# CRITIQUE OF DURABILITY SPECIFICATIONS FOR CONCRETE BRIDGES ON NATIONAL ROADS IN SOUTH AFRICA


*By*

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In partial fulfillment of the degree of Master of Science, Faculty of Engineering, University of KwaZulu-Natal

March 2011

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”As the candidate’s Supervisor I agree/ do not agree to the submission of this thesis”.

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## Acknowledgements


- Stefan Kaffka, Dennis Brown, Harold Horsley and Philip Wyatt, the Engineers Representatives at each of the SANRAL construction sites, who allowed access to the projects and assisted with the testing requirements and logging of the test results.

- Jim Horton from Contest Concrete Services, for guidance and commercial testing in the laboratory

- Professor Philip Everitt for his guidance and supervision

- SANRAL, for permission to undertake additional in-situ testing at the various sites

- My wife and children for their love and support

# ABSTRACT

Damage to reinforced concrete bridges due to carbonation and chloride induced corrosion is widespread in South Africa and prone in environments where carbon dioxide is at high levels as well as in marine environments where chlorides are present. Performance specifications are therefore essential in order that structural concrete can be designed and constructed to the required standards ensuring that the long term durability can be maintained. This dissertation includes a review of SANRAL's current durability specifications. The specifications are critiqued in terms of the testing methodology followed as well as strength and environmental exposure considerations, and recommendations are made for improving the specifications.

The literature review, outlines the background to both carbonation and chloride induced corrosion to reinforced concrete bridges , considering the fundamental causes of deterioration of concrete caused by carbonation and chloride ingress and repair costs during their service life. The South African Durability Index tests are presented and reviewed, in particular the laboratory testing apparatus and procedures. In addition, the index tests are compared with durability test methods currently being used internationally.

The background and previous durability specifications used in South Africa on road bridges as well as details of research into specifications to ensure durable concrete with specific emphasis on curing of concrete is summarised. The indications are that performance based specifications for concrete on bridge structures internationally follow similar criteria to the specifications currently being adopted by SANRAL. Both performance and prescriptive specifications used usually depend on the risk that a constructor needs to carry. Importantly both cement extenders to ensure long term durability and penalties are applied in performance based durability.

SANRAL's current durability specifications are reviewed and both the negatives and positives are presented for the various sections. Amendments to the Committee of Land Transport Officials (COLTO) standard specifications are recommended address shortcomings. The latest project specifications used on SANRAL contracts incorporating target requirements for cover and oxygen permeability are evaluated.

These impose penalties if targets are not achieved, while limits are placed on chloride conductivity values for various blended binders. Data is also included for the sorptivity index values on the five projects which may analysed and target values can be set and implemented in future.

Descriptions of the five projects with regard to durability specifications, their environmental exposure condition and concrete mix designs are presented. Five projects in KwaZulu-Natal, are used as case studies for durability tests and specifications. The only distinct difference in the specifications is that the three projects commencing in 2006 and early in 2007 had the target values for water sorptivity whereas for the project, sorptivity values are only reported on.

Durability index testing results at each of the sites from the trial panels, additional test cubes (cast for coring and testing of durability indexes) as well as coring and testing from the bridge structures are presented. A major change is coring and testing of samples from trial panels and additional test cubes on the site instead of coring of the structure. The information is drawn together and relationships are determined between the various durability indexes as well as to strength. It is evident that the quality of concrete as constructed in the structure which is reflected by the durability index results is different to that produced in the test cubes and trial panels.

It is deduced that while more care is being taken to produce quality concrete on the sites, certain aspects of the specifications need revision in order to remove confusion as well as to ensure that the concrete in the structure meets the target requirements.

Finally it is noted that climate change is having an impact on design of bridge infrastructure, and while the surveys undertaken at Ethekwini and Msunduzi Municipalities shows that carbon dioxide levels being recorded are still average levels, worldwide there has been an increase in $CO_2$ levels and further modifications to specifications in future may be required.

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# 1 NEED FOR RESEARCH

## 1.1 Background

Performance based durability specifications have been in use in South Africa since the late 1990's, and there have been many advances made to further understand the durability criteria required and testing involved to ensure that concrete produced performs to the required service conditions. This was mainly as a result of the research work undertaken by both the University's of Cape Town and Witwatersrand, where monographs were produced to test and classify quality of concrete according to three durability index criteria i.e. water sorptivity, oxygen permeability and chloride conductivity.

The South African National Roads Agency Limited (SANRAL) has since 2001 implemented performance based durability specifications on all of its construction contracts where structural concrete is being used. It was one of the first public sector clients to adopt such specifications, which have seen the standard of workmanship in producing quality concrete increase on its construction sites.

The aim of this research is to assess the currently adopted specifications and durability testing criteria and to determine the variability of durability test results by testing cores extracted from trial panels and additional test cubes cast and cured on the various sites (which are meant to simulate the as-built structure) with cores extracted and tested from the in-situ concrete. The relationship (if any) of compressive strength to certain of the durability index tests was also be verified. Testing of additional cubes and trial panels in this way limits coring on the structure which if done, results in points of entry of moisture carrying chlorides and carbon dioxide which could result in premature failure of concrete. Access for coring of the structure also presents a problem. A key finding will be whether the durability results from the trial panels and cubes simulate the material properties of the as-built structure. to ensure that the as-built structure has been constructed to the correct specifications and a high quality that ensures that its long term durability performance is not compromised.

## 1.2 Whole life cycle costs

Previously, SANRAL specifications addressed only minimum binder content and maximum water/binder ratios. This was insufficient to ensure durable concrete. As a result, many old structures have been failing prematurely well before their design life due to the limited effort that was placed on durability during the design and construction phases of projects.

The costs to repair minor concrete spalls and cracks on busy freeways such as the Ben Schoeman Freeway between Johannesburg and Pretoria are high mainly due to access required for these repairs. When comparing future costs (this includes repair, access and road user delay costs) with initial costs to ensure durable concrete during construction, the initial costs are much lower than future costs, and therefore it makes economic sense to ensure durability is paid for upfront during the initial construction.

In addition, SANRAL considers coring of bridge deck edges to test for durability over freeways and rivers to be expensive due to accessibility as well as creating weak points on the structure for ingress of moisture, chlorides and carbon dioxide, which is the main reason for the durability clause amendments of the specifications. This will therefore make the current specifications even more economical.

## 1.3 SANRAL specification

The SANRAL durability specification has evolved over the last few years. Prior to 2001, it was based only on a minimum binder content in a mix as well as maximum water binder ratios. Subsequent to the concrete industry being introduced to the durability index approach, SANRAL adopted the limits initially specified from the research monographs. There were subsequent amendments to the specifications to keep pace with the ongoing research as well as experience on practical aspects from construction sites.

## 1.4 Objectives of research

As durability specifications are an important aspect in terms of design and construction to ensure the structure is capable of lasting its design life, testing to ensure that the in-situ concrete of a structure has the necessary material properties to ensure long term durability is important. SANRAL has been involved with concrete durability nationally, and has adopted performance based durability specifications. In addition, research currently suggests that the performance of the placed concrete can be tested if cores are extracted from the structure and tested to check that it meets the required durability parameter. In this way, i.e. in-situ coring, the effects of curing, compaction and exposure of the structure to various environmental conditions can be checked against the results of the cores extracted. The hypothesis of this research was that coring of trial panels and/or test cubes cured on site will replicate results from cores drilled from the structure and therefore can be used to predict durability. The coring of trial panels cast on site as well as of test cubes cured on site and in the laboratory was a simple procedure to implement and more practical. Specifically the trial panels had to be constructed and cured similar to the structure. The cubes were also be cured on the site and exposed to the same environmental conditions as the structure as an acceptance control criteria during the construction process. As indicated in Section 1.2 above, access is always difficult and costly.

The dissertation therefore aims to test this hypothesis.

In addition, the current performance based specifications adopted by SANRAL; specifically the durability requirements were reviewed and commented on. The effects of a confined space and controlled curing environment of cores extracted from test cubes and trial panels were investigated and reported on as this was crucial in the durability test results having values different (if any) from the in-situ concrete, which was cured and placed differently from the cubes and panels. The objectives are listed below:

- Survey literature regarding corrosion to reinforced concrete, durability testing criteria and specifications for concrete durability, specifically performance testing;

- Review SANRAL's current specifications;
- Investigate limitations of testing cubes and trial panels for durability;
- Investigate the durability testing on five SANRAL contracts in terms of
    - ◊ Testing of trial panels and cubes
    - ◊ Testing of the in-situ concrete
    - ◊ Limitations;
- Compare the results specific to each of the durability criteria for each of the contracts, and comment on specific relationships between the various durability indexes as well as relationships with compressive strength;
- Make conclusions in terms of SANRAL's current specifications; and
- Give recommendations for improving SANRAL's current specifications

## 1.5 Scope and limitation

The research used four new construction contracts to assess current specifications requirements for both inland and coastal structures, and there was naturally some generalization made when applying results from the sample to general practice. The similarity of the type of construction practice used on the contracts entabled such a comparison to be made. There was however uniqueness for each of the contracts but the comparative results obtained for the inland and coastal type contracts was valuable for future specifications.

By the time the research was completed, three of the five projects were fully completed.

## 1.6 Dissertation Overview and Layout

The dissertation commences with a review of the relevant background on concrete durability and specifications used previously to what is currently being used, and limitations thereof. Durability testing methods and procedures are also discussed.

Chapter 3 outlines the methods of investigation that were used to obtain the data from the initial design mixes to the final coring and testing of the as-built concrete structural elements of the bridges.

Chapter 4 presents a review in more detail of SANRAL's contract documentation, particularly focusing on the durability aspects.

Chapter 5 provides a description of the projects where testing was be undertaken as well as provide details of the specifications and mix designs for each of the contracts.

In Chapter 6, the limitations of testing cubes and trial panels for durability are discussed.

In chapter 7, the results of the various tests are provided for each of the phases of the various projects in the form of tables, figures and graphs.

In chapter 8 the information of the various sites are drawn together and discussions provided for various relationships that will be drawn from the test results. Conclusions are finally drawn and recommendations are made to improve SANRAL's specifications.

# 2 LITERATURE SURVEY

## 2.1 Corrosion of reinforced concrete road bridges

### 2.1.1 Introduction

While many concrete road bridges are designed for at least a 100 year design life, they do fail prematurely as a result of ingress of certain gases and ions causing reactions within the concrete and steel interface leading to cracking and spalling of concrete. While these premature failures occur they do not always render a bridge structure unsafe but need to be repaired depending on the environment the structure is located. They are also repaired in the interest of the public to ensure that they have faith in the road authority owning the structure.

### 2.1.2 Mechanism of Corrosion

Reinforcing steel that is present in fresh concrete is protected from corrosion. A passive oxide film forms on the surface of the steel as a result of the initial corrosion reaction. Concrete in its fresh state develops a high alkalinity as a result of the initial hydration process in cement. As a result of the presence of oxygen, there is stabilization of the film on the surface of the steel embedded in the concrete, which ensures a continuous protection and the high alkalinity of concrete is retained. The presence of three chemical compounds viz. calcium hydroxide, sodium hydroxide and potassium hydroxide in concrete results in it exhibiting a PH above 12.The reinforcement may corrode upon depassivation of the passive layer due to a reduction in the alkalinity of the concrete where the pH drops to below 8, mainly due to the ingress of carbon dioxide (carbonation) and aggressive ions such as chlorides and sulphates (Raath B and Horten J, 2006). Once depassivation of the ferric oxide layer takes place, the reinforcement may corrode provided that sufficient oxygen and moisture is present. Figure 2.1 below shows the mechanism of deterioration of a reinforced concrete element. A durable concrete must therefore be able to resist the movement of chloride ions and carbon dioxide from the exposed exterior surface into the internal area of the concrete (Hoppe, Mackechnie and Alexander, 1994).

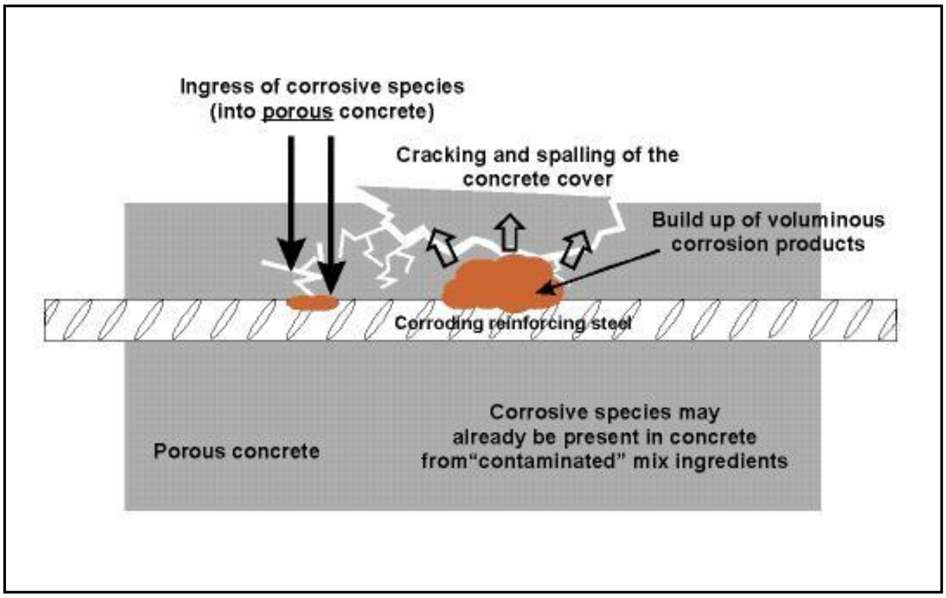



**Figure 2.1 : Mechanism of deterioration of reinforced concrete (Corrosion-Club.com, 2002)**

### 2.1.3 Corrosion damage in reinforced concrete bridges

By nature of the reaction of the aggressive ions and carbon dioxide with concrete and reinforcement, large internal pressures are generated at the interface between the cover concrete and the reinforcement. In general, exposed faces of bridge elements have concrete cover generally in the range between 40 to 60mm. Therefore contaminants can easily reach the level of the reinforcement through the porous concrete, resulting in cracking of the concrete, minor spalling or in more serious cases even large sections of delamination of the concrete from the bridge element.

The following photos below in Figures 2-2 to 2-4 show extent of the damage that can be caused to concrete bridges as a result of the corrosion process. Figure 2-2 shows severe cracking and spalling to a bridge pier, while Figure 2-3 shows large portions of concrete spalling from a bridge deck. Figure 2-4 shows deterioration to parapets of a bridge deck. In South Africa, we are however fortunate that bridges are in relatively good condition when compared to northern hemisphere countries like the UK, the US and Canada. In those countries, deicing salts are one of the major reasons for the premature failure of concrete bridges as well as freeze-thaw attack.

**Figure 2.2- Severe cracking and spalling of a bridge pier (Source- Corrosion-Club.com, 2002)**

**Figure 2.3- Large sections of concrete spalled in a bridge deck soffit (Source-Author)**

**Figure 2.4– Cracking and spalling of a bridge parapet (Source-Author)**

Cracked and spalled concrete bridges are an indication that failure has occurred prematurely under serviceability conditions. While the defect may not represent a direct danger in terms of ultimate failure at the particular position, it could lead to danger to the general public using the particular bridge e.g. spalled concrete falling onto a person or vehicle traveling on a road could lead to serious injury and claims brought against a bridge authority. In certain cases authorities could redirect huge sums of their capital budgets to maintenance due to premature failure of concrete. Between 2001 and 2004 the South African National Roads Agency (SANRAL) spent between R250 to R300 million on rehabilitation and repair of bridges. While majority of the bridges were old (greater than 25 years), there was a high proportion at that time recently constructed.

## 2.2 Specifications

### 2.2.1 Introduction

Specifying requirements to ensure the long term durability of reinforced concrete structures is not a new criterion. In the past in South Africa (prior to the late 1990's), means of specifying criteria to ensure durability were primarily based on content of binders and water binder ratios to be used in the mixes. The question always being posed is why the need to change from the previous recipe specification. Reasons for this is that :

- the environment has become more aggressive,
- cement manufacture has become much faster and greater choice of finer cement blends,
- choice of fine aggregates is becoming increasing limited,
- levels of carbon dioxide are continuously increasing , and
- construction is becoming increasingly fast tracked.

Therefore with research as well as the concrete industry's drive for better quality concrete e.g. the increased amount of binder blends available, there has been a need for a more stringent and detailed criteria specified using performance based specifications. As increased numbers of new contractors enter the industry, performance based specifications assists them by providing criteria e.g. limits on durability indexes, which ensures the end product achieves a certain requirement. In this way, it leaves the contractor freedom to embark on a method of achieving the end product using material selection and other criteria like good workmanship, curing and compaction (Raath B and Horten J, 2006).

It is always a dilemma for the client or its agent to decide on the level of concrete specification to insert into a contract document. If too little detail is specified, then it allows the contractor the opportunity to cut back on quality to maximize his profits. On the other hand if too much detail is specified, it is difficult for the contractor to construct and could result in the client paying a large premium.

### 2.2.2 Specifications and research on durability in South Africa

There has been little research in South Africa with regard to durability specifications in the last 10 years. The most significant has been the research by Gouws et al (1998), which discussed the use of the durability index as a means of controlling and assessing the quality of concrete on site. Further to this there has been further involvement of Stanish et al (2006) on the assessment and controlling of concrete quality on site using the durability index tests.

### 2.2.3 Previous specifications and research on durability of state road bridges in South Africa

2.2.3.1 Specifications and design codes

Specifications for the construction of all state roads and bridges were governed by the Committee of State Road Authorities (CSRA) prior to 1998. Apart from temperature control of concrete delivered to site and methods of curing of the concrete, there were no other criteria specified other than strength that could have had an influence on durability.

However, the experience of good workmanship from experienced concrete foremen and contractors ensured that many state concrete bridges were constructed to a high standard. Examples of this are the major garden route bridges (Bloukrans River Bridge (see Fig 2-5 below), Bobejaans and Groot Brak Bridges) on the national road in the Western Cape that were constructed in the early 1980's.

These bridges have shown little sign of degradation due to environmental exposure, although they are located very close to the sea and highly prone to chloride attack from a saline atmosphere in which they are located.

