures

(i) General

The compressive strength achieved on “W” class concrete shall generally exceed the characteristic strength class structurally required. The Contractor shall note that the process of finalising “W” class mix designs could take up to two months. In order to expedite the process, the Contractor must submit samples of aggregate and cement to an approved laboratory within seven days of the Commencement Date. Should “W” class concrete be required before the mix design is finalized, the Engineer will approve a preliminary mix design in consultation with the Contractor.

(ii) Laboratory designs and site tests, based on Trial Panels

Good mix design practice is essential and the following criteria shall to be taken into consideration when pricing and determination of the mix design:

(1) Selection of sands and aggregates to achieve a good grading is important if the desirable concrete density and durability have to be achieved.

(2) The selection and use of the correct cement grade and type for the environmental conditions (and not based solely on costs) is fundamental

(3) Water: cement ratios are critical, dictating both the structural strength and the durability requirements

Mix proportions for the concrete to be used on site need to be determined by an approved laboratory, Cylindrical specimens, 70 ± 2mm in diameter shall be made or cored from a trial panel during the laboratory trial mix for performance of tests B8106 (g)(i), (ii) (if required). Note that concrete cubes are not cored for durability testing during design trial mix stage or during the construction stage.

Testing for design purposes shall be carried out by an accredited laboratory approved by the Engineer, the costs of which are deemed to be included in the Contractors rates for structural concrete. Concrete as designed shall satisfy the limits set out in Table B6404/3 under the heading “Concrete made, cured and tested in the laboratory, using Trial Panels.” It is therefore a requirement that the trial panels be cast on the site and the cores extracted and tested in the laboratory as part of the mix design approval process. Where the site is remote from the laboratory, the Trial Panels may be cast at the laboratory in accordance with the requirements of sub clause B8106(g).

It will be necessary for the Contractor to establish a target mean strength with a margin above the minimum requirement so that small fluctuations due to material changes or workmanship can be accommodated. In general, mean target strength = characteristic strength + 1,645xSn.

Once the mix is approved, the target mean compressive strength for quality control purposes for durability class concrete shall be the mean compressive strength obtained from the mix that satisfies the durability requirements.

B6409 CURING AND PROTECTION

"Where a curing compound is used, it shall consist of an approved water based low viscosity clear wax emulsion applied in accordance with the manufacturer's instructions."

Add the following paragraphs to the end of this subclause:

"Where curing by retention of formwork is used as the only method of curing the concrete, it must be left in place for the minimum period specified in Table 6206/1 but in no instance shall it be less than 7 days.

The materials used for formwork shall take into account properties such as thermal insulation and moisture absorption when assessing the suitability of the material, to the approval of the engineer.

If impermeable curing membranes are to be used as a curing method, they shall be installed at the same time as formwork is removed and no portion of a concrete surface may be left unprotected for a period in excess of 2hours. If the surface is an unformed finish e.g. top of deck slab, then the surface must be protected immediately by appropriate methods approved by the engineer after it is finished, without damage to that surface, since it is vulnerable to plastic shrinkage cracking due to high rates of evaporation while the concrete is still in a plastic state. Plastic shrinkage and settlement shall not be permitted on any of the structural elements since it compromises the durability of the concrete.

In order to prevent early settlement and shrinkage of the concrete, the concrete placed shall be re-vibrated after initial compaction while the concrete is still in a plastic state. Any remedial measures shall be as approved in writing by the Engineer. On bridge decks, the top surface shall be cured using the method described in clause 6409(d) i.e. "Constantly spraying the entire area of exposed surfaces with water".

For all concrete, curing shall be excluded from the make-up of rates for measurement under items B64.01 and 64.02 and will paid for separately under pay item B64.07. Where the application of a curing compound is used, the type and nominal application rate thereof shall be as specified in the schedule of quantities or to the manufacturer's nominal specified rates."

B6413 PRECAST CONCRETE

(Note to Compiler: For all culverts greater than 1,8m the cast in-situ culvert is preferred due to robustness and cost effectiveness. There have been failures, including durability concerns experienced on some of these bigger precast culverts in certain of the regions

Add the following final paragraph:

"Precast concrete units shall comply with the requirements of the latest SANS 986:2006 specification.

Prior to the manufacture of any units the manufacturer shall submit his Quality Plan to be approved by the engineer. The quality plan must incorporate all requirements and frequency for durability index testing i.e. Sorptivity, Oxygen Permeability, Chloride Conductivity (if required) and Cover Testing. As part of the Quality Plan submitted for approval, copies of calibration certificates of both gauges used for proof loads and cover meters used at the factory shall be supplied to the engineer.

The originals of these certificates shall at all stages also be available for inspection at the factory premises. The manufacturer shall check each precast unit for cover compliance, and random checking of units shall not be permitted. The engineer's representative may visit the factory at any stage to ascertain adherence to the quality plan including test results from the durability index testing as well as to check covers before delivery to site. Any substandard cover shall result in the applicable structural element or part thereof being rejected. Should the manufacturer not be adhering to their Quality Plan, the engineer may exercise the right to reject the use of products from the manufacturer concerned. The employer shall also be informed in all such cases.

For durability requirements due to the reduced cover provided for precast culverts, all such durability testing shall be done in accordance with clause B6404 (h). "

Note to compiler:

For units within the 5km zone from the coast, the very severe exposure category shall be used and increased cover shall be specified by the Design Engineer.

B6414 QUALITY OF MATERIALS AND WORKMANSHIP

(a) Criteria for compliance with the requirement

Add the following paragraphs after the first paragraph:

"The cores shall be taken from the Trial Panels cast using the design mixes made in the laboratory. Where the site is remote from the laboratory, the Trial Panels may be cast at the laboratory in accordance with the requirements of sub clause B8106(g).

In the event that for "W" classed concrete the actual achieved average cube strengths of an element are less than 85% of the target mean strength needed to meet durability requirements or less than 100% of the target mean strength to meet strength requirements, it may result in the durability parameters not meeting the prescribed targets and the Engineer will instruct the taking of cores from the test panel and structure for additional testing. The cost of these in-situ tests shall be borne by the Contractor unless the results are acceptable.

The approved quality control criteria for durability concrete shall be coring and testing of test panels. The frequency of manufacture and coring of test panels shall be as ordered by the engineer and indicated in Tables B8106/1 and 2.

Note to compiler:
When assessing the quantity and frequency of the test panels due consideration shall be given to the different concrete suppliers used under the contract, the number of concrete pours taking place in the day and various elements of bridges being cast on the same day. It may mean that initially the minimum requirements are followed and once a trend with respect to the test results are gathered, that the frequency of manufacture and testing of the test panels can be reduced.