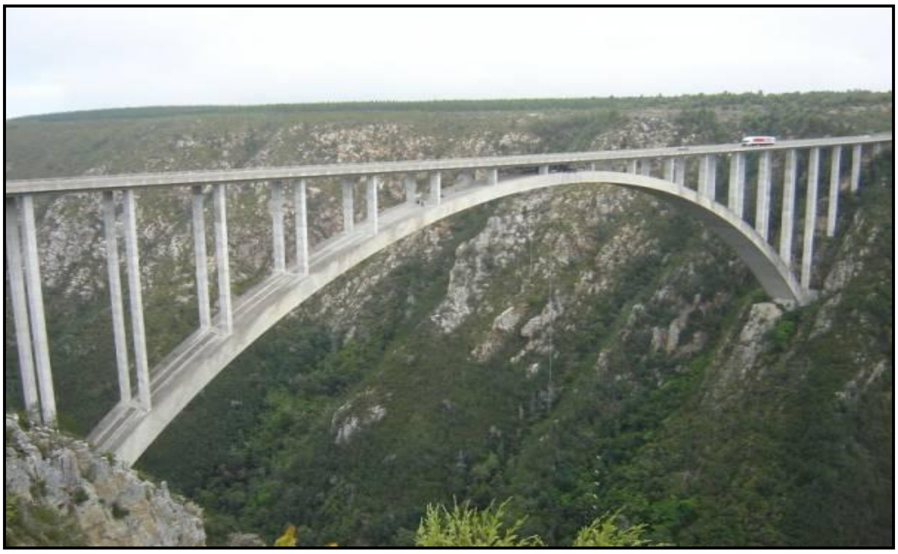


**Figure 2.5: Bloukrans River Arch Bridge on the N2 Garden Route**

Many of the bridge design codes over the decades had requirements in terms of maximum crack widths based on the environmental exposure categories. The majority of South African structural design codes have been based on the British Standards (BS). CP114 (1965) for reinforced concrete which was first issued in 1957 followed thereafter by CP115 (1969-prestressed concrete) and CP116 (1965-precast concrete) and all of these Codes of Practices covered proportioning of mixes. CP114 provided minimum binder contents of between 275 to 489 kg/m$^3$. However, only two environments ('internal' and 'external') were defined, and while different cover to reinforcement was specified for these, there were no references to any other mixes. Only from 1965, did CP116 move towards modern concrete specifications, and defined three internal and six external environments and linked these to both minimum strength grade and cover. As of 1972, CP114, CP115 and CP116 were replaced with a single code, CP110. This built on the CP116 approach and was the basis for many of the current codes in practice. Minimum binder contents of 250 to 360 kg/m$^3$ were specified and linked to minimum strength grades. The adoption of the TMH 7 (1982) codes for bridge design followed mainly the BS8110 code, which in fact adopted majority of the requirements of CP110 (1972).

2.2.3.2 Research

During the late eighties and early nineties, there were many road and bridge contracts undertaken on national roads, and concern was raised both from National Department of Transport and industry on methods of ensuring durable concrete was being produced. This possibly resulted in a report (RR 93/463) produced by Alexander, M.G., Mackechnie J.R. and Hoppe G.E. (1994) titled *"Measures to Ensure Concrete Durability and Effective Curing during Construction"*. It was produced for the Directorate of Transport Economic Analysis of the Department of Transport. Reasoning for undertaking the research was to ensure durability was achieved on the sites either through rigorous supervision or developing tests to accurately measure the degree of durability of concrete. The contribution of good curing to durability and measures to ensure good curing on sites were also investigated. Key findings of the research were as follows:

(a) Available research

The importance of providing adequate curing after casting to ensure long term durability was highlighted, together with the fact that poor curing leads to a porous surface layer allowing easy access of aggressive agents to enter the concrete. Concrete curing practices were investigated locally in South Africa and internationally. It was found that both locally and internationally little attention was given to good curing practice. There was conflicting requirements between the various codes. Both water-added curing and water-retaining curing approaches were discussed. Water-added curing involves application of water through ponding, spraying or saturated covering with Hessian or sand. This generally requires a high level of supervision which is not always available on the sites and may not be practical depending on the element of concrete being cured. Water-retained curing involves placing an impermeable sheet or membrane on the concrete after casting to retain the water inside the concrete. Although this is not as effective as water curing, it is the most feasible, and the most common method of water-retainment is by using curing compounds. There are however limitations of the effective use of curing compounds due to the incorrect method of application as well as the application rates as requirement by the manufacturer.

(b) Effects of curing on concrete properties

Majority of early research was based on the effect curing had on compressive strength mainly due to the emphasis that compressive strength was the most important property of concrete. The effects of curing on the durability related properties of concrete were highlighted such as permeability, sorptivity, carbonation resistance, chloride diffusivity, abrasion resistance and shrinkage. Many of these tests had already been developed by Professors Yunis Ballim and Mark Alexander while further lab and field research is being carried out at both UCT and Wits.

In terms of the types of binders used, it was found that OPC being the most common binder being used was less vulnerable to poor curing. Fly Ash was found to be more sensitive to curing than OPC. In addition, poor curing adversely affected the strength of concrete made with Fly Ash. Slag, another common replacement of cement was also shown to be vulnerable to poor curing, especially when assessing the durability related tests on permeability and sorptivity. At the time of that research, there was little or no work done to check the effects of curing of concrete structures in service and their durability performance. It is however difficult to measure the effect solely of curing, as concrete durability is also influenced by the environment. Other construction processes which can be detrimental to concrete durability are inadequate compaction, over vibration, reduce cover to reinforcement and bad design leading to excessive cracking.

(c) Recommendations to ensure good concrete curing practice

Both prescriptive and performance base specifications were recommended to ensure that adequate curing takes place on sites. For prescriptive specifications, use of curing compounds was found to be most effective when considering research done previously. However, five common methods of curing were provided in a tabular format, as shown in Table 2-1 below.

**Table 2-1: Prescriptive Specifications for Curing (Alexander, Mackechnie & Hoppe ,1994)**

| Type of Curing | Effectiveness | Cost of Curing | Remarks |
|---|---|---|---|
| Ponding of Water | Very effective | Expensive | Difficult to achieve on site (except for slabs) causing disruption to work |
| Plastic/Hessian Sheeting | Fair to poor | Relatively inexpensive | Material must be carefully monitored on site for damage of drying |

| | | | |
|---|---|---|---|
| Intermittent Spraying | Generally ineffective | Moderately expensive | Concrete surface may dry rapidly between spraying applications |
| Forms left in place | Moderately effective | Moderately expensive | Steel forms may allow temperature extremes to damage concrete |
| Curing Compounds | Ineffective to fairly effective | Relatively inexpensive | Application rates and compound used need to be carefully monitored |

With regard to prescriptive specifications, there were no methods of in-situ surface testing that were developed. At that time the sorptivity and oxygen permeability tests were being used for research purposes only. The advantage of performance testing was that the contractor was free to choose a method of curing providing that the concrete met the performance criteria. A selection of durability related tests was provided as shown in Table 2-2 below, with the recommendation that those found to be suitable could be used in later specifications.

**Table 2-2: Performance Specifications for Curing (Alexander, Mackechnie & Hoppe ,1994)**

| **Type of Curing** | **Ease of Use** | **Accuracy** | **Remarks** |
|---|---|---|---|
| In-situ water absorption | Fairly complicated site procedure | Fair to poor accuracy, operator sensitive | Conditioning of in-situ concrete vital |
| In-situ permeability | Fairly complicated site procedure | Moderate to good accuracy dependant on operator and site conditions | Conditioning of in-situ concrete vital |
| Oxygen permeability | Cores extracted on site, fairly simple laboratory test | Very accurate and repeatable test | Concrete preconditioned before test |
| Water Sorptivity | Cores extracted on site, simple laboratory test | Accurate and repeatable test | Concrete preconditioned before test |

| | | | |
|---|---|---|---|
| Humidity Gauges | Gauges placed on concrete after casting, simple procedure | Accuracy of test still to be determined | Concrete curing can be monitored continuously |

It is interesting to note that COLTO specifications after 1994 adopted some of the prescriptive methods given for curing of concrete under Table 2-1. With regard to performance specifications, some clients have now commenced to specify some of the durability index criteria listed in Table 2-2 above.

(d) Methods of defining the potential durability of concrete

Both laboratory and in-situ tests were highlighted in the report which were technically sound and easy to perform. The laboratory tests were oxygen permeability, water sorptivity and chloride conduction. Much of test data was provided as backup to the validity of the laboratory proposed test methods. The in-situ test recommended was the Covercrete Absorption Test (CAT) which measures the rate of water absorption. However it was shown that although results obtained under controlled laboratory environment were reliable, those on site were not.

It was recommended that laboratory tests be used in future specifications and graphs of tentative values of each of the tests varying with water binder ratios were provided. Both acceptance and rejection limits were provided in each of the graphs as shown in Figures 2-6 to 2-8 below.

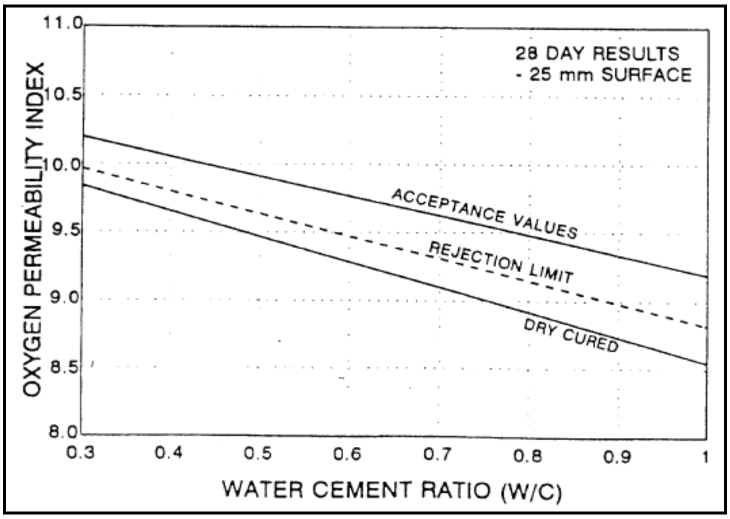



**Figure 2.6: Tentative values for Oxygen Permeability Tests (Alexander, Mackechnie & Hoppe ,1994)**

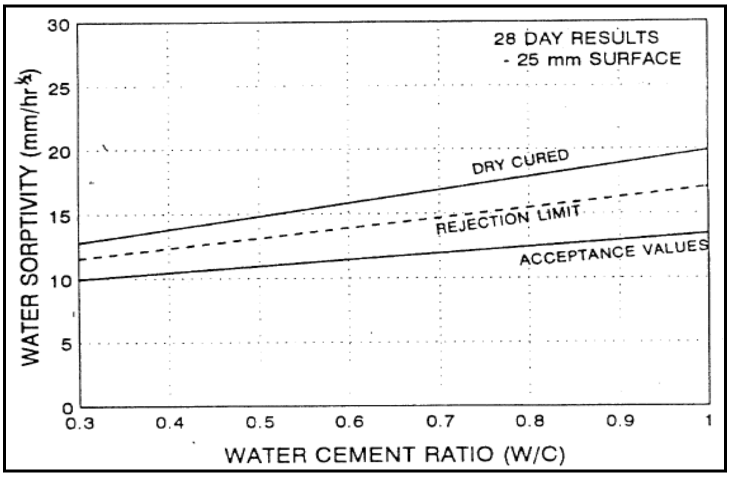



**Figure 2.7 : Tentative values for Water Sorptivity Tests (Alexander, Mackechnie & Hoppe ,1994)**

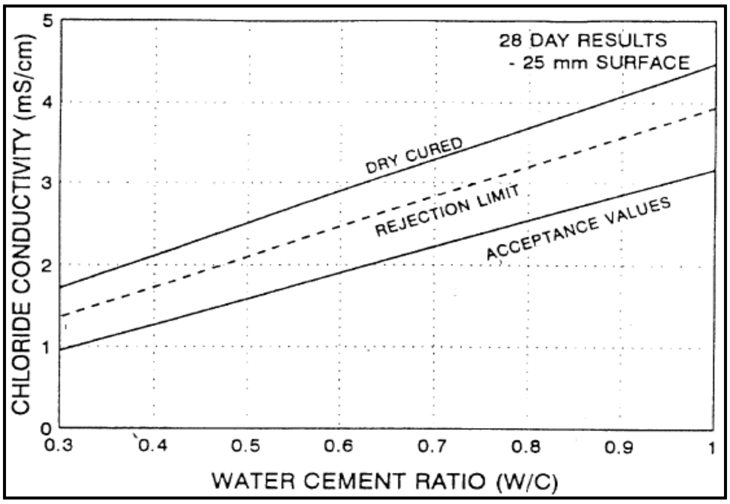



**Figure 2.8: Tentative values for Chloride Conductivity Tests (Alexander, Mackechnie & Hoppe ,1994)**

(e) Recommended Amendments to the standard specifications of CSRA

Recommendations were made for incorporation of these tests to sections 8100 (Testing materials and workmanship) and 8200 (Quality Control) of the standard specifications to CSRA (Committee of State Road Authorities) as part of the conclusions of the 1994 study (Alexander, M.G., Mackechnie J.R. and Hoppe G.E. (1994).

As curing is crucial to ensure long term durability, it was recommended that curing be removed from the rate make up of concrete and be paid for separately. This ensures that the contractor has allowed in the tender price a separate sum of money for the effective curing of concrete. It was recommended that the rate be not less than 5% of the concrete rate under section 6400 for that concrete item. Recommendations were also made to section 6400 regarding curing and protection of concrete. These recommendations are still believed to be useful in ensuring that the long term durability can be maintained.

### 2.2.4 Current specifications for national road bridges in South Africa

In the 1998 specification, which was the Committee of Land Transport Officials (COLTO) publication, and which is the present standard being used on all state roads, there was a shift in thinking with regard to concrete durability.

The following section was inserted under clause 6404(b):
"Where for reasons of durability or other considerations concrete is designated by the prefix "W", e.g. class W30/19, such designations shall denote concrete having a binder content not less than and a water: binder ratio not exceeding the limits specified in the project specifications.
In such cases, characteristic strength of the mix shall be based on the higher of the following values:

(i) the specified 28 day characteristic cube compressive strength, or
(ii) a characteristic cube compressive strength corresponding to the designated maximum water: binder ratio, or
(iii) a characteristic cube compressive strength corresponding to the designated binder content".

There have therefore been many projects since 1998, where under the project specifications, limits were provided for the minimum binder contents (typically in the range of between 400 to 420kg/m$^3$) and maximum water binder ratio (typically 2,37). There was no reasoning on how these limits were arrived at, or on the type of cementitious extenders that could be used. In addition, there was no differentiation between bridges located on the coast to those located inland with regard to choice of binders to be used. This was a recipe type of specification and too generalized to be used as a national specification. Chapter 4 will discuss the current adoption of national specifications using concrete durability.

### 2.2.5 Specifications used internationally related to durability and testing

With the advancement in research gained internationally in the last decade with regard to concrete durability aspects, specifications have become more focused and owners of structures have adapted their specifications to suit the demands from industry as well as recommendations from national and international research.

While under South African conditions, corrosion is mainly due to ingress of chloride and carbon dioxide into concrete, in North America and Europe additional constraints are freeze thaw cycles and deicing salts where chlorides penetrate into concrete causing premature failure. With inadequate specifications to address many of the United States durability problems, an initiative was commissioned in 2006 by the National Ready Mix Concrete Association (NRMCA) in an effort to change its prescriptive specification to a performance based one. Part of the initiative was to review from around the world specifications of concrete, and a report titled "Preparation of a Performance-Based Specification for Cast in Place Concrete", authored by Bickley, Hooten and Hover (2006) was published. The outcome of this work resulted in a performance based specification guide published in March 2008 titled "Guide to Specifying Concrete Performance", also authored by those referenced above. For many of the countries of which the specifications were reviewed, the performance tests varied between only doing tests on specimens and doing tests on the structure.
A brief summary as highlighted by the report will be given below of the adequacy of each countries specification.

2.2.5.1 Australia
Two grades of concrete are used as specified in AS 1379-1997 (amended in 2000). The first is conventional concrete specified by compressive strength. This is generally produced by most of the plants in Australia. All normal requirements are specified to be achieved by the plant producing such concrete. Special grade concrete is only available at limited locations, and is specified as a prescriptive or performance based. Certain key properties of the mixes like the chloride, sulphate contents and shrinkage properties has to be determined by the supplier. Three AS standards provides for use of certain extenders like fly ash, ground granulated iron blast furnace slag (ggbs) and silica

fume. There are limits placed on blended cements containing fly ash, ggbs and silica fume.

For durability requirements, five exposure classes are specified with requirements placed on strength, resistance to freeze/thaw, cover, chemical content and curing provided for each class. A useful guide in the form of a map is provided which divides Australia into three zones viz. tropical, arid and temperate zones and each differs for different locations. Concrete properties e.g. strength, drying shrinkage, etc are checked on lab samples only. For marine structures, the Concrete Institute of Australia has a recommended practice. Corrosion of reinforcement is the prime cause of deterioration of marine structures in Australia. Performance criteria is based on ASTM 1202 which places limits on sorptivity, volume of permeable voids, permeability and chloride diffusion. For marine conditions, both the design codes of AUSTROADS and New South Wales infer a design life of 100 years and two exposure classes. Each class provides normal concrete prescriptive criteria for strength, binder type and content, maximum water/binder ratio, curing, cover and sorptivity penetration. Another performance specification developed by Ho and Chirgwin (1996), where the sorptivity test is discussed and is used by the New South Wales Roads and Traffic Authority since 1990. Interestingly, a performance test specified for concrete is the sorptivity test. Contractors have to propose a mixture and prove that the target requirements can be achieved before the concrete is placed. Sorptivity limits are specified for four environmental exposure classes.

#### 2.2.5.2 New Zealand

A document viz. CCANZ 2000 “Specifying Concrete for Performance” offers guidance to specification writers. Control of internal and external temperatures, gradients and shrinkage are the main criteria related to durability. For marine environments (tidal and splash zones), fly ash, slag and/or silica fume are recommended. Suppliers take full responsibility to ensure that the concrete meets the required prescriptive criteria.

Environmental exposure classes similar to Australia are also presented in the “Concrete Structures Standard” – DZ 3101. Criteria required for the various classes are similar to that of AUSTROADS discussed in 2.2.5.2 above. For marine conditions, use of extenders is mandatory. Only strength, cover and abrasion resistance (pavements) are tests undertaken on the finished concrete. Guidance is also provided on Alkali-Silica Reaction under publication CCANZ TR3.

2.2.5.3 China

As far back as 2006, the Chinese Code Committee considered revising its specifications and at that time, was reviewing the Norwegian Annexure to EN 206-1 specification for concrete, which many countries have been adopting and adapting to suit each of their environmental conditions. Major issues China has to contend with are freeze-thaw cycles, carbonation, alkali-silica reaction and chloride ingress.

2.2.5.4 Europe (General)

Through the European Committee for standardization standard EN 206-1 was produced, and should be uniformly applied to all European Economic Community (EEC) members. Although the aim is for uniformity through all member states, an annexure can be produced by each state to suit specific issues to that state. Twenty eight countries have currently adopted the Norwegian National Annex –NS-EN-206-1. While a complex list of exposure conditions incorporating a number of possible concrete mixes is provided, the intention of the European approach was to produce concrete designed for specific service life under specific exposure conditions.

In EN-206-1, an introductory discussion is given regarding reasons for following a prescriptive methodology instead of a performance based method and that being the limited experience. Some countries that have developed confidence in performance based test and criteria can use these in the specifications.

A total of six exposure classes are provided with a total of seventeen sub classes. The exposure classes are defined in accordance to exposure to carbonation, chlorides (both with and without sea water), freeze-thaw attack, de-icing agents, and chemical attack. The service life is assumed as 50 years. Alternative performance-related durability design guidance is also provided in the form of an annexure. Prescriptive recommendations in terms of minimum binder contents, maximum water: binder ratio, minimum strength and air content are provided. Use of cement extenders are also provided for. The annexure provides a summary of the philosophy for performance based design.

Eight task groups with representatives from six countries were involved in a project to consider the deterioration of concrete considering a number of possible causes including carbonation, chloride penetration, and freeze-thaw attack (with and without salt). Test procedures using standard tests as shown in Table 2-3 below were evaluated.

**Table 2-3: Test methods used for various concrete deterioration criteria (Bickley, Hooten and Hover, 2006)**

| Criteria | Standard test method | Criteria | Standard test method |
|---|---|---|---|
| 1. Carbonation | (i) Natural carbonation<br>(ii) Accelerated carbonation<br>(iii) CEMBUREAU method<br>(iv) TORRENT method | 2. Reinforcement corrosion | (i) Two-electrode method<br>(ii) WENNER probe<br>(iii) Multi-Ring-Electrode |
| 3. Chloride penetration | (i) Rapid chloride migration method<br>(ii) Chloride profiling method | 4. Freeze-Thaw damage | (i) Capillary suction of water<br>(ii) Capillary suction of de-icing salts |

Three levels of project quality control was established containing standard tests (Levels 1 and 2) and in-situ tests (Level 3). This was produced in a document called – "Duracrete Final Technical Report: Probabilistic Performance based Durability Design of Concrete / Structures, May 2000".

2.2.5.5 France

While EN206-1 has been adopted as a national specification, studies have been undertaken on durability indicators such as porosity, diffusion coefficient (chloride intrusion), permeability (to gas and to liquid water) and calcium hydroxide content. Additional research is being carried out on the chloride diffusion coefficient such that it can be used as a durability index that can be used in predictive modeling. Test procedures have been developed for each of these and five classes of potential durability have been established. All of these test requirements are to be achieved by the concrete supplier before the mix is considered for approval. There are however no quality assurance requirements during the construction phase.

2.2.5.6 United Kingdom

The UK Concrete Standard BS 5328 was withdrawn in December 2003, being replaced by two other Standards, viz. European Standard (BS EN 206-1: Concrete – Part 1) and another British one (BS 8500 Concrete). BS 8500 is retained as a complimentary standard to EN 206-1, and contains two parts viz. Methods of specifying concrete and provides guidance to the specifyers (Part 1), and specification requirements for materials and the concrete (Part2). Two methods of testing are adopted i.e. Conformity testing required from suppliers and Identity testing, which in fact is acceptance testing to check whether a particular batch comes from a conforming batch. The British Standards Institute (BSI) and Quality Scheme for Ready Mix Concrete (QSRMC) issues accredited conformity certificates. Five factors are used in terms of EN206 to select a mixture, based on , on the following:

- Cover and characteristic strength (cube or cylinder strength),
- Intended working life of structure,
- Relevant exposure conditions,
- Relevant exposure class, and
- Possible both physical and constructability properties.

BS 8500 follows exposure classes similar to EN 206 (2001), six exposure classes with 28 sub-classes. The commonly used extenders in blended cements like fly ash, slag and silica fume are specified. The design of concrete mixes using this standard can be complex. There are five classifications to the specification as follows:

“Designed concretes” : These are concretes for particular exposure classifications and defined by limiting targets such as binder type, binder content, maximum water-binder ratio and sulphates/chloride conditions.

“Designated concretes” : Similar to designed concretes except that a 3rd party certificate is required to verify concrete. This type of concrete is generally used for building construction.

“Prescribed concretes” : This is completely prescriptive and used on sites generally with minimum requirements as well as for architectural finish.

“Standardised prescribed concretes” : Low strength mixes used generally for housing projects.

"Proprietary concretes" : These are mixes developed by the suppliers e.g. self compacting concretes which meets stringent criteria for abrasion or impermeability and is regarded as a performance specification.

Since April 2003, the UK Highways Agency has embarked on performance specifications for work on roads under its control. The major issue is the transfer of its risk onto suppliers to produce performance based concrete, and targets to ensure the requirements have been met.

2.2.5.7 Norway

The Norwegian National Annex viz. NS-EN-206-1 (2004) is the national standard which is the EN 206 specification that has been adopted and revised to suit its requirements. A total of eight exposure classes and seventeen subclasses are provided for the various environmental conditions. Prescriptive requirements which are based on past experience and historical data, together with exposure classes are specified. These include maximum water-binder ratio, air content, minimum binder content and types of binder. Only the test for water penetration (sorptivity) is recommended. Past records indicate a high variability of results for the water penetration tests in this country.

2.2.5.8 Italy

The national specification used is UNI EN 206-1 (2004) and has similar exposure classes as the Norwegian standard, together with prescriptive requirements for mixtures. The specification is based on prescriptive requirements similar to many of the European countries. The only cementitious extender allowed in the specification is fly ash. No reasons are given for this.

2.2.5.9 USA

The Federal Highway Administration (FHWA) has instituted a programme since 1991 to convert its current specifications to performance specifications. The plan was to adopt the performance specifications in 2008. It has five expert task groups and a technical working group.

The State of Virginia (VDoT, 2004) has since September 2004 published draft end result specifications which are similar to performance specifications, except that here the suppliers have to provide substantial information of their mix designs for review. Two tests are used for payment for structural concrete which are the compressive strength and the rapid chloride permeability test ASTM C1202, (AASHTO T 277)). The C 1202 test is modified here in that it requires 7 days moist curing at 23°C followed by 21 days at 38°C. This dual temperature curing is required to provide for an increased maturity for mixtures containing cementitious extenders like fly ash and slag that better indicates their long term (3 to 6 months) durability performance. Reduced payments are applied and are based on the percentage of the test results within the specified target, provided that the percentage is greater than 50% of the specified target. Bonuses are due if actual values achieved are better than the target values although only a small percentage and penalties are applied if actual values are not close to the target values and the penalty can be a large value. The bonus and penalty also applies to cover to reinforcement, similar to the current specifications used by SANRAL.

In many of the other states, there is a mix of prescriptive and performance specifications for structural concrete, while performance specifications are being used for concrete pavements.

#### 2.2.5.10 Canada

The code being used is the Canadian Standard (CSA A23.1 and A23.2, 2004). The owner is offered two options to specify concrete (as per Table 5 of CSA A23.1 ) i.e. to specify either performance or prescriptive based specifications for concrete. For each option, criteria are clearly spelt out indicating what the employer should specify and what the contractor and supplier must undertake. Performance based specifications are defined as “when the owner requires the concrete supplier to assume responsibility for performance of the concrete as delivered and the contractor to assume responsibility for the concrete in place”. This clearly indicates that responsibility for performance of the mix stops with the supplier after discharge of the wet concrete from the delivery truck. The contractor carries the risk and responsible for placing, compacting and curing the concrete such that it matures and hardens to have the strength and durable requirements required by the owner.

In terms of environmental exposure classes, five major classes of exposure are given together with a total of fifteen sub classes of exposure. The classes are defined in terms of chloride exposure, freezing and thawing, neither chloride nor freeze/thaw exposure ( i.e. concrete not exposed to atmosphere like footings and internal walls and columns), gas vapour exposure and sulphate exposure. Each of the exposure classes are provided with requirements for water-binder ratios, minimum binder strengths, air contents, curing regime, binder restriction and chloride ion penetration limits. Of the provinces, New Brunswick and Ontario Ministry of Transportation have adopted the requirements of CSA A23.1 for High Performance Concrete's (HPC) in the specifications of the provincial bridges, and uses performance based specifications with bonuses and penalties similar to the State of Virginia in the USA. Cores are drilled from the structures and tested for the required durability criteria.

## 2.3 Concrete Durability

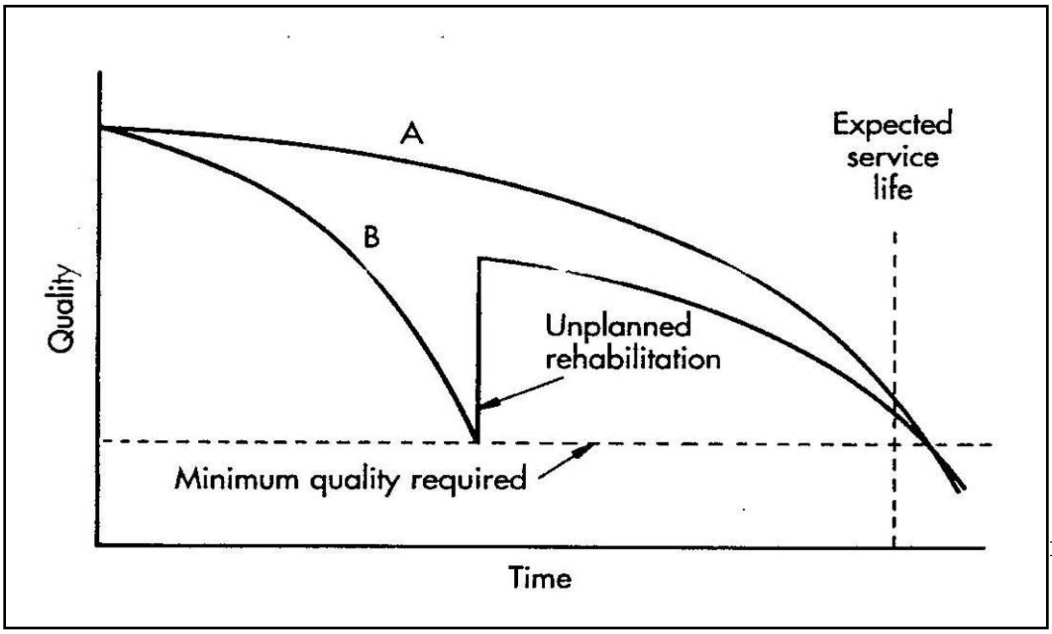
### 2.3.1 Introduction

Concrete has been in existence since the Roman times, and although there are still in existence some of those ancient concrete structures today, many more recent structures made from modern Portland cements have deteriorated due to weathering and corrosion from the environment. It must however be noted that many of the Roman structures were un-reinforced.

The majority of modern day concrete bridges inherently show signs of distress and therefore deemed to have failed as they have not lasted until the end of their design life. Ballim and Basson (2001) define durability as "a material performance concept (rather than an intrinsic material property) associated with the deterioration of the material over the intended service life of the structure in a given environment".

It is important to note that concrete behaves differently when exposed to various environments. This is further illustrated by Figure 2.9 below which shows the deterioration of two structures over their service life. Structure A has been designed and constructed such that it reaches the minimum level of quality after or at its expected service life. On the other hand, structure B has had very little consideration given to durable concrete in the design and construction phases and therefore deteriorates more rapidly than structure A, and requires rehabilitation during its service life. While this structure would have cost less initially, whole life cycle cost could reveal that it will cost more than structure A due to associated costs during the repair.

The high costs of repair as well as the inconvenience placed on authorities on disruption to service are leading owners to demand more from designers and contractors to provide structures that are durable and that lasts its service life.

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### 2.3.2 Need for durability in concrete bridges

Bridges in South Africa have generally been built to a high standard due to good workmanship and materials selection. There is however portions of the bridge stock where severe deterioration has taken place mainly to coastal structures and those exposed to industry pollution. The delayed repair programs of some of the road authorities also results in severe degradation of bridges, and can often lead to the bridge being demolished and reconstructed. Modern day research and technology in concrete durability and testing allows most bridge owners to take advantage of these latest technology and methods and ensure that bridges are designed and constructed to minimize future maintenance costs during its service life. It is an obligation of an authority that uses taxpayers' money in bridge construction to ensure that the latest technology is used e.g. ensuring concrete produced meets latest durability index requirements. Bridge authorities must ensure that concrete bridges have durability built into them for the following reasons:

- It proves economical in terms of whole life cycle costing
- It ensures little to no disruption to traffic during the service life of the bridge e.g. consider closing off a section of the Ben Schoeman Highway between Johannesburg and Pretoria during daytime to undertake repairs to a bridge. The costs to accommodate traffic as well as motorist disruption costs far outweigh the cost of the actual repair.
- It reduces risk associated with a weak structure in terms of third party liability claims e.g. spalled concrete falling onto a vehicle causing injury or death
- It gives credibility and recognition of the authority and will allow other authorities to follow suit, which is good for the country's infrastructure as a whole
- It allows future maintenance budget savings to be spent on other capital works
- It ensures little affect to the environment due to limited use of repair products and from exhaust fumes from traffic congestion during repair contracts, which will be eliminated.

### 2.3.3 Durability problems in concrete bridges

Durability problems of concrete bridges in South Africa are often a result of a multiple of causes associated with the interaction of material, structural and environmental factors. Serviceability failure of bridges may result in a multitude of factors as shown in Figure 2.10 below.

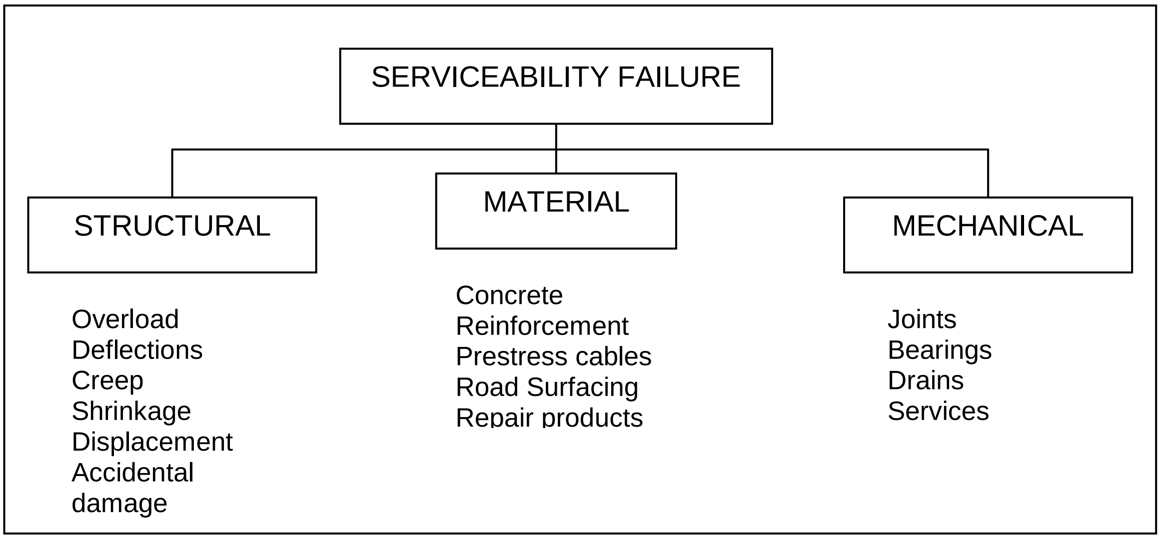


**Figure 2.10: Serviceability failure of concrete bridges (Mackechnie, JR, 1999)**

Durability is primarily concerned with the performance of the concrete to protect the reinforcement steel. Therefore regarding the serviceability failure due to materials failure, majority of bridges that are defective are mainly as a result of carbonation or chloride induced corrosion.

2.3.3.1 Chloride Induced Corrosion

Chloride induced corrosion is primarily a problem in coastal areas due to sea water and air-borne salts affecting the concrete. The high salt concentrations and moisture levels allow rapid diffusion of chloride ions into the concrete. The chloride ions reach the level of the reinforcement and depassivates it. It must also be noted that chlorides could also be introduced into the concrete at mixing stage, either as a contaminant or as a component of an admixture. Chlorides that are present in the concrete are bound in the binder and only after a critical maximum concentration of free chlorides is reached, depassivation of the steel takes place (Mackechnie, J.R. (1999).

The chloride front can reach the reinforcement at fairly deep cover depths with the aid of moisture. In South Africa, there has been severe damage to some of the coastal bridges due to chloride induced corrosion, resulting in either large sections of the bridge requiring replacement, demolition and reconstruction of the bridge or desalination (an electrolytic process of removal of chloride ions from the concrete) This type of corrosion can be so severe that chunks of concrete could spall off bridge elements. Figure 2.11 below shows the deterioration of concrete in a saline environment due to a number of causes from reinforcing steel corrosion, abrasion and chemical attack, temperature gradients, and alkali aggregate reaction. Figure 2.8 indicates the effect of the saline environment on a reinforced concrete member.

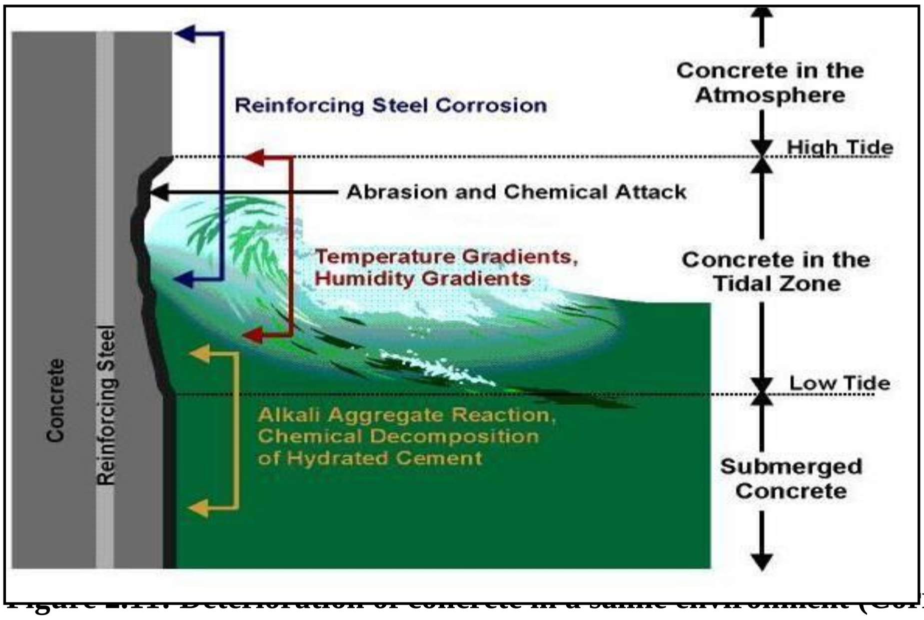



**Figure 2.11: Deterioration of concrete in a saline environment (Corrosion-Club.com, 2002)**


**Figure 2.12: Chloride induced corrosion of a bridge pier in Port Elizabeth (Source, Author)**

2.3.3.2 Carbonation induced corrosion

Carbonation induced corrosion is a process where atmospheric carbon dioxide reacts with the calcium hydroxide in the concrete (present from the hydration process) and can be represented by the following equation:

$$Ca(OH)_2 + CO_2 = CaCO_3 + H_2O$$

This effectively reduces the high alkalinity of the concrete (PH above 12) allowing moisture and oxygen as well as other contaminants to enter the concrete leading to oxidation of the reinforcement. Majority of diagnostic tests undertaken to existing bridges indicate that the carbonation depths are shallow and seldom exceed between 30 to 40mm into the concrete. Elements with reduced cover are therefore prone to corrosion. While increasing cover will eliminate the need for durable concrete, high cover values results in cracking of the concrete due to the limited tensile property of concrete. Slender members also have limited cover requirements.