Tests B8106(g)(i), (ii) and (iii) (when required), shall be conducted on cores extracted from the test panels when the concrete reaches the age of at least 28 days. To allow for variability in the material potential, the type of chloride conductivity values shall be limited to 90% of the values indicated in table B6404/4. Test no. B8106 (g) (iv) shall be conducted to confirm that the specified depth of concrete cover has been achieved. The frequency of these tests shall be as described under item B8106 (g). The test results shall be accepted or rejected on the criteria set out in Table B6404/3 and B6404/4 based on the following categories:

(i) Full Acceptance

Concrete shall be accepted unconditionally and full payment shall be made.

(ii) Conditional Acceptance

Concrete may be accepted, based on the cube strength and durability index results, with a warning that construction methods be examined to improve the durability criteria. A reduced payment shall be applied to all the relevant pay items of the specific element under B6300 where the cover requirements are not achieved, and B6400 where the oxygen permeability and strength requirements are not achieved for the non-conforming element or concrete pour as set out in Tables B8212/1 and B8212/2. The decision to accept the substandard concrete at reduced payment shall rest solely with the Employer.

Should the test result(s) indicate conditional acceptance of the element tested, the Contractor shall have the option of carrying out additional tests (on 4 extracted cores) on that element of the structure, at his own expense to confirm or disapprove the original test result(s). These cores shall be extracted within 56 days from the date of the element being cast.

Should the additional test confirm the original test result, then the original result shall serve to determine payment in accordance with Tables B8212/1 and B8212/2.

Should the additional test show that the structure meets the targets, the penalty shall be halved.”

(iii) Rejection

The concrete shall be removed and replaced with fresh concrete at the expense of the contractor, as directed by the engineer.

SECTION B8100: TESTING MATERIALS AND WORKMANSHIP

(g) Trial Panels for Durability Concrete (W class concrete)

As part of the durability class concrete mix design approval process, trial panels shall be constructed on the site (or at the laboratory) before construction of structural elements commences, to ensure that the contractor can successfully achieve the oxygen permeability and sorptivity targets set for the in-situ concrete with method of construction to be adopted. Each trial panel shall be constructed using the same type of concrete mix, shuttering type, placing and curing methods (including application rates of curing compounds if applicable) as to be used on the final structural element to be constructed. The dimensions of such a trial panel shall be 0,40m wide, 0,60m high and 150mm thick. The panel shall be constructed vertically. It is suggested that 2 lifting hooks be cast into the panel to facilitate lifting, moving or disposal of panel. It most likely will be that one trial panel will be required for substructures (piers, abutments, retaining walls, etc) if the same grade concrete is specified for all substructures and another for the decks due to type of casting and curing methods.

The test area for taking of cores (taken in horizontal direction) shall not be less than 100mm from all horizontal and vertical edges. The number of cores to be extracted and tested is described under B8106(i).
The costs for construction of the trial panels shall be deemed to be included under rates for pay item 64.01.

(h) Test Panels for Durability Concrete (W class concrete)

During casting of concrete on site, test panels shall be constructed on the site adjacent to where the concrete element is being placed. Each test panel shall be constructed with the same concrete, shutter type, compaction and curing methods being used in the element being cast (including same vibrator frequency and curing compound application rates), and be left to cure for 28 days adjacent to the concrete element. Thereafter it shall either be cored on site or transported to the laboratory for testing of the required durability parameters. The dimensions of the test panels shall be 0,4m wide, 0,6m high and 150mm thick and be cast vertically to simulate vertical casts of the substructures and vertical faces of bridge decks. It is suggested that 2 lifting hooks be installed at both top ends of the test panel to assist with transport. For precast concrete, test panels will not be constructed, as cores will be drilled from the concrete elements at the precast yard before being placed at its final location. For the horizontal faces of in-situ bridge decks and culverts, test panels will also not be constructed. Instead cores will be extracted from the top surface of the decks.

The frequency of the testing and number of cores to be extracted is described under B8106(i).
The test area for the taking of cores (taken in a horizontal direction) shall not be less than 100mm all horizontal and vertical edges.
The costs for construction of the test panels shall be deemed to be included under rates for pay item 64.01.

(i) Testing for concrete durability

Durability predictions for durability concrete prefixed "W" will be based on the following tests that shall be carried out by an accredited laboratory approved by the Engineer :

(i) Oxygen permeability
(ii) Water sorptivity
(iii) Chloride conductivity (if specified)

Notes:-

The test methods shall be as described below.
For test no's (i) and (ii) (and (iii) when required), cores of 70 ± 2 mm diameter shall be extracted from the test panels when the concrete reaches the age of at least 28 days and tested for the durability criteria set out in clause B6404 (h) and used to determine the payment as per Table B8212/1. Test No. (iii) may only be required where specified (e.g. within a chloride environment along the coast or where chlorides are present in ground water).

A sample for the purposes of durability testing is as defined in Table B8106/1. The cores for durability testing shall be extracted from the test panels for process and acceptance control (at the frequency as shown in Table B8106/2). Durability testing shall only be required for concrete specified as durability concrete with the prefix "W". The number of samples to be taken shall be as shown in Table B8106/2.

Table B8106/1

NUMBER OF CORE RESULTS REQUIRED FOR A SINGLE SAMPLE FOR DURABILITY TESTING

Durability Parameter	*No. of Core Results*
a. Sorptivity	*2*
b. Oxygen Permeability	*4*
*c. Chloride Conductivity**	*4*

* Test undertaken only if specified and within a chloride environment.

Table B8106/2

NUMBER OF TEST PANELS REQUIRED FOR DURABILITY TESTING

Element	No. of Test Panels to taken (see Table B8106/1 for number of core results required for a single sample)
In-situ Bridge Decks	1 (per pour)[1]
Bridge Piers/Abutments	1 (per element)[2]
Precast Elements	1 (per element)[2,3]
Bridge/ Culvert Parapets	1 (per element)[2]
Culvert walls/wing walls/slabs	1 (per wall section)[1,2]
Retaining walls	1 (per wall section)[2]
All bases	1 (per element/pour)[2]

Note

1. Test panels required to be cast vertically. Additional cores required to be extracted from top of deck/major culvert slabs i.e. in-situ cores

2. Note that where group of elements are cast on the same day, only one test panel will be required but only if the same grade concrete is used.

3. Sample required to be taken from precast element in casting yard. For edge beams, inner face to be cored.

For cores to be extracted from precast elements and top of bridge decks, the Engineer will indicate the positions at which the cores will be extracted. Filling of the holes left by the drilling of the cores shall be the responsibility of the Contractor and shall be carried out using an approved proprietary non-shrink repair mortar so as to restore structural integrity and durability of the structural element tested.

Note:

If the test results from the test panels indicate that the durability requirement has not been achieved, then the structural element shall be cored and tested for the durability criteria. The Engineer will indicate the positions at which the cores will be extracted. The costs for testing of the structure shall be borne by the contractor. Filling of the holes left by the drilling of the cores shall be the responsibility of the Contractor and shall be carried out with material as described in the paragraph above.

Note that if testing has to be undertaken on sides of decks and walls, the cores shall be taken on the exposed faces of the concrete i.e. the sidewall face taking care not to cut the reinforcing bars. Where the cores do contain pieces of reinforcing steel, they shall not be used for the tests.

The cores shall be extracted through the cover concrete from the Test Panels or constructed concrete element as applicable. The outer 5mm of the exposed surface of the core shall be cut off and then a slice (30 ± 2 mm thick) shall then be cut and prepared for testing. The Engineer will indicate the positions at which the cores will be extracted submitted at least once a month in the required format to the University of Cape Town.

B8212 DETERMINING REDUCED PAYMENTS FOR ‘W’ CLASS CONCRETE AND COVER METER TESTING

Payments for all durability concrete prefixed ‘W’ shall be based on the test results of the compressive strengths and of the durability parameters i.e. oxygen permeability (from test panels) and cover meter testing as indicated in Table B8212/1 and B8212/2.

General note:

The percentage payment shall be applied to a specific concrete member and shall apply to the relevant pay items of sections 6300 (based on concrete cover test) and 6400 (based on the worst results from oxygen permeability and compressive strength tests.

Table B8212/1: TABLE OF REDUCED PAYMENTS FOR OXYGEN PERMEABILITY INDEX ‘W’ CLASS CONCRETE

Description of test	Oxygen permeability index (log scale)	Percentage (%) payment
Full acceptance	> 9.70*	100%
Conditional acceptance (with reduced payment)	> 8.75* ≤ 9.70*	80%
Rejection	< 8.75*	Not applicable

* *Note to compiler: The values shown in the table above as well as Table B6404/3 will in future be obtained from the table titled: Table 6000 – Concrete durability specification targets – Update July 2009.*

Table B8212/2: TABLE OF REDUCED PAYMENTS FOR CONCRETE COVER

Concrete cover (mm)	**% of specified cover**		**Percentage (%) Payment**
	Overall	**Individual bar**	
Full acceptance	≥ 85% <(100% + 15 mm)	≥ 75% <(100%+25 mm)	100%
Conditional acceptance (with reduced payment)	<85% ≥75%	<75% ≥ 65%	85%
Conditional acceptance (with remedial measures as approved by the Engineer and reduced payment)	<75%	<65% ≥ 55%	70%
Rejection	<65%	<55%	Not applicable

The following notes shall apply to Table B8212/2:

1. For cantilevers, the cover shall in no instance be greater than 10 mm of the specified cover for the top reinforcement.
2. Percentage payment for concrete cover shall be based on the average number of cover meter tests performed on a particular concrete element.

TESTING FOR CONCRETE COVER

Concrete cover testing shall be conducted using an approved calibrated electromagnetic cover meter, able to comply with requirements as defined in linear and block scans, and has the ability to save and calculate data measured.

The testing (non destructive) shall be conducted to confirm that the specified depth of concrete cover has been achieved. The cover meter tests shall cover at least 1m^2 for every 20m^2 surface area of concrete placed.

Readings shall be taken to identify individual bars, with at least 3 readings at 100mm spacing on every single bar within 1 m^2. The average cover of the 1 m^2 subjected to the test shall be used to determine the payment as per Table B8212/2 unless the Contractor chooses to carry out additional tests as detailed in the final paragraph of clause B6414 (a). The cover meter must be calibrated whenever being used to test for cover on each project. Standard Calibration block to be use on each project, and where substantial testing is required, the calibration block shall be kept on site. Cover meters shall comply with the relevant modern standards (e.g. EN55011, 50082-1, 6100-6-1, 6100-6-2, 6100-6-3, 6100-6-4 and BS1881 Part 204).

Critical elements for cover surveys are parapets, deck edges including underside of cantilevers, lower portions of columns and abutments and walls. Soffits should be excluded from measurements. All parapets (F-shaped) including the parapet beam shall be fully tested for cover compliance. In addition, the entire area up to 1,5m high on piers, walls and abutments, including the rear of abutments and wing walls shall be fully tested before being backfilled. The Engineer will identify other critical areas requiring to be surveyed. Should any of these areas show deficiencies, the Engineer may order additional cover tests on other areas at the Contractors costs.

The procedure for testing for depth of reinforcement from concrete surface shall be in accordance with the manufacturer’s requirements for the relevant electromagnetic cover meter, but further requirements are set out in clause B8119. All cover meters shall be calibrated on site under the control of the Engineer. The number of readings taken of the layer of rebar closest to the concrete surface to each 1m^2 to be tested shall be such that an accurate average cover can be determined for the tested area.

For the purposes of calculating the average depth of cover bars that have covers 15mm or greater than what is specified shall be capped at specified cover plus 15 mm in the calculations. For calculation of payment, specified cover to be reduced by 5mm (allowance for variation of equipment), before applying criteria as defined in Table B8212/2.

Example, where Specified cover = 40mm, test as 35mm, then apply limits, 85% * 35= 30mm.

Quick Scan readings are to be taken perpendicular to the layer of rebar closest to the concrete surface for each scan area (+/- 30 per m^2), so that an *average cover* to reinforcement can be determined for the tested area.

Readings are to be taken to identify individual bars within each1 m^2. At least three cover readings, at 150mm spacing, per an individual bar shall be shown in the test results but only overall cover measurement would be used for payment purposes. Reports generated by the equipment shall be used for determining payment. Where more than 10% of readings are below specified lower limit, the area shall be re- scanned, by *Image, Block or Grid scan method*, to verify the average cover. For calculation, refer to specific worksheet (attached).

Cognizance to be taken of the effect to cover depth measured, where spliced bars are measured in same area as single bars. The size of rebar shall be corrected manually on the device by means of applying the following formula (approx., 1, 41 x diameter of rebar as shown in design).

Where insufficient cover are established before placing of concrete, e.g. Starter bars from base not correct position, remedial action to be performed before continuing with next concreting – these actions to be clearly recorded and area identified.

B6307 COVER AND SUPPORT

"Where no cover is indicated, the contractor shall inform the engineer who shall after consultation with the design engineer indicate the required cover in writing and the as-built drawings shall indicate such cover".

"Concrete cover and spacer blocks shall be made using the same cement and aggregate type as the main concrete with the same water/ cement ratio so that differences in shrinkage, thermal movements and strain are minimized. Cover blocks shall be water cured by submersion for a minimum of 7 days and thereafter kept submerged in water until immediately before fixing onto reinforcing steel. Where concrete cover blocks, subsequent to fixing, have visually dried out they shall be remoistened by an appropriate method so that they are damp before the placing of concrete. Only semi-spherical concrete cover blocks shall be used. Where fixing wire is inserted into cover blocks, it shall be galvanized. Cover and spacer blocks manufactured from other materials e.g. plastic or wood shall be not be permitted. All cover blocks regardless of the type of material manufactured from, shall not be visible on exposed concrete surfaces."