Corrosion of the reinforcement leads to the formation of expansive oxide products, which exerts large forces onto the surrounding concrete thereby causing cracking and eventual break outs of the concrete. Figure 2.13 indicates the extent of carbonation induced corrosion of a bridge deck.

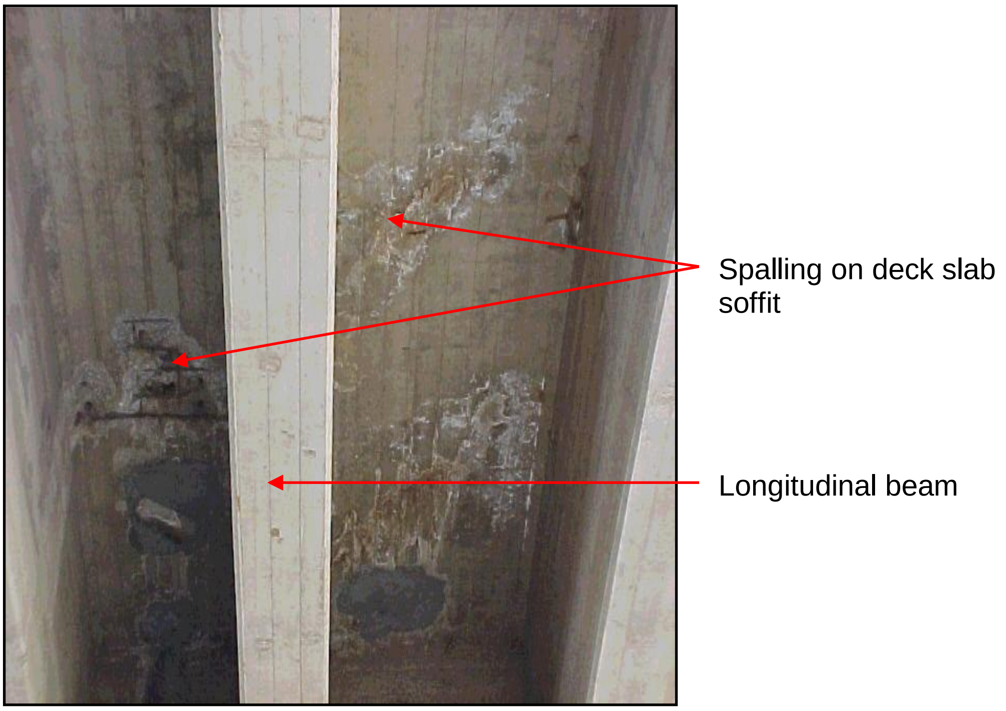



**Figure 2.13: Carbonation induced corrosion of a bridge deck soffit in Gauteng (Source, Author)**

### 2.3.4 The durability index tests

The durability index tests have been described in detail in the research monograph that was produced in 1999 by the University of Cape Town (Alexander, Mackechnie and Ballim, 1999), as well as summarized by Gouws et al (2001). The release of this monograph was a key milestone for many client bodies who then began to incorporate durability specifications into contract documents. The development of these tests has put South Africa in the forefront of the technology development. The technology does not require the use of specialised procedures, chemicals or materials but relies on the measurement of quality workmanship to design, compact and cure concrete to achieve the desired durable concrete. A brief summary of the durability tests are given here for completeness.

2.3.4.1 The oxygen permeability test

The oxygen permeability test involves the use of a falling head permeameter devised by Ballim (1991), and is shown in Figure 2.14 below. It involves oven drying concrete samples at 50°C for 7 days which are 68mm in diameter and 25mm thick (recently revised to 70mm diameter and 30mm thickness due to standard core barrel sizes and to allow for larger aggregate sizes up to 25mm in the mix). These are secured on top of the permeameter cell, which is filled with oxygen to a pressure of 100kpa before being isolated, where after the pressure decay with time (over several hours) is monitored. The Darcy coefficient of permeability, k, is obtained from the slope of the line produced by plotting the log of the ratio of initial pressure to decaying pressure against time.

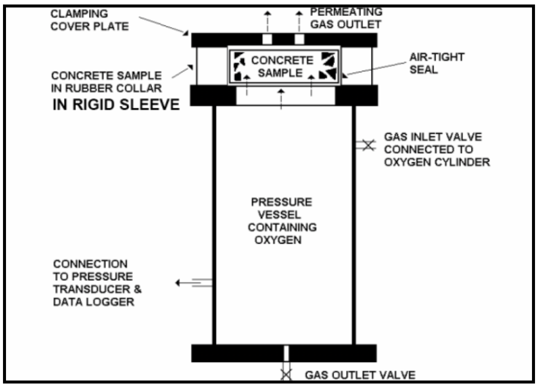


**Figure 2.14: Oxygen permeability apparatus (Ballim, 1991)**

This index is then defined as:

Oxygen permeability index = -log (k)

The oxygen permeability indexes are logarithm values because of being simpler to express and can be expected to be in the range from 8,75 to 11. The higher the value, the less permeable the concrete is. Mackechnie (1996) undertook testing on three grades of concrete, using CEM1, fly ash and slag blended concrete. He concluded that the permeability indexes increased with increased grade of concrete and extent of moist curing. Fly ash and slag was less permeable than CEM1 concrete when well cured and more permeable when dry cured. He stated that oxygen permeability index was more dependent where the most flow will take place and on the amount and continuity of the larger pores or channels in the concrete. This is likely to be caused by poor compaction of the concrete or bleed channels. He further indicated that the test was less sensitive to the finer capillaries and that the oxygen permeability index did not reflect the finer pore structures which are characterized by fly ash and slag concretes.

The results of investigations by Ballim et al (1994) showed that unlike for high strength concrete, the oxygen permeability of low strength concrete was much more sensitive to the length of wet-curing. They further noted that any particular index could be obtained either by extending the duration of low strength concrete curing or by decreasing the water binder ratio in the event that curing was low or ineffective.

2.3.4.2 The water sorptivity test

Sorptivity can be defined as the rate at which fluid is attracted into a porous, unsaturated material under the action of capillary forces. The Kelham's (1988) sorptivity test (modified version) and that of Ballim (1993) was chosen for accuracy and their ease of use. It involves the unidirectional absorption (by sealing edges with epoxy) of water into a single face of pre-conditioned (dried at 50°C to ensure low moisture content), concrete disk sample of 68mm diameter and 25mm thickness, and shown in Figure 2.15 below. This was recently revised to 70mm diameter and 30mm thickness due to standardized core barrel sizes and to allow for larger aggregate sizes up to 25mm in the mix. The sample is weighed at calculated predetermined time intervals in order to determine the mass of water absorbed. This sample is then vacuum saturated with water to determine its mass. The sorptivity is determined from the plot of mass of water absorbed versus square root of time. The index range works opposite to the oxygen permeability index in that the smaller the index value the better the potential durability of the concrete.

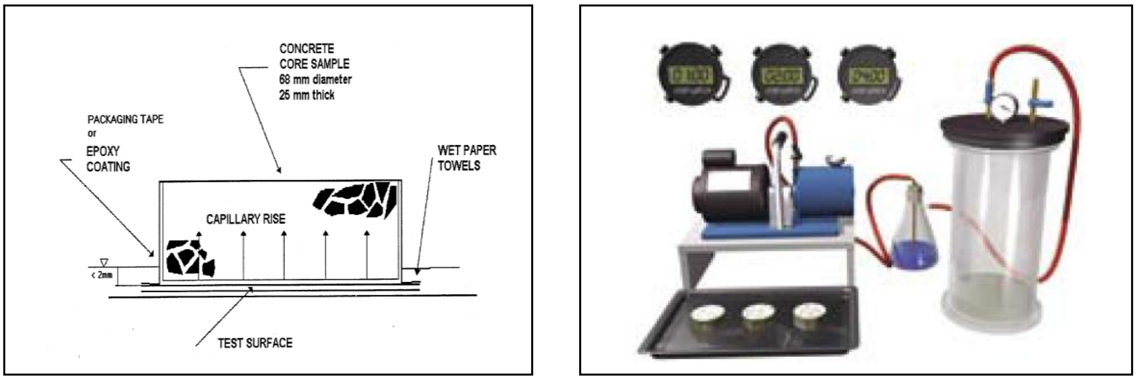



**Figure 2.15: Water sorptivity test (Kelham, 1988 & Ballim, 1993)**

The sorptivity index varies according to the grades of different binders. Mackechnie (1996) performed water sorptivity tests on three different binder grades of CEM1, fly ash and slag concrete. He concluded that absorption rates of concrete reduced with increasing grade of concrete and duration of moist curing. Wet cured concrete produced similar results while dry cured concrete had much higher sorptivity values. He further stated that the test measured a surface property and should be sensitive to the early age drying effects which influence the micro structure of the near surface concrete, and that the test may be used to assess the curing effectiveness on a site. Ballim (1994) stated that the sorptivity test is sensitive to the gradation of concrete quality with depth from the surface, and the test is sensitive to the extent of curing especially during the early age i.e the first seven days after casting. He noted that for moist curing periods longer than 3 days, increasing the strength to above 30MPa had only a small effect on the sorptivity results. Another finding he made was that the sorptivity results reduced with a reduction in water binder ratio of CEM1 concrete and with 28 days of wet curing, the sorptivity of the surface concrete became almost insensitive to changes in normal range of water binder ratio.

2.3.4.3 The chloride conductivity test

Chlorides are able to enter the concrete microstructure in three main ways, namely capillary absorption, permeation and diffusion. Of these diffusion is the primary means of ingress and allows ions to reach the level of the reinforcement steel causing premature failure of the concrete. Chloride diffusion is the process by which chloride enters a concrete substrate through the action of a chloride concentration gradient in a marine environment. In this environment, diffusion of the chloride ions is very important to reinforced concrete. Corrosion of the reinforcement is caused by the depassivating effect of the chloride ions on the embedded steel.. Streicher and Alexander (1995) developed a rapid chloride conductivity test in which almost all ionic flux occurs by the process of conduction to a 10V electrical potential difference between the two faces of a concrete sample. The apparatus, as shown in Figure 2.16 consists of a two cell conduction rig in which the concrete samples (68mm diameter and 25mm thick) are exposed on either side to a 5M NaCl solution and chloride ion migration is due to the potential difference being applied. The cylindrical sample is vacuum saturated with the NaCl solution. Diffusion and conduction are related using Ficks Law.

Chloride ions move through the sample through any pores of sufficient size that are present and therefore the test provides an indication of the diffusivity of the material where the test is sensitive to pore structure and cement chemistry. The lower the chloride conductivity index, means there is an increased potential of the durability of the concrete. Mackechnie (1996) further observed that the 28 day results decreased with increased binder grades (i.e. higher concrete strengths) and affected by the degree of curing and type of binder. Proper curing and use of cement extenders such as fly ash and slag, resulted in a very fine pore structure and the test was found to be extremely sensitive to these changes.

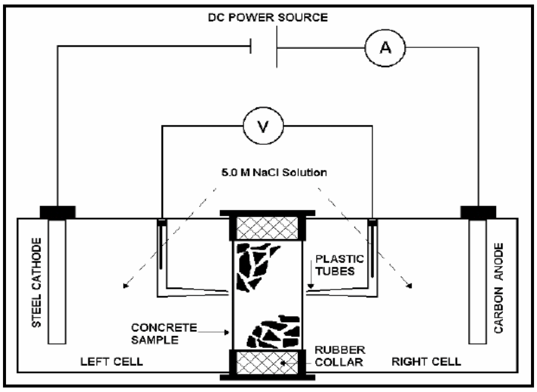



**Figure 2.16: Chloride conductivity test (Streicher and Alexander, 1995)**

### 2.3.5 Comparison of durability tests used in South Africa with those used internationally

Over the last decade there has been major advancement internationally with regard to durability testing of the cover-crete (concrete between the exposed surface and the outer layer of reinforcing steel). South Africa has also been advancing in terms of durability research and testing as a result of the research at both the Universities of Cape Town and Witwatersrand. The output of the research has been shared globally in order that there is progress internationally on concrete durability.

There has been similar research projects carried out internationally on methods of testing for concrete durability. Comparisons on research results, material properties and test methods therefore assist in promoting the development of concrete and specifications. There is a move to standardize specifications for concrete durability and therefore appropriate tests developed by various countries will need to achieve the required criteria. A research project was therefore carried out by researchers from around the world under the auspices of The International Union of Laboratories and Experts in Construction Materials, Systems and Structures (RILEM) under committee RILEM TC 189-NEC. The South African Durability Index test methods were compared with other international non –destructive and destructive tests from both Europe and North America, developed to evaluate the thickness and quality of concrete cover to ensure durable concrete. All three developed test for penetrability i.e. testing for permeation, absorption and conduction. Beushausen and Alexander (2008) who were involved with the testing programme, representing the South African tests have produced well documented results, which will be repeated here to emphasis the acceptability of the South African tests.

The testing involved constructing 6 test panels made with different water/binder ratios, binder types and curing regimes, as indicated in Table 2-4 below:

**Table 2-4: Test conditions investigated in the testing for RILEM TC 189-NEC (Beushausen and Alexander, 2008)**

| Panel No | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| w/b | 0,4 | 0,55 | 0,6 | 0,4 | 0,55 | 0,55 |
| Binder type | OPC | OPC | OPC | OPC/slag | OPC/slag | OPC |
| Moist curing (days) | | | 7 | | | 1 |
| $f_{cu,\ cube}$ @ 28d (MPa) | 62,7 | 48,5 | 34,4 | 52,4 | 38,2 | 42,7 |

Testing for penetrability was then done using non-destructive tests. Cores were extracted from the test panels and sent to several laboratories to perform tests under controlled laboratory conditions as reference tests. The following tests were conducted as shown in Table 2-5 below:

**Table 2-5: Reference tests for RILEM concrete penetrability study**

| Description of test | Test Method |
|---|---|
| Chloride resistance (3 test methods) | NT Built Test, ASTM C1202 test, SA Chloride Conductivity test |
| Oxygen Permeability (3 test methods) | Cembureau method, Torrent Permeability test (TPT), SA OPI test |
| Water Penetrability (2 test methods) | RILEM water absorption test (TC116-PCD), SA Sorptivity test |

All the tests follow a similar philosophy in that they mimic transport mechanisms in concrete samples preconditioned under controlled laboratory conditions. The South African tests were conducted at the University of Cape Town, and all the others done in Lisbon, Portugal.

2.3.5.1 Description of international test methods

(a) Oxygen Permeability

The tests used for oxygen permeability were Cembureau and Torrent.

Under the Cembureau test, a unidirectional gas flow is caused by a constant pressure gradient to a sample 150mm diameter and 50mm thick. This is different to the South African test where there is pressure decay instead of constant pressure and the sample is typically 30mm thick and 70mm in diameter.

Under the Torrent permeability test, the permeability characteristics of concrete can be determined in-situ using the Torrent meter. The equipment has a two-chamber vacuum cell and a regulator that balances the pressure in the inner (measuring) chamber and the outer (guard-ring) chamber. The outer guard-ring prevents air from the surrounding areas from flowing into the pressure measurement chamber. During the test, the cell is placed on the concrete surface and a vacuum is produced with the pump. The rate at which the pressure rises in the inner chamber is recorded and this rate is related to the permeability of the underlying concrete.

(b) Chloride Penetration tests

The tests used for chloride penetration were the Rapid chloride permeability test (ASTM C 1202) and the Bulk diffusion test (NordTest NTBuild). The North American rapid chloride permeability test is in accordance to ASTM C1202-97. A migration cell is used where a water saturated 50mm diameter by 95mm thick sample is placed and to it is applied a 60V DC current for 6 hours. Both cells of the device is each filled with 3% NaCl solution and 0,3M NaOH solution respectively, thus creating a chloride concentration difference between both exposed faces of the sample. The total charge is then determined and the sample given a concrete rating. In this method ionic flux is caused by both diffusion and conduction as opposed the South African chloride conductivity test which is solely based on conduction of chloride ions.

The Scandinavian bulk diffusion test (NordTest NTBuild) involves saturating the concrete samples with limewater, sealing all surfaces except the top surface and submerging into a 2,8M NaCl solution for 35 days. Thereafter 0,5mm of the top surface is ground off for chloride profiling and measuring the chloride at different depths. The diffusion value and surface concentration is then determined using the chloride concentration profile. This test is considered the most essential in its form and is not affected by the implications of using an electric current to accelerate the diffusion process as by the other tests. Due to its time consumption, this test is used rather as a calibration test than a quality control test.

(c) Water Penetration tests

The test used for water absorption was the method used for obtaining the capillary absorption of water of the concrete substrate as suggested by RILEM (RILEM TC116) and involves measuring the unidirectional ingress of water into a preconditioned concrete sample. The test method is undertaken similar to the South African test except for the analysis and reporting. In the RILEM method, the results are expressed in terms of mass of water absorbed over test area and time ($kg/m^2/\sqrt{h}$). In the South African test, the speed of water that is absorbed is measured over time ($mm/\sqrt{h}$). The South African test therefore provides a means to measure the porosity of the concrete.

#### 2.4.5.2 Comparative testing results

The objective of the RILEM study as discussed under 2.4.5 above was to check if the relevant test used to determine penetrability of the covercrete was able to detect changes to water/binder ratio, curing regime and binder type. Statistical analysis was applied to check whether the result of the test method was 'highly significant' (a good indicator), 'significant' (a fair indicator) or 'non-significant' (a poor indicator) level (Romer & Fernandez Luco 2005; Torrent & Fernandez Luco 2007). If the results were in reverse order, the results were deemed to be 'wrong'. The results of the testing are presented in Table 2-6 below.

**Table 2-6: Results of comparative testing, expected penetrability rating and significance of test method (Beushausen and Alexander, 2008)**

| **Transport mechanisms investigated** | **Compared test panel** | **1-2** | **2-3** | **1-3** | **4-5** | **2-6** |
|---|---|---|---|---|---|---|
| | **Variable method** | **w/b OPC** | **w/b OPC** | **w/b OPC** | **w/b OPC/slag** | **Curing** |
| | **Expected penetrability rating** | **2>1** | **3>2** | **3>1** | **5>4** | **6>2** |
| | **Test** | **Differentiation capability (significance)** | | | | |
| Gas permeability | Coefficient of $O_2$ permeability (Cembureau) | ++ | ++ | ++ | ++ | ++ |
| | Coefficient of $O_2$ permeability (South Africa, OPI test) | ++ | ++ | ++ | ++ | ++ |
| | Torrent permeability tester, TPT (Torrent 1992) | ++ | ++ | ++ | o | ++ |
| Chloride ingress | ASTM C1202 – $Cl^-$ electromigration | ++ | ++ | ++ | ++ | ++ |
| | $Cl^-$ electromigration BT-'difusivty' (NT Build 1992) | ++ | ++ | ++ | ++ | ++ |
| | Chloride conductivity (South Africa) | ++ | ++ | ++ | ++ | ++ |
| Water penetrability | Absorption rate and 24hr absorption (RILEM 1999) | ++ | ++ | ++ | ++ | ++ |
| | Water sorptivity (South Africa) | ++ | -- | ++ | + | o |

(++ highly significant, + significant, o non significant, -- wrong)

As indicated in Table 2-6 above, for both the permeability and chloride ingress, the results obtained from the reference tests were very consistent, with only the TPT showing a slight variance for the slag mix. For the water penetrability tests, only the RILEM test was successful in differentiating between the mixes at a highly significant level. By contrast, the South African water sorptivity test failed to achieve the desired results for two of the conditions. The results are not consistent with the experience of the tests as it was carried out using the standard test method.

Therefore all the test methods investigated for permeability and chloride conductivity allow for specifications to be adopted for concrete durability, and demonstrates that the South African tests adopted are successful in evaluating concrete durability characteristics. Further work is required to understand the reasoning for the discrepancy for the sorptivity test. However, with the intensive work carried locally in South Africa regarding reproducibility and repeatability of this test, there were many shortcomings of this test, and may therefore be a difficult measure to adopt as a standard test for performance based specifications for concrete durability currently.

## 2.4 Conclusions

A brief review of the aspects concerning deterioration of concrete bridges has been presented, looking at the fundamental causes of deterioration of concrete caused by carbonation and chloride ingress. In addition, durability testing criteria was reviewed, particular the laboratory testing apparatus and procedures. A RILEM international test program compared various test methods used internationally for concrete penetrability including the three well known South African Durability Index test methods. The results proved the acceptability of both the oxygen permeability index and chloride conductivity index tests. Further work is however still required for acceptability of the water sorptivity test.

A review was undertaken of previous specifications used in South Africa as well as research into specifications to ensure durable concrete with specific emphasis on curing of concrete. There has not been a major focus on durability in past specifications and although research indicated changes to specifications, this was not implemented. A brief review was also undertaken of concrete specifications currently being used in certain of the major countries of the world. The indications are that performance based specifications for concrete on bridge structures are being investigated, researched and adopted in many countries and majority follow similar criteria as the specifications currently being adopted by SANRAL. Many of the European countries have adopted and adapted the Eurocodes to suit their climates. Norway has advance significantly in this aspect, and many European and Asian countries have used the Norwegian code as a basis for their codes. Both performance and prescriptive specifications are used by certain countries depending on the risk that a constructor needs to carry. Importantly both cement extenders to ensure long term durability and penalties are applied in performance based durability. To note however is that South Africa is not prone to Freeze thaw cycles and the effects of de-icing salts on bridges like many of the European and North American countries.

The chapters to follow will review the current SANRAL specifications for concrete durability used on projects where testing was undertaken, as well as destructive and non-destructive testing to be undertaken on certain projects within KwaZulu-Natal. An overall critical evaluation will then be provided of the SANRAL specifications.

# 3 METHODOLOGY FOLLOWED TO TEST HYPOTHESIS

## 3.1 Introduction

This chapter discusses the methodology that was followed to test the hypothesis. As stated previously under section 1.4, the hypothesis of this research was that coring of trial panels and/or test cubes cured on site will replicate results from cores drilled from the structure and therefore can be used to predict the durability of the structure. The methodology serves as a tool by which the four projects can be assessed and quantified, and the results for each can be compared and critically evaluated.

The purpose of this chapter is to present the work activities that were followed and provide the limitations and mechanisms of each activity. The following are the investigations that were used in the methodology:

- Review of Contract Documentation, specifically the project specifications and test procedures
- Observing the results from the non-destructive testing undertaken as follows:
    - ◊ Trial panels for water sorptivity, oxygen permeability and chloride conductivity (on contracts where required)
    - ◊ Wet and air cured test cubes for water sorptivity and oxygen permeability
- A scientific method of modeling and predicting durability of the in-situ concrete from the trial panels and test cubes as follows:
    - ◊ Checking test results of trial panels against the specifications
    - ◊ Comparing the wet cured laboratory cubes and in-situ structure test results
    - ◊ Comparing the air cured site cubes and in-situ structure test results
- Checking the results against actual destructive testing results from the in-situ concrete for water sorptivity, oxygen permeability and chloride

conductivity (on contracts where required), and drawing conclusions in terms of the stated hypothesis.

## 3.2 Review of Contract Documentation and Test Procedures (trial panels, additional cubes, in-situ)

Chapter 4 reviews the project specifications i.e. standard specifications and particular specifications of the projects. With SANRAL being a national organisation, most of the specifications, especially the project specifications contain the same durability requirement on all of its contracts. However, there were amendments made to the recent contracts. The need to differentiate between various environments where structures are located in South Africa appears warranted.

The specifications given in these projects are commented on in terms of suitability and practicality, and likely problems to be encountered.

## 3.3 Non-destructive Testing

### 3.3.1 Objectives

Visits were conducted at all of the contracts that were investigated. The objective of the various visits was twofold. Firstly to gather the practical aspects of undertaking non-destructive testing for concrete durability indexes and the general adoption /acceptance at site level of implementing such a new philosophy, and secondly to ensure that the index testing methodology followed the prescribed requirements. Non destructive durability index testing was undertaken in both the trial panels and test cubes. These investigations form the basis of much of the discussions at the end of the dissertation.

### 3.3.2 Trial panels

SANRAL's specifications involves construction and testing of trial panels for the durability indexes prior to any of the bridge elements being constructed in order to prove that the durability indexes can be achieved with the type of concrete mix that has been designed. The panels are 1m x 1m x 0,15m thick. The trial panels are cast and left on the site adjacent to where the bridge is being constructed for it to be exposed to the same environmental conditions as the bridge. In this way, any effects of the environment will be equally received by both the structure and the trial panels.

### 3.3.2.1 Construction

The trial panels are constructed using the same method of construction as the bridge elements. Therefore for all substructure elements viz. the piers and abutments, the panels are cast vertical using the same type of formwork i.e. either steel or timber forms, and for bridge decks, a horizontal panel is also cast to simulate the large horizontal area of the deck. The concrete for the panels is compacted using vibrators as will be used for the bridge construction. The panel is then left to cure either within the shutters (if this will be done on actual structural elements), or the shutters are removed and either the concrete is kept moist or curing compound is applied. The type of curing to be used must also be followed for construction of the bridge. Figures 3.1 (a) and (b) below shows a typical panel being cast on one of the sites.

**(a)**

**(b)**

**Figure 3.1 a & b : Construction of a vertical trial panel (Source, author)**

**Figure 3.2 : Curing of a horizontally cast trial panel representing a deck top slab (Source, author)**

Figure 3.2 shows the curing of horizontally cast trial panels on a particular project.

3.3.2.2 Core extractions,

Once the concrete reached an age of 28 days, cores were extracted from the panels and tested for the different durability indexes as required of the project specifications. The cores are to be extracted within an area 150mm away from the edges in order that any edge effects from compaction and curing will not influence the results. The cores are then taken to a laboratory which can undertake the required durability index tests as described in section 2.3.4.  Figures 3.3 (a) and (b) shows extraction of cores to horizontal panels.

**(a)**

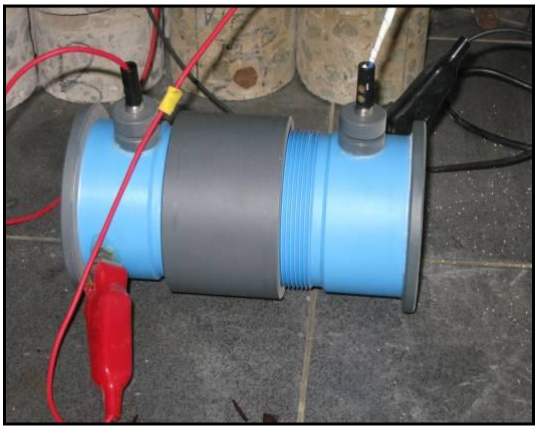
**(b)**

**Figure 3.3 (a) & (b) : Extraction of cores from horizontal cast trial panels (Source, author)**

3.3.2.3 Laboratory testing

The cores were tested in a laboratory using the standard testing procedures as described in section 2.3.4.  In certain instances, the cores extracted from the trial panels are bigger in diameter than the standard size required for each of the tests.  In these cases, the laboratory extracted the required size from the site cores. Figure 3.4 (a) shows a typical 70mm diameter core to be tested. Figures 3.4 (b) to (e) shows the apparatus used to undertake the relevant durability index tests.

**(a)**



**(b)**

**(c)**

**(d)**

**(e)**

**Figure 3.4: (a) Typical disk during sorptivity test, (b) Chloride conductivity cell, (c) Oxygen Permeability rig with sample positioned in collar , (d) Typical Oxygen permeability rig, (e) Collar containing concrete disc ready to be assembled and placed in OPI rig (Source, Contest Concrete Services)**

### 3.3.3 Test Cubes

SANRAL's specifications also require that additional test cubes be taken during concrete casting. Cores are extracted from the additional cubes and tested for the required durability indexes. The 150mm standard test cubes are cast and left to cure on the site adjacent to where the bridge is being constructed for it to be exposed to the same environmental conditions as the bridge. In this way, any effects of the environment will be equally applied to both the structure and the test cubes. Additional test cubes are also cast and cured in the laboratory under standard conditions. This is done so that effects of the environment could be determined on the durability index results, as well as to check if concrete supplied has met the durability requirements. Figure 3.5 shows the casting of cubes on the site for durability testing.

**Figure 3.5 : Casting of test cubes on site for durability testing (Source, author)**

3.3.3.1 Core extractions

Once the test cubes reached an age of 28 days, cores were extracted from both the site exposed cubes as well as the laboratory cured cubes and tested for the different durability indexes as required for by the project specifications. Two cores of 70mm diameter were extracted from each cube.

3.3.3.2 Laboratory testing

The cores were tested in a laboratory using the standard testing procedures as described in section 2.3.4.

### 3.3.4 Limitations

SANRAL's specifications to construct trial panels and additional test cubes and test for the required durability indexes are still evolving. This can be seen from the projects where testing has been undertaken of the different requirements for each. Therefore on some of the projects used for testing under this dissertation, all of the requirements for the trial panels and test cubes were not met in terms of the number of panels and cubes to be provided. It must be noted that the limited size of the panels and cubes may compromise the quality of the concrete in terms of compaction and curing. This will be further discussed in later chapters on the results from the testing.

Of the four projects where testing was undertaken, one was still being completed at time of submission of this dissertation. Full testing will however still be carried out on this project, separate to this dissertation.

## 3.4 Destructive Testing

### 3.4.1 General

In order to test the hypothesis stated previously, destructive testing was undertaken under the dissertation to test whether the in-situ concrete was produced, compacted and cured to the same quality as the trial panels and test cubes. For this statement to hold true the results obtained from both the trial panels and/or test cubes should match closely to the test results from the structure.

Destructive testing was undertaken by extraction and testing of cores from both the substructure and superstructure elements of the bridges under each of the contracts. This was an important aspect of the investigation as the results were used for correlation with the results from the non-destructive testing and relationships (if any) were derived from the results.

### 3.4.2 Method of Testing

Testing of the in-situ concrete incorporated the following aspects:

3.4.2.1 Accessibility

Testing of the in-situ concrete was undertaken by providing access to the substructures and superstructures. Access will always be a problem, and more especially for all substructure and superstructure elements of river bridges, unless it can be done during the dry season when water levels are fairly low. For road and rail bridges, access to the superstructure is a problem due to the continuous stream of vehicles on the road and rail below the superstructures. Access was provided by erection of scaffolding at the required positions where testing was undertaken.

3.4.2.2 Core extraction and sampling

Core extraction and sampling of the in-situ concrete was undertaken using a rotary core drill. Drilling horizontally at elevated heights on platforms constructed from scaffolding is challenging and safety of the laboratory staff is always a concern. For beam type superstructures, core extraction was done in the casting yard once the concrete reached a minimum of 28 days strength, as can be seen from Figure 3.6 below.

**Figure 3.6: Extraction of cores from a precast beam in the casting yard (Source, author)**

For the projects where testing was undertaken, no site laboratories had the equipment set up for durability testing. The commercial lab was called to the site to extract the cores which where close within the Durban and Pietermaritzburg areas. Due to the remoteness of one of the sites, the cores were extracted from the trial panel and structure by the contractor and sent to the commercial laboratory. Care was taken during the transportation not to damage the cores and they were protected from drying out and covered with plastic wrapping.

#### 3.4.2.3 Laboratory testing

The cores were tested in a commercial laboratory using the standard testing procedures as described in section 2.3.4. In certain instances as previously stated, the cores extracted from the in-situ concrete were bigger in diameter than the standard size required for each of the tests. In these cases, the laboratory extracted the required size from the site cores.

#### 3.4.2.4 Limitations

While in-situ durability testing is a key to ensure that bridges have been constructed to the required durability specifications, testing of critical areas like bridge deck soffits and cantilever edges may be difficult due to restricted access. Also, due to bleed water migration, tops of piers are more prone to having increased porosity, and are therefore a critical area to test for oxygen permeability. Some if these areas were however difficult to access and testing therefore could not be undertaken at all of these critical locations under each of the projects. This will be further discussed in later chapters on the results from the testing.

Of the five projects where testing was undertaken, one was still being completed at time of submission of this dissertation. Full in-situ will however still be carried out on those projects, separate to this dissertation

## 3.5 Scientific Method

The scientific method was followed to test the hypothesis. This entailed the following steps as shown in Figure 3.7 below.

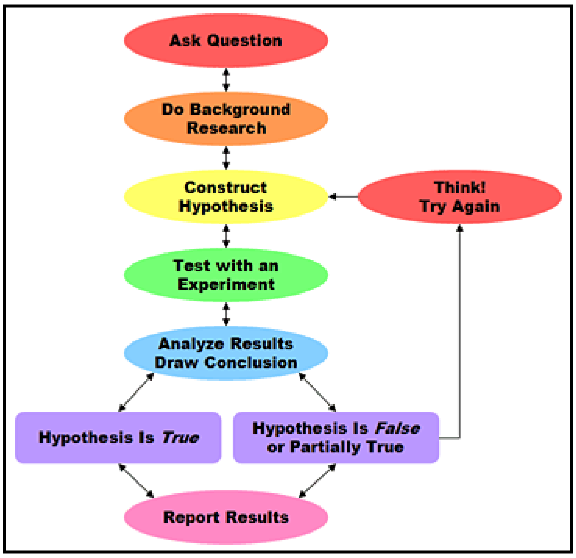



**Figure 3.7: Scientific method followed (www.sciencebuddies.org)**

## 3.6 Discussion

A brief methodology was presented to test the hypothesis that coring of trial panels and/or test cubes cured on site will replicate results from cores drilled from the structure and therefore can be used to predict the durability of the structure.

A review of both destructive testing (by drilling cores from the structure) and non-destructive (using both trial panels and cubes) were provided to show the extent of the testing that was undertaken on the projects.

# 4 CURRENT CONTRACT DOCUMENTATION AND PROJECT SPECIFICATIONS

## 4.1 Introduction

This chapter presents the latest specifications used on SANRAL current construction contracts as well as the specifications used previously for the five contracts where in-situ testing has been carried out. Particular focus will be given to the concrete durability aspects. It must be noted that since SANRAL's durability specification is in the form of a working document and amendments are being made from time to time as new test data evolves from the projects being undertaken. Further research information locally and internationally also has assisted in fine tuning durability index parameters to ensure that certain targets in the specifications are achievable. In addition, research is being undertaken at the Universities of Cape Town and Wits on durability of concrete, and amendments required to ensure that the durability index targets can be achieved are addressed.

## 4.2 Standardised Specifications

The current standard specifications are based on the Committee of Land Transport Officials (COLTO -Green book). As discussed previously under section 2.4.2, there has been very little included into this specification regarding concrete durability. It is not clear why recommendations made through research and practice as was highlighted by Alexander, Mackechnie & Hoppe (1994) was not incorporated into the COLTO specifications, which was published a few years after (in 1998). It was only in 2002 that SANRAL began amending the standard specifications to incorporate additional concrete durability requirements.

In the 1998 standard specifications, comments were included in terms of project specifications for durable "W class" concrete where a minimum binder content and maximum water binder ratio could be specified under the project specification and that the minimum strength requirements shall be governed by either of the above as well as the minimum strength required for structural purposes. Due to there being no further publications of the COLTO standard specifications, any amendments and additions to the standard specifications are being reflected in the project specifications and the key amendments in terms of concrete durability are described below.

## 4.3 Project Specifications

Since 2002, SANRAL has incorporated many amendments and additions to the standard specifications to ensure concrete durability is addressed in both the design and construction phases of a project. Under this section, only certain of the key amendments to the standard specifications are discussed. Certain of the tables from the project specifications are included under Annexure 1. Emphasis has been placed on the key areas that result in low concrete permeability, resulting in penetration of moisture and gases causing premature failure of concrete. These are known as the four C's ( Wilmot, R.E., 2007) as follows:

- **C**oncrete Mix

  Low permeability is a function of the bond between aggregate and the binder, the type of binder, water/binder ratio and size and grading of the aggregates.
- **C**ompaction

  There needs to be adequate and controlled compaction which has an influence on the quality and therefore permeability.
- **C**uring

  Effective site curing is important and leads to good quality concrete, strength and ultimately in impermeability.
- **C**over Depth

  Depth of cover is very important to prevent corrosion of reinforcement. Notwithstanding the requirements of the specifications for cover, often poor detailing and practical aspects on the site leads to changes in cover, or poor fixing details on the site.