Table B6404/5

DURABILITY PARAMETERS ACCEPTANCE RANGES: Cover for all concrete types

Test No.	Description of Test	Specified Cover (mm)	Acceptance Range	
			Min	Max
			Overall cover	Overall cover
B8106(g) (iv)	Concrete cover to reinforcement (mm)	30 to 80	85% of specified cover – 5mm	Specified cover + 15mm or where member depth is less than 300mm the limit accepted in writing by Design Engineer.

B8119 CONCRETE COVER TESTING PROCEDURE

1. Scope

This procedure covers all measurements to be done on concrete structures to establish conformance to specified concrete cover requirements.

2. Guidelines and Preparation

2.1 The contractor is to complete a cover survey request and forwarded to the engineer.

2.2 The selected area for cover measurement is to be indicated on a sketch (see example attached).

2.3 The responsible person must identify the area to be scanned, take measurements on the required date and calculate the results in terms of project specification.

2.4 The cover meter is to be checked against a calibration box/block constructed with typical reinforcement of known parameters, on each day of use. Any deviations from actual measurement must be recorded on Cover Survey Request.

2.5 Cognizance shall be taken of the effect to cover depth measured, where spliced bars are measured in same area as single bars. (Typically, the rebar diameter is increased by a factor of 1.44.)

2.6 The depth of cover shall be determined with equipment, complying with BS 1881, Part 204 and capable of identifying the location and depth of reinforcement on a scanned area. The results shall be recorded electronically by the equipment software.

2.7 Measurements are to be taken in accordance with cover meter manufacturer's guidelines.

2.8 The person responsible for measurements must indicate the position, dimension, type and splicing of reinforcement on the sketch for each scanned area.

3. Method of Measurement

Two methods of measurement are proposed as follows:

3.1 Quick/Linear Scan Method:

(i) Readings are to be taken perpendicular to the layer of rebar closest to the concrete surface for each scan area (+/- 30 per m^2), so that an *average cover* to reinforcement can be determined for the tested area.

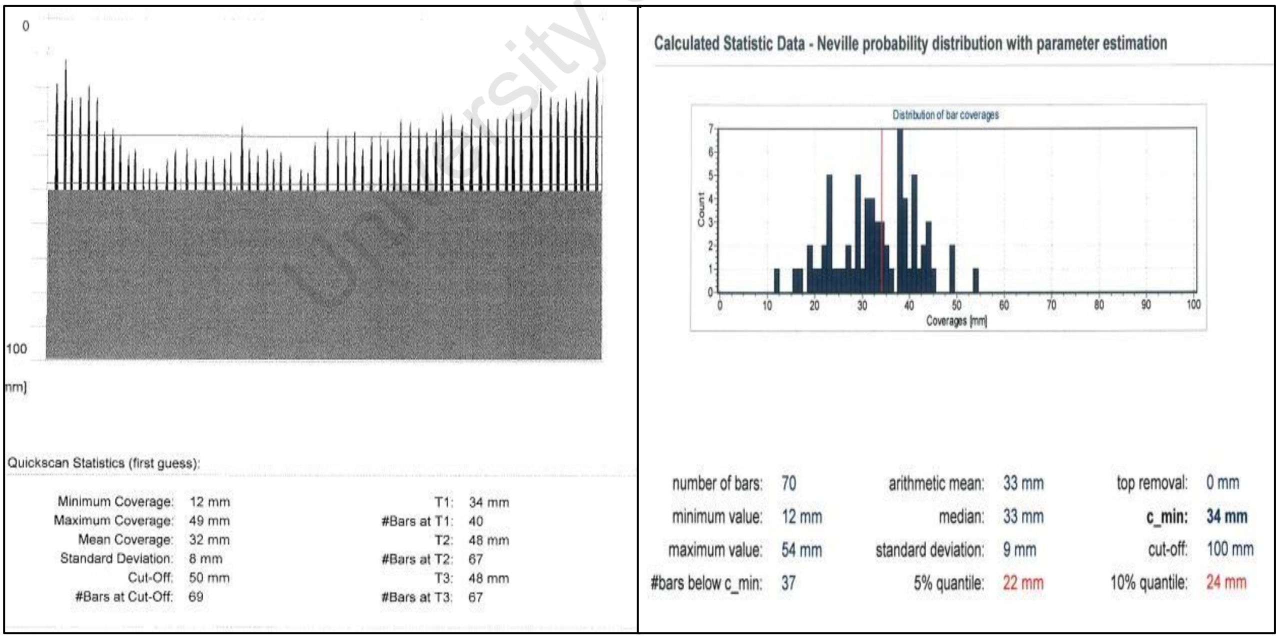
(ii) Readings are to be taken to identify individual bars within each1 m^2. At least three cover readings, at 150mm spacing, per an individual bar shall be shown in the test results but only the overall cover measurement would be used for payment purposes. Reports generated by the equipment shall be used for determining payment. Further specified cover to be reduced by 5mm (allowance for variation of equipment), before applying criteria as defined in table B8212/2a. e.g.: If specified cover is 40mm, the lower limit for full acceptance is:

(40 mm-5 mm) x 85% = 30mm

(iii) Where more than 10% of readings are below specified lower limit, the area shall be re-scanned, by *Image*, *Block or Grid scan method*, to verify the average cover. Refer to item 3.2 below.

An example of Quick Scan information and presentation is shown in Figure B8119-1 below:

Figure B8119-1: Example of a Quick scan output



```
Quickscan Statistics (first guess):

  Minimum Coverage:  12 mm               T1:  34 mm
  Maximum Coverage:  49 mm     #Bars at T1:  40
     Mean Coverage:  32 mm               T2:  48 mm
Standard Deviation:  8 mm      #Bars at T2:  67
           Cut-Off:  50 mm               T3:  48 mm
 #Bars at Cut-Off:   69        #Bars at T3:  67
```

Calculated Statistic Data - Neville probability distribution with parameter estimation

```
   number of bars:  70          arithmetic mean:  33 mm     top removal:  0 mm
    minimum value:  12 mm                median:  33 mm          c_min:  34 mm
    maximum value:  54 mm    standard deviation:  9 mm          cut-off:  100 mm
#bars below c_min:  37             5% quantile:  22 mm    10% quantile:  24 mm
```