In addition, additional durability requirements in terms of concrete temperatures, and durability testing requirements will be discussed. 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).

### 4.3.1 Cover Depth

4.3.1.1 Cover blocks

Cover to reinforcement is crucial in ensuring that the long term durability of the structure is not compromised. It is of no use to design a concrete mix to resist the most severe environmental conditions if little importance is placed in control of cover. It is believed that majority of problem with bridges that have undergone repairs or are in a state of disrepair are due to premature failure of concrete as a result of a lack of cover to the reinforcement. It has been shown that as an example, for external concrete sheltered from rain, 30mm of cover will give 135years of protection to the reinforcement, but 10mm of cover will only give 10 years of life (Shaw, 1994). The method of providing cover to the reinforcement is therefore important to ensure that there is adequate protection. The following paragraph has been included in the specifications:

"Concrete cover blocks shall be made using the same binder and aggregate type as the main concrete with the same water/ binder ratio so that differences in shrinkage, thermal movements and strain are minimised. 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 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"

While it may not be clear in the above insertion, SANRAL insists that only spherical concrete cover blocks shall be permitted. Plastic cover blocks are not recommended due to it having different thermal and elastic modulus values to concrete. This leads to debonding of the interface with concrete and therefore a flow path for moisture carrying chlorides and carbon dioxides attacking the reinforcing steel. The other major incorporation under cover is a reduced payment due to a lack of sufficient cover. Testing is carried out on concrete cover using an electromagnetic cover meter.

4.3.1.2 Cover Requirements and environmental exposure classes

Cover requirements and environmental exposure classes are governed by the amended Table B6301/1 of the SANRAL generic specifications (2008), included under Annexure 1. The conditions of exposure and environmental classes have been amended such that it ties to the recommendations of Stanish, Alexander and Ballim (2006). These environmental conditions and classes of exposure are in the process of being adopted in

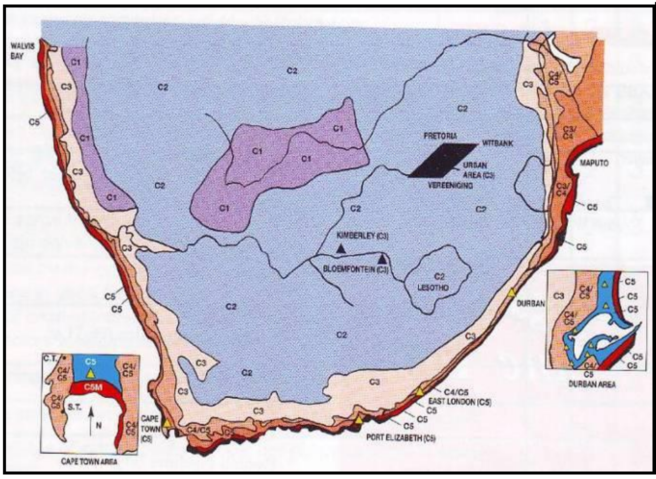
South Africa such that it complies with Eurocode EN206. As extensive descriptions have been given in the table to the various structural members, it was unwise to completely revise the table with the descriptions. Table 4-1 below provides the requirements of EN206.

**Table 4-1: Environmental Exposure classes (Natural environments only) (after EN206-1)**

| **Carbonation-Induced Corrosion** | |
|---|---|
| Designation | Description |
| XC1 | Permanently Dry or Permanently Wet |
| XC2 | Wet, Rarely Dry |
| XC3 | Moderate Humidity (60-80%) Cyclic Wet and Dry |

| **Corrosion Induced by Chlorides from Seawater** | |
|---|---|
| Designation | Description |
| XS1 | Exposed to airborne salt but not in direct contact with seawater |
| XS2a | Permanently submerged |
| XS2b | XS2a + exposed to abrasion |
| XS3a | Tidal, splash and spray zones Buried elements in desert areas exposed to salt spray |
| XS3b | XS3a + exposed to abrasion |

It must be noted that the cover depths provided are greater than that proposed in EN206, and it may be that in future specifications values in Table B6301/1 of the SANRAL generic specifications (2008) may be revised. The only major change was to re-define the "Very Severe" category for members exposed to airborne salts in a saline atmosphere. The previous definition included all structures located within a 30km radius from the coast being prone to chloride attack. However research carried out by the SA Corrosion Institute suggests that this limit is between 1 to 5km from the coast. Figure 4.1 below shows the typical graph produced in the South African Hot Dip Galvanisers Association for corrosion rates in South Africa.




| CATEGORY | DESCRIPTION | LEGEND | SEVERITY OF CORROSION | CORROSION RATE UNCOATED MILD STEEL |
|---|---|---|---|---|
| C5M | USUALLY LESS THAN 50m FROM H.W.M | | SEVERE MARINE | 300µm/YEAR |
| C5 | USUALLY 50m TO 1km FROM H.W.M | | MARINE | 200 - 300 µm/YEAR |
| C4/C5 | 1km TO 40km FROM SEA | | AVERAGE MARINE | 20 - 200 µm/YEAR |
| C3 | 40km TO 100km FROM SEA | | MILDLY MARINE | 10 - 20 µm/YEAR |
| C2 | INLAND | | RURAL | LESS THAN 10µm/YEAR |
| C1 | INLAND | | DESERT | LESS THAN 5µm/YEAR |
| C3 | URBAN ISLAND | | INDUSTRIAL | 10 - 30 µm/YEAR |
| C5M | URBAN COASTAL | | MARINE INDUSTRIAL | 50 - 150 µm/YEAR |
| H.W.M - HIGH WATER MARK | | | | |

**Figure 4.1: Aggressive environments in South Africa (Barnard J, 2007)**

It must however be noted that the map produced in Figure 4.1 is for galvanised sheeting only and not for reinforcing steel whose corrosion rates will be different to the galvanised sheeting. However, it does provide some basis for future research to produce a map of South Africa indicating the various corrosion areas. Another exposure category may still need to be defined for the Karoo region, which is a dry region with little atmospheric moisture and salts present. Durability concerns in this region are a much lesser requirement and the specifications will need to address this. A further amendment to the specification is that exposure conditions for the various structural elements of a bridge are specified instead of an exposure condition for the entire bridge. Groundwater can sometimes contain salts and chlorides in areas inland of the coast and therefore foundations and portions of substructures may be exposed to more severe conditions than the exposed concrete elements. Further examples are where foundations may be subject to chloride attack such as in estuaries, whereas the decks may be only subject to carbonation.

However the minimum cover requirements for the different classes of concrete will need to be revised such that they relate to the cover requirements of the index limits for oxygen permeability, water sorptivity and chloride conductivity. It may be that under the current specifications a high premium is being paid in ensuring that durability indexes are being met but for a reduced cover than that being specified. Further discussion and recommendations are made in Chapter 8 in this regard.

#### 4.3.1.3 Limits for cover

Table B6404/5 of the SANRAL generic specifications (2008) and included under Annexure 1 has been provided for acceptance and rejection limits for concrete cover. It is a requirement on all SANRAL contracts that cover surveys be undertaken to all critical areas i.e. on parapets, deck edges including underside of cantilevers, lower portions of columns, abutments and walls. Should any of these areas showed deficiencies, then SANRAL’s agent may order additional cover tests on other areas at the contractors costs. Reduced payments are applied to reinforcement pay items to those elements which are defective, as discussed further. If the cover is below the specified threshold, then the specimen is rejected.

### 4.3.2 Concrete Mix

4.3.2.1 Binder type

The choice of binder to be used in a structure is based on the environmental exposure class where the structure is located. It is irresponsible to specify ordinary Portland cement (OPC-CEM1) on bridges exposed directly to sea water/ sea or in a chloride environment as the concrete will provide very little resistance to penetration of chlorides. Extenders in binders like fly ash and slag increases the finer particles in the concrete mix and therefore makes it far more impermeable than ordinary CEM1 (Alexander, M.G., Mackechnie J.R. and Ballim, Y. (1999). In harsher environments, it is therefore desirable that extenders be used as replacement to clinker in the binder to prevent ingress of undesirable ions and gases. In addition for low temperatures (less than 20°C) it is not desirable to use binders that have a high percent of extender. This is due to the longer time blended binders require to gain strength, which is not desirable in low temperatures. The following additional clauses have been provided to supplement the standard specifications:

"The type of binder to be used in any concrete element shall take into account the environmental conditions and durability requirements at the location of the site of the works, and shall be as approved by the engineer"

.

Table B6402/1 of the specifications and included under Annexure 1 provides the possible binder types to be used in different conditions of exposure as well for be used under different temperature ranges. The table is provided as a guide to design engineers when they need to assess the contractors design mixes. This table could also be used by the contractor initially in the design of their mixes.

4.3.2.2 Binder content

The most important element and critical component in the concrete is the cement paste that contributes to durability in the hardened state. Materials that make up the cement paste are cement, fine aggregate, water and admixtures. The binder required will depend on two criteria, viz. for strength requirements and for durability requirements either by specifying minimum binder content and/or maximum water binder ratios.

In past specifications, as discussed in Chapter 2, minimum binder contents and maximum water binder ratios were being specified on SANRAL contracts. Normal strength concrete used in bridge super- and sub-structures varies between 30 to 40 MPa, and in general, binder contents for strength vary between 300 and 350 $kg/m^3$. However, in order to meet durability index requirements, the binder content could vary between 350 to 430 $kg/m^3$.

Initially when SANRAL embarked on revisions to the specifications with inclusion of targets for durability indexes, there was resistance from contractors and suppliers, since the specifications were not clear regarding payment for durability concrete. The schedule of quantities only specified strength concrete for the various elements of the bridge structure. Contractors were requesting additional payment for durability concrete. To make it fair to all contractors, revisions were made, such that all contractors can now tender on the same nominal contents, and only when the mix designs are finalised during the project, there are adjustments made on actual binder content required. Some of the ready mix concrete suppliers have been creating their own databases on durability mixes, and can now “tweak” mixes such that they can lower the binder contents but still achieve the required strength and durability requirements. It is unlikely that binderitious contents will vary greater than 450$kg/m^3$ unless high grade concretes are specified for structural requirements and therefore no payment will be made in this regard. For contents lower than 400 $kg/m^3$ it is felt that durability is achievable by “tweaking” the mixes which is to the benefit of ready mix suppliers should they be able to achieve this. A database is being collected on projects where durability concrete is being used, to check limit required that may be set in future specifications.

### 4.3.3 Curing

Curing is a very important aspect in ensuring that the strength, impermeability and long term durability of the concrete can be maintained. Critical for freshly cast concrete gaining strength is retention of moisture and temperature during the hydration process in order that pores are not dried which could result in voids in the concrete matrix, making it permeable and less durable. A small cost is attached to curing during casting of concrete elements yet its long term benefits are huge. It is therefore critical that curing be correctly undertaken. It must also be noted that curing is the last step in the construction process in ensuring that good quality concrete can be produced.

Curing has in the past mostly been poorly carried out on national road bridges. Not much emphasis has been placed on it for the following reasons (Concrete Society of South Africa-1991):

- There have been limited requirements for curing action for different applications in various environments in the standard specifications
- The cost of curing has been included in the payment rate for concrete and no specific payment item has been allowed for it
- No specific training and education has been provided to all levels of personnel involved in the design, construction and management of concrete on projects
- The misconception that cube strength was a sufficient indicator of the durability of concrete
- Fast track construction where concrete is retained in shutters for a limited period only, and stripped and exposed to the atmosphere resulting in drying of the surface of the concrete
- Majority of bridge elements are cured using impermeable curing membranes which are proprietary products that require specific application rates and method of application that are not being adhered to on the construction sites.

The COLTO standard specifications (1998) addresses many of the above concerns and provides sufficient clauses on methods of curing as well as minimum periods of shutter retention for slabs, beams and vertical members. In addition, a range of possible curing methods are also provided. The following additional clause have been included:
"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."
Resin based compounds are not very common in South Africa and are often difficult to remove to undertake repairs if required. In addition, the resin based compound tends to leave a concrete surface that is dark and patchy in appearance.

Research has shown that when stripping shutter to freshly cast concrete, there is a limited period between stripping and applying of a protection coating to the concrete surface in order to maintain the moisture and temperature of the freshly cast concrete. The following additional clause has been provided:

"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".

In-situ bridge deck construction as well as certain concrete elements involves retention of formwork as a means to ensure strength gain and curing can take place. The minimum period specified in Table 6206/1 of the SANRAL generic specifications (2008) shall be complied with in this regard.

The type of formwork plays an important role to ensure that there is no early loss of moisture and temperature from the concrete. While both timber and steel formwork is allowed in the standard specifications, thermal insulation and moisture absorption are certain of the main issues that have to be considered.

The SANRAL specifications have incorporated the use of additional test cubes and trial panels for durability testing, which will be discussed later in this section. In essence, the trial panels are required to be constructed and cured similar to particular vertical and horizontal elements of the bridge structure and later tested for the relevant durability testing criteria.

A major revision to the specifications is the incorporation of curing as a separate pay item. Due to the limited attention being paid by the contractor to curing in the past as well as poor control by supervision staff, the additional pay item will ensure that more attention is paid to this aspect of the construction. This has resulted in more effort on the sites, to ensure curing is undertaken in accordance to the specifications such that payment is made. The employers' representatives on the site are also paying closer attention that the contractor complies with the requirements of the specification and that of the manufacturer where curing compounds are used.

### 4.3.4 Temperature of concrete

Both the temperature of the concrete placed in the element as well as the maintenance of temperature during the hydration process is important to ensure that durability of the concrete is maintained. The issue of the temperature of concrete manufactured or delivered to a construction site has always been a contentious issue. It is a requirement that for all site batched concrete, the temperature of the concrete shall be within the range of 10ºC to 30º C, while for all ready mix concrete, the requirements of SANS 878 2004 shall be complied with. Site staff are required to monitor the temperature of concrete delivered to the site, and if it is not within the required limits, the concrete shall be rejected. An additional pay item is allowed to control the concrete temperature, but only applies where hot weather concreting or large concrete elements are relevant.

### 4.3.5 Durability Design

4.3.5.1 General

All concrete used on SANRAL projects and designed for durability are designated by the prefix 'W'. This differentiation is done so that not all structural concrete is designed and constructed to the same standard, mainly due to costs involved in producing durability concrete. Examples of where concrete does not need to be designed for durability are piles and bases of substructures which are not affected by groundwater containing salts. However, minimum cover needs to be maintained as defined in Table B6301/1 of the SANRAL generic specifications (2008).

#### 4.3.5.2 Previous Design and Testing Requirements

Since introducing requirements that structural concrete meets durability requirements, SANRAL based its past specifications for design and construction on the monographs produced by Wits and UCT in March 1999 (Alexander, M.G., Mackechnie J.R. and Ballim, Y. (1999). A summary table was provided suggesting a range of Oxygen Permeability, Sorptivity and Conductivity values for a range of durability classes, the worst being ‘Very poor’ and the most appropriate being ‘Excellent’. These values were therefore adopted into a set of performance based specifications, and contractors had to achieve all of these values for both design mixes as well as in-situ test results.

Since the publication of the monographs, there has been further research and testing undertaken to refine the suggested index ranges. In addition there has been a lot of interaction with the industry in general as well as that SANRAL is represented on a national working group on concrete durability together with researchers, suppliers, practitioners and specifyers. Further, there has been a lot of objection from suppliers and contractors mainly because of specifications providing durability indexes together with reduced payments, without understanding the background to the indexes, and the sensitivity of index values. The other major issue was the reproducibility and repeatability of the tests, and various laboratories were used for this program. The result was that the Sorptivity test which provided the greatest variability of the results from the laboratories, and should not be used as a performance criteria until such time that further research and testing had taken place.

#### 4.3.5.3 Current Design Requirements

Stanish, Alexander and Ballim (2006) provided a guideline document for specifying durability index limits for reinforced concrete construction. This has been used by the industry, and SANRAL has also adopted sections of it into its current specifications. Two methods are suggested in specifying durability index values, either a “deemed to satisfy” approach, or a “rigorous” approach. The deemed to satisfy approach is generally very conservative and will be adequate for a vast majority of structures. The rigorous approach will be required for durability critical structures, e.g. structures exposed directly to sea water or where design parameters assumed in the deemed to satisfy approach are not applicable to the bridge in question. Relevant service life models are used in the rigorous approach and conditions of the structure e.g. cover depth, environmental class, desired service life, and material information are input that are appropriate for the structure. This approach allows the designer to input all relevant information appropriate to the structure for a given situation, rather than pre selected conditions and index values. The disadvantage is that this method requires expertise on the part of the designer to ensure that the models are used correctly. SANRAL has chosen not to adopt this method currently, unless absolutely necessary.

The flowchart provided in the guideline document (Stanish, Alexander and Ballim (2006)), for the "deemed to satisfy" approach will be used to indicate SANRAL's current criteria below.

(i) Environment

The environment classifications have been provided in accordance with Table B6301/1 of the SANRAL generic specifications (2008), similar to the classification provided in Table 4-1, from EN206. The guideline document follows the EN206 classifications.

(ii) Desired Service Life

The desired service life followed in terms of the guideline document is category 5, for monumental structures and bridges in which the design working life is 100 years.

(iii) Required cover

While the guideline document recommends typical cover depth of 30mm for a carbonating environment and 50mm for a seawater environment, SANRAL has adopted Table B6301/1 as indicated in the SANRAL generic specifications (2008). Generally, all concrete exposed faces in a carbonated environment is 40mm, and 50mm for buried faces, while parapets have a minimum of 35mm. Cover is measured as a performance criterion, as discussed under section 4.3.1 (b) above.

(iv) Required Durability Index Test Value

(1) Oxygen Permeability Index (OPI)

For carbonating conditions, an OPI value of 9,70 for 40mm cover has been adopted in terms of the guidelines. This is the minimum value required in the as-built structure. In addition, criteria are provided to ascertain a value for the material potential (during mix design stage) and the final as-built value. It must however be noted that this value is adopted on all of SANRAL's structures nationally, and a distinction needs to be made where structures are located in an environment that does not affect the durability of the concrete. An example of this is bridge decks located in a carbonated zone but falling under environmental class XC1 and XC2 i.e. moderate exposure conditions where decks protected from alternative wetting and drying. In these cases, a minimum cover needs to be specified only (at least 30mm) at a minimum strength of 30MPa. Substructures on the other hand will be located in environment class XC3 i.e. severe exposure conditions and exposed to hard rain and alternative wetting and drying cycles. This will be discussed further in chapter 8.

(2) Water Sorptivity Index

Sorptivity only relates to construction factors such as degree of curing and has not been related to a transport process related to deterioration, and therefore cannot be used as a design parameter. The required sorptivity value therefore needs to be established on the site during the mix design stage, and the value increased by 1,1 for acceptance of the actual value in the structure. A maximum value of 12 mm/√hr is recommended in the guidelines. However, due to the uncertainties of this test, data is gathered during the mix design and during construction and only reported on at this stage. It must be noted that in the previous specifications (2007), sorptivity testing was a performance criteria which had to be achieved as well. On some of the projects where testing was undertaken, sorptivity targets had to therefore be achieved. A check will also be done of the ratio of the as-built value and that from the design mix. Table 4-2 provides requirements in the current specifications.

**Table 4-2: Durability Parameters Acceptance Ranges (Table B6404/3)**

| *Acceptance Category* | *Test No./ Description/ Unit* | |
|---|---|---|
| | *B8106(g)(i) Water Sorptivity (mm/⁄h)* | *B8106(g)(ii) Oxygen Permeability (log scale)* |
| *Concrete made, cured and tested in the laboratory* | *Report[1]* | *> 9,80* |
| *Full acceptance of in-situ concrete (Trial panel included)* | *Report[1]* | *> 9,70* |
| *Conditional acceptance of in-situ concrete (with remedial measures approved by the engineer)* | *Not applicable* | *8,75 – 9,70* |
| *Rejection* | *Not applicable* | *< 8,75* |

A note has been included in the specifications that sorptivity results are only reported on at this stage and will be incorporated into future specifications.

(3) Chloride Conductivity Index

A minimum of 50mm together with a range of Chloride Conductivity values has been adopted for monumental structures (including bridges) as recommended by the guideline document. Table 4-3 below has been incorporated into the specifications.

The table shows typical blends only and therefore other blends will need to be tested in the laboratory during the design of the mix and adopted.

**Table 4-3: Appropriate Limits for Chloride Conductivity (mS/cm) (SANRAL generic specifications (2008))**

| *ENV Class* | *70:30 CEM1 : FA* | *50:50 CEM1 : GGBS* | *50 : 50 CEM1 : GGCS* | *90 : 10 CEM1 : CSF* |
|---|---|---|---|---|
| *XS 1* | *2,50* | *2,80* | *3,50* | *0,80* |
| *XS 2a* | *2,15* | *2,30* | *2,90* | *0,50* |
| *XS 2b, XS 3a* | *1,10* | *1,35* | *1,60* | *0,35* |
| *XS 3b* | *0,90* | *1,05* | *1,30* | *0,25* |

*"(For a range of possible cement blends, with minimum cover of 50mm)*

4.3.5.4 Mix Design Approval Process

Approvals of mix design in time for construction to commence are always a difficult issue to control, and in general in order that results for the durability index to be available, finalisation of the mixes can take between 8 to 10 weeks. The contractor is therefore required within 7 days of the commencement date of the contract to provide all relevant materials required for testing.

A major change in the specifications is the addition of the trial panels. Each trial panel is 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 1,0m wide, 1,0m high and 150mm thick. The panel is constructed vertically (for substructures) and horizontally for deck slabs. It most likely will be that one trial panel will be required for substructures (piers, abutments, retaining walls, etc) and another for the decks due to type of casting and curing methods. The same construction practice is followed when constructing the trial panels and the in-situ concrete to ensure that there is a relationship between the two in terms of compaction and curing.

A two stage mix design approval process is followed, the first being for the laboratory mixes which needs to meet the laboratory target requirements. Thereafter, the trial panels are to be constructed and tested.

### 4.3.6 Durability Testing

During the construction, additional test cubes are taken for each structural element and cored for durability testing, the requirements of which are shown in Table B8106/1 in the SANRAL generic specifications (2008) and included in Annexure 1. This is in lieu of the coring of the structure after reaching 28 days strength. Half of the cubes will be cured on site at the position of the element, and half taken to the laboratory for curing. Cores are extracted from these cubes and tested for the durability requirements for each of the concrete elements. The additional cubes are placed on the site where the structural element is being cast so as to simulate similar environmental conditions. If the test results indicate that the durability requirement has not been achieved, then the structural element shall be cored and tested for the durability criteria.

The guideline document of Stanish, Alexander and Ballim (2006) suggests that for each of the index tests an average of three consecutive test results represent a single sample. However, due to the fact that results for the Water Sorptivity test are only being recorded currently to monitor and possibly incorporate into future specifications, an average of two results are being are recorded as a single sample. For the oxygen permeability and chloride conductivity (where required), an average of four tests represent a single result.

Table 4-4 below provides the number of minimum durability core samples required from the test cubes to be cast. Half of the additional cubes taken per pour/element to be cored for durability shall be placed on the site where the structural element is being cast so as to simulate similar environmental conditions and the other half per pour/element cured in the laboratory under controlled conditions. The reason for this is due to avoid any dispute between the ready mix supplier (if used) and the contractor regarding supplying of durable concrete and placement thereof. This method of testing i.e. site cured and laboratory cured samples is very expensive and needs further discussion.

**Table 4-4: Number of Samples required for Durability Testing (SANRAL generic specifications (2008))**

| *Element* | *No. of samples (n) to taken (see Table B8106.1 for definition of one (1) sample and number of cores and required cubes per sample)* |
|---|---|
| *Bridge Decks (<100mm$^3$)* | *1 (per pour)* |
| *Bridge Decks (101m$^3$) to 200m$^3$)* | *2 (per pour)* |
| *Bridge Decks (200m$^3$ and greater)* | *3 (per pour)* |
| *Bridge Piers/Abutments* | *1 (per element)* |
| *Bridge/ Culvert Parapets* | *1 (per element)* |
| *Culvert walls/wing-walls* | *1 (per wall section)* |
| *Culvert bottom slabs* | *1 (per element)* |
| *Culvert top slabs* | *1 (per element)* |
| *Retaining walls* | *1 (per wall section)* |
| *All bases* | *1 (per element/pour)* |

## 4.3.7 Quality Control and Acceptance Criteria

### 4.3.7.1 General

As have been discussed previously, since SANRAL has commenced with specifications for durable concrete, the quality of concrete produced has increased as the workmanship in both production of concrete and placement has increased.

More effort is being paid to curing on the sites since this has become a payment item in the schedule of quantities. When SANRAL embarked on performance based specifications for durable concrete in 2000, it prematurely imposed penalties on all of the durability index test parameters. There was no differentiation between laboratory and in-situ limits, and in addition all concrete was tested for all of the durability requirements.

Currently, only three criteria are used to ensure quality of concrete, viz:

- Strength,
- Oxygen Permeability,
- Chloride Conductivity and
- Cover to reinforcement

Strength requirements have always been imposed as it is a requirement of the standard COLTO specifications. In addition, the Oxygen Permeability, Chloride Conductivity and cover to reinforcement have been included for durability requirements. Limits have been set and these are monitored during the construction phase. Chloride Conductivity is only monitored during the mix design stage and during the construction when sources of materials changes. Where reduced payments apply to more than one of the above criteria, then only the maximum percentage reduction will apply between the criteria on the pay items of the element. It is unfair on a contractor that where all of the above criteria have reduced payments, then all must be imposed on the element i.e. cannot have reduced payment being applied more than once to a specific pay item of the element. It is unwise to owners of infrastructure to spend funds to ensure durable structures are constructed without mechanisms in place to monitor and ensure that what has been paid for has been provided. It is also unwise that a contractor be provided with limits that are not achievable and thereby be imposed with penalties.

#### 4.3.7.2 Limits for cover

Table B8212/2 of the SANRAL generic specifications (2008) included under Annexure 1 shows the limits of full acceptance, partial acceptance and rejection for cover requirements. The reduced payment is applied to the payment item for reinforcement under section 6300 of the schedule of quantities for the specific element which has been tested. The percentage in reduction due to non compliance is considered reasonable. The introduction of the requirements for concrete cover has had a marked change in mindset of the fact that monitoring by both consultants as well as contractor’s site staff needs to take place, and therefore results in improved workmanship.

#### 4.3.7.3 Limits for Oxygen Permeability

Table 4-5 extracted from the specifications shows the limits of full acceptance, partial acceptance and rejection for oxygen permeability.

**Table 4-5: Reduced payments for Oxygen Permeability (SANRAL generic specifications (2008))**

| *DESCRIPTION OF TEST* | *Oxygen permeability index (log scale)* | *PERCENTAGE (%) PAYMENT* |
|---|---|---|
| *Full acceptance* | *> 9,70* | *100 %* |
| *Conditional acceptance (with reduced payment)* | *> 9,25 ≤ 9,70* | *85 %* |
| *Conditional acceptance (with remedial measures as approved by the Engineer and reduced payment)* | *≥ 8,75 ≤ 9,25* | *70 %* |
| *Rejection* | *< 8,75* | *Not Applicable* |

The reduced payment is applied to the payment item for concrete under section 6400 of the schedule of quantities for the specific element which has been tested. Limits for full acceptance and total rejection are based on values in the guideline document as well as the monographs produced previously. Intermediate values for partial payment has been based on previous experience as well as risk exposed to SANRAL to accept substandard work and future maintenance costs thereof.

### 4.3.8 Conclusion

SANRAL specifications have evolved over the years. While the COLTO standard specifications were intended to address shortcomings in the previous CSRA specifications, very little was included in terms of performance based durability specifications. The latest project specifications used on SANRAL contracts incorporates target requirements for cover and oxygen permeability, with the imposition of penalties if not achieved, while limits are placed on chloride conductivity values for various blended binders. Data is being captured for the sorptivity index values on SANRAL sites, before it can be analysed and target values can be set and implemented as a target criterion. However, a distinction needs to be made in terms of elements of a structure required to be designed for durability protected in a carbonating environment.

A major change is coring and testing of samples from trial panels and additional test cubes on the site instead of coring of the structure. Testing undertaken on certain projects will provide conclusions whether this has proved successful or not, and discussed in chapter 7.

# 5 DESCRIPTION OF PROJECTS WHERE TESTING WAS UNDERTAKEN

## 5.1 Introduction

Initially four projects were proposed to undertake testing during the course of 2007 and 2008 to be presented in this dissertation. These projects were chosen as they were the only one's where structural concrete was being constructed within KwaZulu-Natal for the South African National Roads Agency Limited. The Mgeni River Bridge project was included later because of additional trial panels tested on the site. The geographical position of each of the projects is shown in Figure 5-1 below.


**Figure 5.1 : Map showing geographical position of projects (Source, Author)**

Two of the four projects were completed in 2008, one to be completed in 2009 and two due for completion in 2010.

This chapter is divided into five sections, each describing the background for each projects inclusion, the construction being undertaken and mix designs. Important aspects of each project are highlighted and commented on, while discussions on testing and reporting will be presented in chapter 6.

## 5.2 New England Road Interchange

### 5.2.1 Location of the site and Contract Details

The site is located on national route N3 section 3 at the intersection of New England Road, which crosses over the N3. It is located on the northern side of the Pietermaritzburg CBD, and within the Msunduzi Municipality. The contract was let in May 2007, and was completed in July 2008

### 5.2.2 Description of the project (Structural and Concrete Details only)

The project involved the construction of a new bridge adjacent to the existing bridge on the eastern side. A longitudinal joint tied both bridges together. The new bridge has four spans of lengths 10m, 2 x 17m and 12m. The deck on each span consisted of 8 x 1,2m deep prestressed post tensioned concrete beams tied together with diaphragm beams at third points with a 180mm reinforced concrete top slab. The piers consisted of 3 columns each on piled footings. The abutments are perched solid concrete type founded on piles. The parapets are precast reinforced concrete F-Shaped type. An extension of a four cell in-situ reinforced concrete box culvert located beneath New England Road was also constructed under this contract. Further details of the construction are given in Annexure 2.

### 5.2.3 Description of the Environment

This project is located inland from the coast, and therefore not affected by a chloride atmosphere. In terms of SANRAL's specifications, the environment can be classed as Severe, (defined in Table B6301/1 as an environment with moderate humidity of between 60 to 80% and where concrete surfaces are exposed to hard rain and alternative wetting and drying conditions).

The location of the project in Pietermaritzburg is adjacent to heavy industry. If corrosion is to occur to reinforcement steel, it is likely to be induced due to carbonation. Enquiries were made with the Msunduzi Municipality to check if air pollution measurements are available and discussed later in this chapter.

### 5.2.4 Durability and Strength Requirements

For this project, the following durability performance criteria were specified:

- Water sorptivity,
- Oxygen permeability, and
- Concrete Cover

Tables 5-1 and 5-2 below shows the criteria that were specified.

**Table 5-1: Durability requirements for New England Road Bridge (SANRAL Contract N003-003-2005/1, 2007)**

| Acceptance Category | Test No./ Description/ Unit | |
|---|---|---|
| | **Water Sorptivity (mm//h)** | **Oxygen Permeability (log scale)** |
| Concrete made, cured and tested in the laboratory | Average of 2 tests | > 9,80 |
| Full acceptance of in-situ concrete (Trial panel included) | Value above x 1,1 (Max = 12) | > 9,70 |
| Conditional acceptance of in-situ concrete (with remedial measures) | 12,00 – 15,00 | 8,75 – 9,70 |
| Rejection | > 15,00 | < 8,75 |

**Table 5-2 : Other Durability Requirements for New England Road Bridge (SANRAL Contract N003-003-2005/1, 2007)**

| Member | Strength | Curing Regime | Cover (mm) |
|---|---|---|---|
| Abutments/Piers | W30/19 | Curing Compound | 45 |
| Deck | W40/19 | Mist spray/sand | 40 |

### 5.2.5 Concrete Mix Designs

The concrete mix designs and laboratory testing was undertaken by a commercial laboratory, which had the necessary facilities to undertake the durability tests. Table 5-3 below summarises the mix design that was finally adopted.

**Table 5-3: Concrete Mix Design for New England Road Interchange (SANRAL Contract N003-003-2005/1, 2007)**

| **Constituent** | **W30/19 Mix** | **W40/19 Mix** |
|---|---|---|
| Stone | 19mm Dolerite | 19mm Dolerite |
| Sand | Msunduzi River | Msunduzi River |
| Binder | CEM II AS 42,5 | CEM II AS 42,5 |
| Binder Content (kg/m$^3$) | 324 | 351 |
| Slagment (kg/m$^3$) | 91 | 99 |
| Total Binder (kg/m$^3$) | 415 | 450 |
| Water Content (l/m$^3$) | 200 | 200 |
| Binder/ Water Ratio | 2,075 | 2,250 |

## 5.3 Black Mfolozi River Bridge

### 5.3.1 Location of the site and Contract Details

The site is located on a new access road that will connect to provincial roads P702 in the west (the Xasana Community) to P703 in the east (the Esizinda community), and which crosses over the Black Mfolozi River. It is located north of Ulundi and within the Mhlabatini District Municipality. The contract was let in July 2007, and completed in September 2008. This project has been included mainly because it was constructed using labour intensive construction and all concrete was batched on the site. As SANRAL is undertaking some of these community projects, it was essential that the quality was not compromised even though it was being constructed using less plant intensive methods.

### 5.3.2 Description of the project (Structural and Concrete Details only)

The project involved the construction of a new low level bridge linking both communities located either side of the river. The bridge has nine spans of 11,4m lengths. The deck on each span consists of 9 x 0,54m deep reinforced concrete inverted tee beams, which is in-filled to form a solid slab. The piers are solid wall type, four of which are directly anchored into rock, and the other four founded on piles. The abutments are solid concrete type founded and anchored into rock. Further details of the construction are given in Annexure 2.

### 5.3.3 Description of the Environment

This project is located inland from the coast, and therefore not affected by a chloride atmosphere. In terms of SANRAL's specifications, this environment can be classed as Severe, as defined in Table B6301/1 and exposed to alternative wetting and drying of the concrete surface.
The location of this project in northern KwaZulu-Natal has little presence of chlorides and possibly limited carbon dioxide with no major industries in the area.

### 5.3.4 Durability Requirements

For this project, the following durability performance criteria were specified:

- Water sorptivity,
- Oxygen permeability, and
- Concrete Cover

Tables 5-4 and 5-5 below shows the durability criteria that were specified.

**Table 5-4: Durability requirements for Black Mfolozi River Bridge (SANRAL Contract P006-032-2007/1, 2007)**

| **Acceptance Category** | **Test No./ Description/ Unit** | |
|---|---|---|
| | **Water Sorptivity (mm//h)** | **Oxygen Permeability** |
| Concrete made, cured and tested in the laboratory | Average of 2 tests | > 9,80 |
| Full acceptance of in-situ concrete (Trial panel included) | Value above x 1,15 (Max = 12) | > 9,70 |
| Conditional acceptance of in-situ concrete (with remedial measures) | 12,00 – 15,00 | 8,75 – 9,70 |
| Rejection | > 15,00 | < 8,75 |

**Table 5-5: Other Durability Requirements for Black Mfolozi River Bridge (SANRAL Contract P006-032-2007/1, 2007)**

| Member | Strength | Curing Regime | Cover (mm) |
|---|---|---|---|
| Abutments/Piers | W30/19 | Curing Compound | 45 |
| Deck (In-situ) | W30/19 | Mist spray/sand | 45 |
| Precast Beams | W40/19 | Curing Compound | 30 |

### 5.3.5 Concrete Mix Designs

The concrete mix designs and laboratory testing was undertaken by a commercial laboratory, which has the necessary facilities to undertake the durability tests. Table 5-6 below summarises the mix designs.

**Table 5-6: Concrete Mix Design for Black Mfolozi River Bridge (SANRAL Contract P006-032-2007/1, 2007)**

| Constituent | W30/19 Mix | W40/19 Mix |
|---|---|---|
| Stone | 19mm Dolerite | 19mm Dolerite |
| Sand | Mfolozi River | Mfolozi River |
| Binder | CEM III A 32,5 | CEM III A 32,5 |
| Total Binder (kg/m$^3$) | 400 | 425 |
| Water Content (l/m$^3$) | 195 | 175 |
| Binder/ Water Ratio | 2,05 | 2,43 |

## 5.4 Richmond Road Interchange Bridge Upgrade

### 5.4.1 Location of the site and Contract Details

The site is located on national route N3 section 1 at the intersection of Richmond Road, which crosses over the N3 and links the N3 to Pinetown and Marianhill. It is located on the western side of the Durban CBD, and within the Ethekwini Municipality. The contract was let in March 2008, and due for completion in May 2009.

### 5.4.2 Description of the project (Structural and Concrete Details only)

The project involves the construction of a new bridge adjacent to the existing bridge on the western side to increase traffic capacity. A longitudinal joint will tie both bridges together. The bridge has four spans with a total length of 67m and a 26,2m wide skew deck. The deck is continuous and consists of a 1,3m deep prestressed concrete box girder deck. The piers consist of 3 columns each on spread footings. The abutments are perched solid concrete type founded on spread footings. The parapets are precast reinforced concrete F-Shaped type. Further details of the construction are given in Annexure 2.