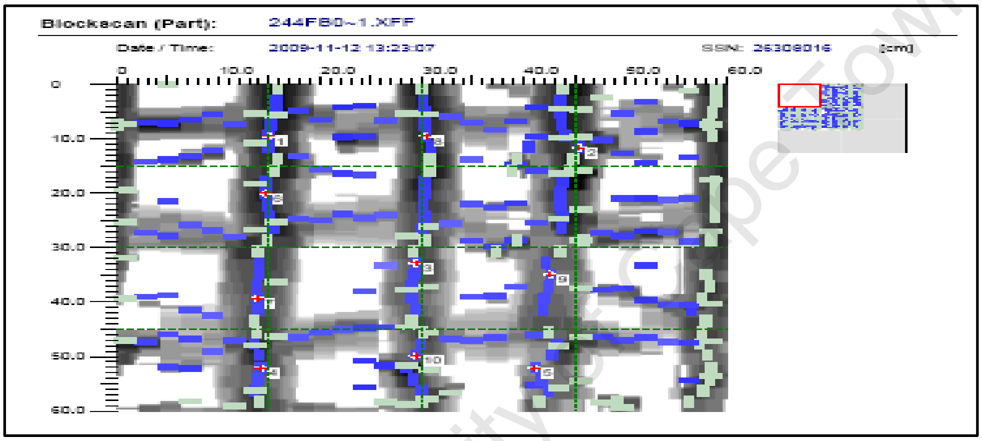
3.2 Image/ Block/Grid Scan Method

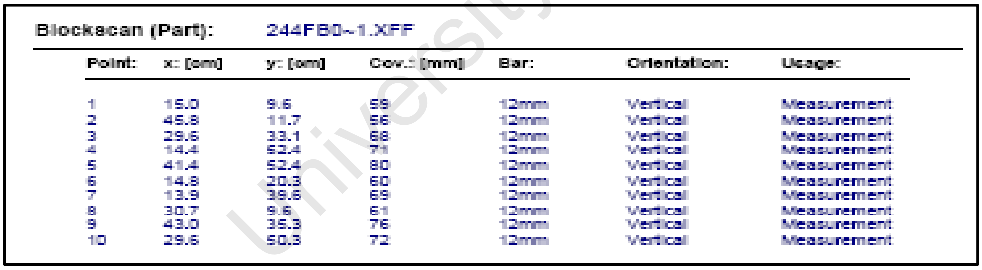
(i) Readings are to be taken in both directions of a marked grid as per the equipment manufacturer’s recommendations.

(ii) This method shall be used to determine the average cover to reinforcement when more than 10% of the Quick/Linear Scan results do not meet the specified lower limit for overall cover.

(iii) For purposes of calculation of the averages for cover of a rebar layer, readings exceeding upper limit (cover + 15mm) to be capped on upper limit. Further specified cover to be reduced by 5mm (allowance for variation of equipment), before applying criteria as defined in table B8212/2a. e.g. if specified cover is 40mm, the lower limit for full acceptance is: (40 mm-5 mm) x 85% = 30 mm

An example of Image Scan information and presentation is shown in Figure B8119-2 below:





Blockscan (Part): 244FB0~1.XFF

Point:	x: [cm]	y: [cm]	Cov.: [mm]	Bar:	Orientation:	Usage:
1	15.0	9.6	59	12mm	Vertical	Measurement
2	45.8	11.7	56	12mm	Vertical	Measurement
3	29.6	33.1	68	12mm	Vertical	Measurement
4	14.4	52.4	71	12mm	Vertical	Measurement
5	41.4	52.4	80	12mm	Vertical	Measurement
6	14.8	20.3	60	12mm	Vertical	Measurement
7	13.9	39.6	69	12mm	Vertical	Measurement
8	30.7	9.6	61	12mm	Vertical	Measurement
9	43.0	35.3	76	12mm	Vertical	Measurement
10	29.6	50.3	72	12mm	Vertical	Measurement

Figure B8119-2: Example of an Image scan output if the equipment used is not able to provide the above presentation it has to be done manually by determining the grid of rebar, first and second layer closest to surface, and manually record readings in order to establish the depth of rebar, as shown in Figure B8119-3 below.

APPENDIX B:
DATA COLLECTION

Table B1: Mix properties for Ready mix concrete

RMC plant	**Concrete mix code**	**Admixture Omega 101**	**Aggregates #1(kg/m^3)**					**Clinker**	**GBBS****	**Fly ash**	**Total binder**	**Water content (litres)**	**Density**	**w/c ratio**
			Stone			**Sand**								
Plant: W		**(g)**	26mm	22mm	13mm	Crusher	Filler	**kg/m^3**						
6.11.2008	W140E4FA	2160	0	884	221	511	126	383	0	68	451	199	2392	0.44
	W140E4FC	2238	0	669	287	570	188	396	0	70	466	204	2384	0.44
	W130E4DA	2249	0	0	1055	527	130	398	0	70	468	204	2384	0.44
Plant: X														
3.09.2008	W140E4EA	2703	0	884	221	579	30	383	0	68	451	200	2365	0.44
	W140E4EC	2836	0	686	294	589	103	402	0	71	473	208	2353	0.44
Plant: Y														
12.2008	W140E4FA	2592	0	900	225	633	146	383	0	68	451	184	2539	0.41
	W140E4EC	2734	0	750	250	652	201	403	0	71	474	192	2519	0.41
Plant: Z														
6.11.2008	W130E4FA	2160	0	884	221	583	144	360	90	0	450	202	2484	0.45
	W130E4FC	2237	0	686	294	613	202	373	93	0	466	207	2468	0.44
	W130E4DA	2213	0	0	980	658	163	369	92	0	461	207	2469	0.45

#1 Aggregate description

- Juksei, Wynberg and Kya Sands plant

For 22 mm, 13 mm stone and crusher sand: Granite
Filler sand – weathered granite

- Spartan plant

For 22 mm, 13 mm stone and crusher sand: Dolomite
Filler sand – Labuschagne
****** Ground Granulated Blast furnace Slag obtained from slagment in Vanderbijlpark

- Values in brackets beside the concrete mix code indicate specified strength for the concrete.

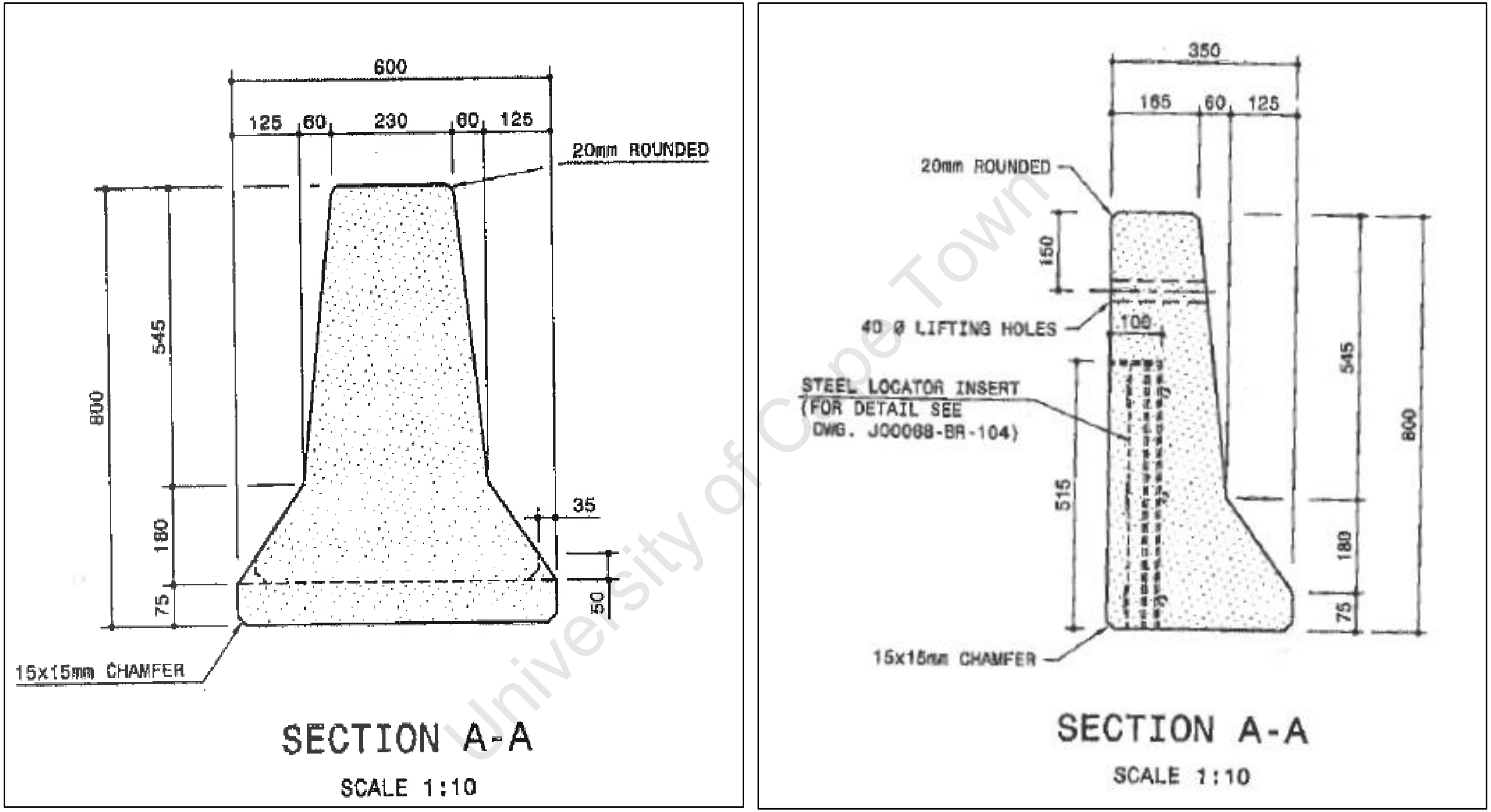
Table B2: Concrete mix properties of precast elements

Mix constituents		Proportion (kg/m^3)
Cement		410
Fly ash		176
Total binder content		586
Water content (L/m^3)		220
Aggregate: Dolomite	26 mm	1160
	19 mm	394
	13 mm	96
Water/binder ratio		0.38

Table B3: Durability index test values for concrete mixes from RMC plants

RMC plant	Concrete mix code	OPI (log scale)	Sorptivity (mm/√hr)	Porosity (%)	Measured strength (MPa)
W	W140E4FA (40)	10.14	7.80	7.50	61
	W140E4FC (40)	10.40	7.30	7.80	62
	W130E4DA (30)	10.13	8.30	8.80	61
X	W140E4EA (40)	9.89	8.10	11.80	59
	W140E4EC (40)	9.86	6.90	12.70	60
Y	W140E4FA (40)	10.34	7.80	7.60	63
	W140E4EC (40)	10.62	8.50	7.80	64
Z	W130E4FA (30)	9.96	8.60	7.30	60
	W130E4FC (30)	10.07	7.40	7.60	61
	W130E4DA (30)	10.34	6.70	8.60	60

NB: The values in brackets adjacent to concrete mix code indicate the specified characteristic strength.



(a)

(b)

Figure B1: Schematic representation of precast barriers, (a) Double sided barrier, (b) Single sided barrier.

Questionnaire

The questions below are aimed at obtaining on-site information on the construction processes used in the Work packages involved in the Gauteng Freeway Improvement Project (GFIP).

1. What form of curing is used and what is the duration of curing?

2. What method of compaction is used?

3. What is the source of samples used for testing – cubes, test panels or the actual structure?

4. Was the age at which samples are extracted recorded?
   (By keeping a record of the time of casting and time at which sample was obtained from its source)

5. Were checks done to ensure correct placing of concrete e.g. checks on cover depth using cover meters, use of concrete spacers?

6. What problems/difficulties are encountered on site in the implementation of the DI based specifications?

7. What effects has implementation of the DI based specifications had on the construction processes used?

CHECKLIST FOR AUDIT OF PREPARATION OF TEST SPECIMEN AND OXYGEN PERMEABILITY INDEX TEST BASED ON DURABILITY INDEX TESTING PROCEDURE MANUAL VERSION 2_ MAY 2010.

Part 1: Preparation of test specimen

Ref.	Question	Yes	No	Notes to the auditor
1	**Test specimen**			
(a)	Was the diameter of the test specimen 70±2mm?			Recorded data
(b)	Was the thickness of the test specimen 30±2mm?			Recorded data
(c)	Does the maximum nominal size of aggregate exceed 26.5mm?			Recorded data
2	**Apparatus**			
(a)	Is the water cooled diamond tipped core attached to a suitable coring drill?			Auditor
(b)	Does the holding device firmly and securely clamp the cubes to ensure they remain in position during coring?			Auditor
3	**Preparation of specimen from cubes (site or laboratory cast)**			
(a)	Were the concrete cubes cast in accordance with the relevant specification?			Recorded data. If this information is not reported by the site, this will not be a finding against the laboratory. However, the laboratory has to report the fact that the site did not make the information available, if the reporting of the non-availability of information is omitted, it will constitute a finding. For laboratory prepared concrete this information has to be recorded by the laboratory.
(b)	Was the minimum dimension of the cast cube at least 100mm?			Recorded data
(c)	Were the cubes cured according to standard or project specification?			Recorded data. If this information is not reported by the site, this will not be a finding against the laboratory. However, the laboratory has to report the fact that the site did not make the information available, if the reporting of the non-

				availability of information is omitted, it will constitute a finding. For laboratory prepared concrete this information has to be recorded by the laboratory.
(d)	Was the duration, method of curing reported?			Recorded data. If this information is not reported by the site, this will not be a finding against the laboratory. However, the laboratory has to report the fact that the site did not make the information available, if the reporting of the non-availability of information is omitted, it will constitute a finding. For laboratory prepared concrete this information has to be recorded by the laboratory.
(e)	Was the age of the concrete at time of coring recorded?			Recorded data
(f)	Were the cubes cored within 28±3 days after casting or as per the project specifications?			Recorded data
(g)	Was the direction of coring perpendicular to casting direction?			Auditor visual check/Record shall indicate visual check by operator
(h)	Was the core barrel placed perpendicular to and at the centre of concrete cube face to be cored, no less than 2mm off-centre in any direction?			Auditor visual check/ Record shall indicate visual check by operator
(i)	Was coring done the entire way through the cube ensuring that far side was reached and the core broke off to an extent that the rough zone created does not exceed more than 5mm from the end of the core?			Auditor visual check/ Record of core condition required
(j)	Are the core sides parallel and within 5° of perpendicular to the face?			Recorded data
(k)	Was the time and date of cutting recorded?			Recorded data
(l)	Was the time of cutting within 3 days of the time of coring?			Recorded data
(m)	Is the first 5mm cut from the cored face of the core and discarded and nothing more?			Auditor visual check of cutting machine/ Record shall indicate visual check of blade vs core positioning by operator
(n)	Were the test specimen marked with the correct reference number on the interior face?			Auditor visual check
(o)	Were specimens damaged during the coring and cutting process discarded?			Auditor visual check/Recorded data of visual check of specimen condition (chipping of surface/cracks) required

(p)	Was the time that the specimen was placed in the oven recorded?			Recorded data
(q)	Was the specimen placed in the oven immediately after cutting?			Recorded data
4	**Preparation of specimen from site elements (test panels or structural element)**			
(a)	Was the duration, method of curing reported?			Recorded data. If this information is not reported by the site, this will not be a finding against the laboratory. However laboratory has to report the fact that the site did not make the information available, if this is omitted, it will be a finding.
(b)	Was age of concrete at time of coring recorded?			Recorded data. If casting date on site has not been made available to the laboratory and age is not known, the laboratory has to report that this information has not been made available and then this will not be a finding against the laboratory. If the laboratory omits to report a reason for not having the information regarding casting date or the availability thereof, the omission will constitute a finding.
(c)	Was coring done between 28 and 35 days after casting or as per the project specifications?			Recorded data. If casting date on site has not been made available to the laboratory and age is not known, this will not be a finding against the laboratory, but the laboratory has to reflect it as such on the report sheet.
(d)	Was the core barrel placed perpendicular to surface of concrete and secured to ensure it does not move?			Visual check by auditor during coring process for case when element was cored in the laboratory. If cores were done on site or not in presence of auditor this can be checked in the laboratory by checking the parallelism of sides and perpendicularity of sides to face. See (e).
(e)	Were the sides of the core parallel and within 5° of			Recorded data. If the coring

	perpendicular to the face?			on site was not done by the test laboratory, and the sides are not parallel within 5 degrees, the laboratory has to report it as a deviation, else it will be a finding.
(f)	Was 35mm nearest to the surface undamaged?			Recorded data. If the coring on site was not done by the test laboratory, and damaged cores are received from site, the test laboratory has to report it as a deviation, else it will be a finding.
(g)	Was each core marked and transported to the laboratory in sealed plastic bags?			Recorded data, If the coring on site was not done by the test laboratory, and unwrapped cores are received, the test laboratory has to report it as a deviation, else it will be a finding.
(h)	Were cores from site elements protected from conditions of adverse drying and damage on site and during transport to a laboratory?			Recorded data, If the coring on site was not done by the test laboratory, and unwrapped cores are received, the test laboratory has to report it as a deviation, else it will be a finding.
(i)	Was specimen from site wrapped in plastic-wrap and transported in containers that protected them from shock, damage and high temperatures?			Recorded data, If the coring on site was not done by the test laboratory, and unwrapped cores are received, the test laboratory has to report it as a deviation, else it will be a finding.
(j)	Was the first 5mm from the exposed face of the core cut and discarded, and nothing more?			Auditor visual check of cutting machine/ Record shall indicate visual check of blade vs core positioning by operator
(k)	Was the time and date of cutting recorded?			Recorded data
(l)	Was the time of cutting within 3 days of the time of coring?			Recorded data
(m)	Were the test specimen marked with the correct reference number on the originally interior face?			Auditor visual check
(n)	Were the specimens damaged during the process of coring and cutting discarded?			Auditor visual check/Recorded data of visual check of specimen condition (chipping of surface/cracks) required
(o)	Was the time that the specimen was placed in the oven recorded?			Recorded data
(p)	Was the specimen placed in the oven immediately after cutting?			Recorded data

Part 2: Oxygen Permeability Index test

Ref.	Question	Yes	No	Notes to the auditor
1.0	**Apparatus**			
1.1 (a)	Oven Is the oven capable of maintaining a temperature of 50± 2°C?			Recorded data
(b)	Is the oven of the forced draft ventilated type?			Auditor
(c)	If the answer to (b) is no, is the relative humidity inside the oven maintained by inclusion of trays of saturated calcium chloride?			Auditor
1.2 (a)	Permeability test apparatus Does the permeability cell have a volume of 5L with a tolerance of not more than ±5%?			Auditor
(b)	Is the room in which test apparatus is kept maintained at a temperature of 23± 2°C?			Recorded data
(c)	Is the air tightness of the equipment tested by using an impermeable test specimen? How often is this testing done?			Recorded data
(d)	During the test for air tightness, was a 0kPa drop in pressure from an initial chamber pressure of 100kPa over a 24 hour period obtained?			Recorded data
1.3 (a)	Compressible rubber collars Does the rubber collar fit tightly around the specimen eliminating any leakage?			Auditor
(b)	Are the rubber collars regularly checked? How often is this done?			Recorded data/Auditor
(c)	Are rubber collars replaced when cracks and tears occur?			Recorded data/Auditor
1.4	Do the gauges or pressure transducers have an accuracy of at least 0.5kPa?			Recorded data (verification/calibration certificate)/Auditor
1.5	Was the supply of oxygen at a standard grade of 99.8%?			Auditor
1.6	Is regulator capable of regulating pressure to at least 120kPa?			Auditor
1.7	Was the vernier caliper capable of reading to 0.02mm?			Recorded data (verification/calibration certificate)/Auditor
1.8 (a)	Desiccator Is the desiccator large enough to hold as many specimens as will be tested simultaneously?			Auditor

(b)	Is the humidity in the desiccator < 60%?			Recorded data
1.9 (a)	Is evidence ,details and records of calibration and verification available with regards to:- Temperature in the oven?			Recorded data
(b)	Temperature in the laboratory?			Recorded data
(c)	Cell volume of permeability test apparatus?			Recorded data
(d)	Cell volume remaining constant in the pressure range of 0 – 120 KPa?			Recorded data
(e)	Accuracy of the gauges or pressure transducers?			Recorded data/Auditor
(f)	Reading of vernier caliper to an accuracy of 0.02mm?			Recorded data/Auditor
(g)	Air tightness of permeability cell by using blank specimen?			Recorded data
2.	**Test specimen**			
(a)	Were four specimens used for each test?			Recorded data
(b)	Were the specimens prepared in accordance with Part 1?			Recorded data
(c)	Was the diameter of the test specimen 70± 2mm?			Recorded data
(d)	Was the thickness of the specimen 30±2mm?			Recorded data
(e)	Are specimens of the same reference marked 1,2,3,4 on the inner face?			Auditor
3.	**Conditioning of specimen**			
(a)	Was the specimen placed in oven at 50± 2° C directly after cutting for 7 days ± 4 hours and no longer or shorter?			Recorded data
(b)	Are the specimen adequately spaced to provide uniform drying?			Auditor
(c)	Were records kept of when the specimens were placed in the oven and when they were removed?			Recorded data
(d)	Was this procedure (a-c) followed with the calibrated laboratory standard ceramic disc and are records as such available?			Recorded data
4.	**Testing of specimen**			
(a) i	Were the specimens placed in the			Recorded data

	desiccators immediately after removing from the oven?			
(a) ii	Was the time when the specimens were taken out of the oven and placed into the desiccators recorded?			Recorded data
(a) iii	Was the temperature in the desiccators maintained at 23±2°C whilst the specimens were being cooled in the desiccators?			Recorded data – adequate if laboratory area in which the desiccators are housed's temperature is maintained and recorded as 23±2°C
(b)	Was the time when the specimens were removed from the desiccators recorded?			Recorded data
(c)	Was the cooling period not less than 2 hours and not more than 4 hours?			Recorded data
(d)	Was the diameter of each specimen measured with the vernier calliper at four points equally spaced around the perimeter of the specimen?			Auditor
(e)	Was each reading of the diameter measurement recorded?			Recorded data
(f)	Is the average of the diameter measurements obtained and recorded to the nearest 0.02mm?			Recorded data
(g)	Was the thickness of each specimen measured with the vernier calliper at four points equally spaced around the perimeter of the specimen?			Auditor
(h)	Was each reading of the thickness measurement recorded?			Recorded data
(i)	Was the average of the thickness measurements obtained and recorded to the nearest 0.02mm?			Recorded data
(j)	When placing the specimen in the compressible collar within the rigid sleeve is the test face (outer face) placed at the bottom?			Auditor
(k)	Are any gaps visible between the sides of the test specimen and the collar?			Auditor
(l)	Was the specimen placed so that the outer face rests against the lip of the collar?			Auditor
(m)	Was the specimen, collar and rigid sleeve placed on top of the test chamber so that it covered the hole?			Auditor
(n)	Where the solid ring is placed on top of the collar is a check made to ensure that no gaps are visible between collar and sleeve?			Auditor
(o)	Is the cover plate placed on top of the solid ring?			Auditor
(p)	Was the top screw partially tightened to			Auditor

	ensure that the cover plate was centered?			
(q)	For a specimen that has been centered was the apparatus tightened – first to finger tightness then one and a half revolutions with a spanner?			Auditor
(r)(i)	Was oxygen allowed to flow through the permeameter for 5 seconds so as to purge test chamber of gases other than oxygen?			Auditor
(ii)	Was the pressure between 100 and 120 kPa during the 5 seconds?			Auditor
(s)(i)	At the start of the test was pressure in the cell maintained at 100 ± 5 kPa?			Recorded data Auditor
	Was the outlet valve closed and the valves checked for leaks?			Auditor
(t)	Was the initial time t_0 of the test recorded to the nearest minute at the start of the test immediately after pressure was brought up to 100±5 kPa after closing of the outlet valve?			Auditor
(u)	Was t_0 within 30 minutes after removing specimen from the desiccators?			Recorded data
(v)	Was the initial pressure recorded as P_o to the nearest 0.5kPa at time t_o?			Recorded data Auditor
(w)	Was the first reading after P_o and t_o taken within 5 minutes after t_o?			Recorded data Auditor
(x) (i) (ii) (iii) (iv) (v)	Check for leaks and corrective action. After 5 minutes of starting the test does the pressure drop quickly, > 5 kPa per minute, and if so, was the test stopped and the cell checked for leaks? Was the sample checked if it fits tightly in the collar; and if so was the leak or sample fit immediately corrected and test restarted? If the leak could not be corrected was the test aborted and the specimen tested in another cell? Was the time it took to take the corrective action recorded? Was the specimen placed back in the desiccator and retested within the 2 – 4 hour time slot during the corrective action procedure?			Auditor
(y)	Were the readings thereafter taken with			Recorded data Auditor

	sufficient frequency such that pressure had not dropped by more than 5±1kPa between readings?			
(z)	Was the test terminated when the pressure had dropped to at least 50±2.5kPa or after 6 hours±15 minutes, whichever occurred first?			Recorded data Auditor
(za)	Was a minimum of 8 readings taken?			Recorded data
(zb)(i) (ii)	Was the test procedure that is referred to in questions (a-z) above also executed and followed through by using the calibrated laboratory standard ceramic disc and are records of previous such verification processes available for each of the steps (a to z)? Was the individual steps executed in such a manner so as to satisfy each of the individual audit questions (a-z)?			Auditor to be present and view this verification process. Recorded data of previous verifications also required
5.	**Calculation and Reporting**			
(a)(i)	Is a spreadsheet used for the calculations?			Auditor
(ii)	Is the spreadsheet used to record and for the calculations the latest standard spreadsheet provided on the Cement and Concrete Institute website?			Auditor
(iii)	If not was the calculation method checked for correctness, preferably against the spreadsheet provided on the website?			Recorded data
	Is evidence of such a check calculation available?			Auditor
(b) (i)	Does the spreadsheet used record:- The individual test determination of a specimen (k value) recorded to three decimal places?			Auditor Recorded data
(ii)	Oxygen permeability index to two decimal places?			
(iii)	The identification number of the test specimen?			
(iv)	Description of the test specimen with regards to whether they are visible cracks, honeycombing defects or visible bleed paths?			
(c)	Was the above procedure (a-b) also executed for the calibrated laboratory standard ceramic disc and are records of this available			Auditor and Recorded data
(d)(i)	Does the spreadsheet used record:			Recorded data

	The source of specimen?			
(ii)	Location of specimen within cube, core or member?			Recorded data
(iii)	Type of concrete – binder type, water/cement ratio?			Recorded data
(iv)	The curing history?			Recorded data
(v)	Unusual surface treatment such as removal of surface treatment?			Recorded data
(e)	Is the information that is captured on the spreadsheet transferred to a laboratory test report which is supplied to the customer and are records of these reports available.			Recorded data

APPENDIX C

DATA ANALYSIS

The attached CD contains the statistical methods that were used in data analysis. The contents of the CD are:-

1. A brief description of the statistical methods that were used in data analysis.
2. An inventory of data obtained from the different projects.
3. Computed examples of the data analysis carried out and presented in Chapter 4. The analysis consists of the analysis of variance (ANOVA), t-test.
4. The goodness of fit test for the data – OPI values, sorptivity and cover depth.