### 5.4.3 Description of the Environment

This project is located approximately 15km from the coast, and will only be slightly affected by chlorides in the atmosphere. In terms of SANRAL's specifications, this environment can be classed as Severe, defined as moderate humidity (60 – 80%) and where concrete surfaces are exposed to hard rain and alternative wetting and drying conditions. The location of this project close to industry means that concrete will be affected by carbonation as well. Enquiries were made with Ethekwini Municipality on data of air quality measurements, and discussed further at the end of this chapter.

### 5.4.4 Durability Requirements

For this project, the following criteria were incorporated in the specifications for durability:

- Water sorptivity,
- Oxygen permeability,
- Chloride conductivity, and
- Concrete Cover

Tables 5-7 to 5-9 below shows the criteria that were specified.

**Table 5-7: Durability requirements for Richmond Road Interchange (SANRAL Contract N003-010-2008/1, 2008)**

| **Acceptance Category** | **Test No./ Description/ Unit** | |
|---|---|---|
| | **Water Sorptivity (mm//h)** | **Oxygen Permeability** |
| Concrete made, cured and tested in the laboratory | Average of 4 tests | > 9,80 |
| Full acceptance of in-situ concrete (Trial panel included) | Value above x 1,15 (Max = 12) | > 9,70 |
| Conditional acceptance of in-situ concrete (with remedial measures) | 12,00 – 15,00 | 8,75 – 9,70 |
| Rejection | > 15,00 | < 8,75 |

**Table 5-8: Appropriate limits for chloride conductivity – Richmond Road Interchange (mS/cm) (SANRAL Contract N003-010-2008/1, 2008)**

| ENV Class | 70:30 CEM1 : FA | 50:50 CEM1 : GGBS | 50 : 50 CEM1 : GGCS | 90 : 10 CEM1 : CSF |
|---|---|---|---|---|
| XS 1 | 2,50 | 2,80 | 3,50 | 0,80 |
| XS 2a | 2,15 | 2,30 | 2,90 | 0,50 |
| XS 2b, XS 3a | 1,10 | 1,35 | 1,60 | 0,35 |
| XS 3b | 0,90 | 1,05 | 1,30 | 0,25 |

(For a range of possible binder blends, with minimum cover of 50mm)

**Table 5-9: Other Durability Requirements for Richmond Road Interchange (SANRAL Contract N003-010-2008/1, 2008)**

| **Member** | **Strength** | **Curing Regime** | **Cover (mm)** |
|---|---|---|---|
| Abutments/Piers | W35/19 | Curing Compound | 45 |
| Deck (In-situ) | W55/19 | Mist spray/sand | 45 |

### 5.4.5 Concrete Mix Designs

The concrete mix designs and laboratory testing will be undertaken by a commercial laboratory, which has the necessary facilities to undertake the durability tests. Table 5-10 below summarises the mix designs.

**Table 5-10: Concrete Mix Design for Richmond Road Interchange Bridge (SANRAL Contract N003-010-2008/1, 2008)**

| **Constituent** | **W35/19 Mix** | **W45/19 Mix** |
|---|---|---|
| Stone | 19mm Tillite | 19mm Tillite |
| Sand | Mkomaas/Mhlali River | Mkomaas/Mhlali River |
| Binder | CEM II A-S 42,5<br>CEM III A | CEM II A-S 42,5<br>CEM III A |
| Total Binder (kg/m$^3$) | 317 | 444 |
| Water Content (l/m$^3$) | 165 | 185 |
| Binder/ Water Ratio | 1,92 | 2,40 |

It must be noted that the binder content for the W35/19 mix is low compared to other 30MPa mixes.

## 5.5 King Shaka International Airport (KSIA) Interchange Bridges

### 5.5.1 Location of the site and Contract Details

The site is located on national route N2 section 26 and will be the main link for traffic of the N2 with the airport. Two bridges are to be constructed under this project. The site is located on the northern side of the Durban CBD, and within the Ethekwini Municipality. The contract was let in July 2008, and due for completion in March 2010.

### 5.5.2 Description of the project (Structural and Concrete Details only)

The project involves the construction of two new bridges on this interchange described below.

The N2 Overpass Bridge is the link across the N2, allowing for inbound traffic from the northbound carriageway, and will be the future link to the M4 to the east. The bridge will have four spans with a total length of 80m and a 14,97m wide deck. The deck is continuous and consists of a 1,65m deep prestressed single cell concrete box girder deck. The piers consist of 2 columns each on piled footings. The abutments are closed solid concrete type founded on piled footings. The parapets are precast reinforced concrete F-Shaped type.

Bridge 2 (Ramp E Bridge) will carry the outbound traffic from the airport, heading south onto the southbound carriageway of the N2 (loop Ramp E). The bridge will have six spans with a total length of 204m and a 12,5m wide deck, and is 20m above the current N2. The deck is continuous and consists of a 2,50m deep prestressed single cell concrete box girder deck. The piers are solid concrete each on piled footings. The abutments are closed solid concrete type founded on piled footings. The parapets are precast reinforced concrete F-Shaped type. This bridge will be constructed using the Incremental Launching Method, and due to this method of construction, high strengths are required within very short periods in order that the weekly launch cycles can be maintained. Further details of the construction are given in Annexure 2.

### 5.5.3 Description of the Environment

This project is located approximately 5km from the coast, and will be significantly affected by chlorides in the atmosphere. In terms of SANRAL's specifications, this environment can be classed as Very Severe as defined in Table B6301/1.

### 5.5.4 Durability Requirements

For this project, the following criteria were incorporated into the specifications for durability:

- Water sorptivity (record only),
- Oxygen permeability,
- Chloride conductivity, and
- Concrete Cover

Tables 5-11 and 5-12 below shows the criteria that were specified.

**Table 5-11: Durability requirements for (KSIA) Interchange Bridges (SANRAL Contract N002-260-2005/1, 2008)**

| **Acceptance Category** | **Test No./ Description/ Unit** | |
|---|---|---|
| | **Water Sorptivity (mm/h)** | **Oxygen Permeability (log scale)** |
| Concrete made, cured and tested in the laboratory | Report | > 9,80 |
| Full acceptance of in-situ concrete (Trial panel included) | Report | > 9,70 |
| Conditional acceptance of in-situ concrete (with remedial measures s approved by the engineer) | Not applicable | 8,75 – 9,70 |
| Rejection | Not applicable | < 8,75 |

**Table 5-12: Strength Requirements for (KSIA) Interchange Bridges (SANRAL Contract N002-260-2005/1, 2008)**

| **Member** | **Strength** | **Curing Regime** | **Cover (mm)** |
|---|---|---|---|
| Abutments/Piers | W30/19 | Curing Compound | 50 |
| Deck (In-situ) | W40/19 | Mist spray/sand | 40 |

### 5.5.5 Concrete Mix Designs

The concrete mix designs and laboratory testing are being undertaken by a laboratory setup on the site. The durability testing will however be undertaken by a commercial laboratory off-site.

**Table 5-13: Concrete Mix Design for KSIA Interchange Bridges (SANRAL Contract N002-260-2005/1, 2008)**

| **Constituent** | **W30/19 Mix** | **W40/19 Mix** |
|---|---|---|
| Stone | 19mm Tillite | 19mm Tillite |
| Sand | Oaklands River/ Pit Sand | Oaklands River/ Pit Sand |
| Binder | CEM II A-S 42,5<br>Slagment | CEM II A-S 42,5<br>CEM III A |
| Binder Content (kg/$m^3$) | 321 | 337 |
| Slagment (kg/$m^3$) | 70 | 74 |
| Total Binder (kg/$m^3$) | 391 | 411 |
| Water Content (l/$m^3$) | 176 | 185 |
| Binder/ Water Ratio | 2,22 | 2,22 |

## 5.6 Mgeni Interchange River Bridges

### 5.6.1 Location of the site and Contract Details

The site is located on national route N2 section 25 in Durban and the interchange is one of the most congested on the N2. The existing N2 bridges as well as the service road bridges over the Mgeni River will be widened to allow for direct links onto the N2. The contract was let in August 2008, and due for completion in March 2010. This project has been included into the study because of numerous trial panels that were cast and tested the results of which were compared to the results from the in-situ beams.

### 5.6.2 Description of the project (Structural and Concrete Details only)

The project involves the construction of the extension of three bridges on this interchange. The widened bridges will allow direct links of traffic onto the service road bridges thereby bypassing the Inanda Intersections and reducing the congestion. The bridges will have five spans with a total length of 250m consisting of tee beams of 2,1m depth.

### 5.6.3 Description of the Environment

This project is located approximately 5km from the coast, and will be significantly affected by chlorides in the atmosphere. In terms of SANRAL's specifications, this environment can be classed as Very Severe as defined in Table B6301/1.

### 5.6.4 Durability Requirements

For this project, the following criteria were incorporated into the specifications for durability:

- Water sorptivity (record only),
- Oxygen permeability,
- Chloride conductivity, and
- Concrete Cover

Tables 5-14 and 5-15 below shows the criteria that were specified.

**Table 5-14: Durability requirements for Mgeni Interchange Bridges (SANRAL Contract N002-250-2008/2, 2008)**

| Acceptance Category | Test No./ Description/ Unit | |
|---|---|---|
| | **Water Sorptivity (mm/h)** | **Oxygen Permeability (log scale)** |
| Concrete made, cured and tested in the laboratory | Report | > 9,80 |
| Full acceptance of in-situ concrete (Trial panel included) | Report | > 9,70 |
| Conditional acceptance of in-situ concrete (with remedial measures s approved by the engineer) | Not applicable | 8,75 – 9,70 |
| Rejection | Not applicable | < 8,75 |

**Table 5-15: Strength Requirements for Mgeni Interchange Bridges (SANRAL Contract N002-250-2008/2, 2008)**

| Member | Strength | Curing Regime | Cover (mm) |
|---|---|---|---|
| Deck (In-situ) | W55/19 | Mist spray/sand | 40 |

### 5.6.5 Concrete Mix Designs

The concrete mix designs and laboratory testing are being undertaken by a laboratory setup on the site. The durability testing will however be undertaken by a commercial laboratory off-site.

**Table 5-16: Concrete Mix Design for Mgeni Interchange Bridges (SANRAL Contract N002-250-2008/2, 2008)**

| **Constituent** | **W55/19 Mix** |
|---|---|
| Stone | 19mm Tillite |
| Sand | Umkomaas River Sand |
| Binder | CEM II A-S 42,5<br>Slagment |
| Binder Content (kg/m$^3$) | 335 |
| Slagment (kg/m$^3$) | 140 |
| Total Binder (kg/m$^3$) | 475 |
| Water Content (l/m$^3$) | 173 |
| Binder/ Water Ratio | 2,78 |

## 5.7 Air Quality Monitoring

As discussed previously, enquiries have been made with both Ethekwini Municipality and Msunduzi Municipality with regard to air quality monitoring since many of the projects under discussions fall within these two municipalities. Below are some of the results from the survey undertaken.

### 5.7.1 Ethekwini Municipal Boundary

With regard to atmospheric $CO_2$ measurements, current concentration levels provided by the municipality are 383,5 parts per million (ppm). Internationally, the Intergovernmental Panel for Climate Change (IPCC) has reported a value of 379 ppm for 2005 (The Independent UK, February 2007). What this indicates is that levels have increased due to emissions from industries. The municipality believes that the average growth rate in the region is approximately 1,5 ppm per annum. This indicates that $CO_2$ emission levels within the region are average, although there may be some areas especially the industrial areas that may have higher levels. This gives reasons to ensure that with rising $CO_2$ emission levels, carbonation induced corrosion is becoming an issue, and therefore bridges are needed to be constructed to ensure that the long term durability is maintained.

With regards to chloride levels, the municipality undertook surveys on concentrations of NaCl (salt) in certain key areas. From the data surveyed from the suburb of Wentworth located on the coast in July 2006, the average concentration of Chloride (Cl) found was 19,3 $\mu g/m^3$. In general, chloride levels at the coast are in the order of 19,000 $mg/m^3$ which means that levels recorded are average coastal levels of chlorides. The important issue is level of chloride migration away from the coast. No data was available to quantify chloride decrease with distance from the coast.

### 5.7.2 Msunduzi Municipal Boundary

Unlike Ethekwini Municipality, Msunduzi has limited facilities to monitor air condition. The last monitoring undertaken was from November 2006 until October 2007. A number of different gases were monitored. Since carbon monoxide (CO) is a major industrial gas which burns in air to form $CO_2$, its levels are monitored within the city due to the heavy industries present. Average levels measured where 8,7 ppm. This level is not high as average household levels are between 0,5 to 5ppm. However it must be noted that CO levels can change drastically from time to time depending on industrial usage at time of measurements.

## 5.8 Discussion

SANRAL is currently embarking on substantial infrastructure spending. These five projects in KwaZulu-Natal are only a portion of the bridge projects being undertaken. Other bridge projects are also due to commence in the latter of 2008 and early 2009 in KwaZulu-Natal, with major spending (approx. R12 billion total project cost) planned in Gauteng for the Gauteng Freeway Improvement Plan (GFIP) over the next two years.

All five projects are located at various places within the KwaZulu-Natal province and exposed to different environmental conditions. The projects vary in nature from labour intensive construction to substantially heavy civil structures across major highways and rivers. The mix designs for each project have been undertaken by the contractor through a commercial laboratory facility. As will be seen from the durability requirements of the projects, the only distinct difference in the specifications is that the three projects commencing in 2006 and early in 2007 had the target values for water sorptivity whereas for the project, sorptivity values are only reported on. This was due to the revision in specifications as the exact effects of the workmanship and material design parameters on sorptivity are still to be verified. Testing was undertaken on all five sites by casting a number of trial panels and coring form them as well as from the structure. In addition, on certain of the projects, coring was also done on test cubes as part of the testing requirement.

Testing will still be undertaken on those projects which are incomplete at time of submission of this dissertation in order that a database of the results can be created and further trends can be investigated and these results will be incorporated into future specifications.

What is clear is that climate changes is having an impact on design of bridge infrastructure, and while the surveys undertaken at Ethekwini and Msunduzi Municipalities shows that levels being recorded are still average levels, worldwide there has been an increase. The World Road Association (PIARC) has chosen as one of its themes over the next four years the issue of impact of climatic change on bridge infrastructure.

# 6 LIMITATIONS OF CORING TEST CUBES AND PANELS FOR DURABILITY TESTING

## 6.1 Introduction

SANRAL's specifications require that testing for durability be undertaken on both trial panels cast prior to any concrete construction commencing on site as well as on additional test cubes during the construction. This method of non-destructive testing prevents cores being drilled out of structures which render it vulnerable to ingress of corrosive agents at the core hole positions. In addition, access has always been an issue in order to drill cores at critical areas of bridge elements. The issue being raised is that this method of testing does not represent what has been cast in the structure. It is similar to the testing for compressive strength using the standard cube testing method, which may not represent the true compressive strength of the structure concrete. However, compressive strength is not as sensitive to workmanship i.e. curing and compaction, as is the sorptivity and oxygen permeability tests.

## 6.2 Testing Environment

The major problem with using the trial panels and test cubes for testing of durability parameters is that their sizes restrict them to providing a fair comparison of the structural concrete element. Compacting and curing small concrete elements are much simpler and easy to undertake than large elements like top slabs of bridge decks. However, the results of testing of trial panels for durability that are constructed with the same techniques as the in-situ structure will be discussed further based on the results of the various sites where testing is intended to be undertaken.

### 6.2.1 Trial panels

The trial panels were 1m x 1m x 150mm depth. Panels cast vertically are to represent substructures and webs of bridge decks while horizontally cast panels are to represent wide open areas like top of bridge decks. These panels are to be cast using the same methods of construction as the structural element i.e. use same type of shutter, stripping time, compaction of concrete as well as curing regime.

The size of the panels has chosen where to ensure that good compaction could be achieved. The contractor will need to ensure that the good curing processes are followed since the core results need to meet the required target values i.e. similar to in-situ results should the structure be cored and tested. An inspection of the trial panels cast on each of the sites indicated good sound concrete exists and a uniform curing system applied. It is a requirement that both oxygen permeability and sorptivity test be undertaken as well as chloride conductivity (should it be required).

Based on the above discussions, it is believed that all of the required tests should meet the target requirements.

### 6.2.2 Test Cubes

The standard 150 x 150 mm test cubes used for compressive testing are used to take additional samples for coring for durability. At least half of the test cubes taken are cured in the laboratory under standard conditions and the other half cured and exposed to the environment similar to the structure. The reasons are twofold; firstly to monitor any specific trends between the lab and site cured samples, and secondly, to ensure that no conflicts arise between the concrete supplier and the contractor with the regard of the concrete meeting the required durability indexes. The costs of additional samples and testing for the latter issue should however be between the supplier and the contractor and not SANRAL.

The concrete in the test cube is compacted similar to test cubes produced for the compressive cube tests using a metal rod. Two issues could arise out of the compaction; either there would be voids in the cubes because of the type of hand compaction compared to vibration of the in-situ concrete or because of the size of the cube, hand compaction could allow full compaction of the concrete thereby eliminating the presence of any voids. This has been the standard and accepted method of cube compaction to test for compressive strength and therefore it is unlikely that sub-standard concrete quality will result. It must also be noted that if the OPI results are below the required limit, it would be an indication that the concrete test cubes have not been compacted or cured properly. An alternative method of ensuring good compaction of test cubes could be to use the vibrating table which was a standard method of compaction for test cubes in past specifications. It will also reduce the number of test cubes required for compressive testing if a single vibrated cube result represents a sample result unlike the standard cube test where three cubes represent a single sample result because of the variability in compaction using the tamping rod. The vibrating table is however currently not used on any of SANRAL contracts.

With regard to core extraction from test cubes for durability testing, the latest durability test methods indicates that cores are to be drilled at right angles to the direction of casting i.e. to apposite cast (formed) faces, as concrete of high workability results in the top trowelled surface not being representative of the concrete. Cores have in the past been extracted and tested from both horizontal and vertical cast faces by Contest Concrete Services (the only commercial laboratory setup for durability testing in KwaZulu-Natal), and the orientation did not affect the results. All core extraction will therefore be done from the vertical faces i.e. horizontally on the test cubes. Cores may also be extracted from horizontal faces like top of decks to check any variability in durability quality as it may be that decks due to access constraints be drilled on the top in future. This however needs to be tested and proven on structures that no variability exists on top of finished deck surfaces.

### 6.2.3 In-situ Testing

SANRAL's specification currently does not require that cores be drilled from the structure. However, as part of this dissertation, cores are drilled from the structure to compare in-situ durability index results versus core results extracted from the test cubes. Limited in-situ testing will be undertaken however, as the durability of the in-situ concrete may be compromised by drilling into the structure at various locations. Coring on bridge decks over rivers and roadways may also be difficult due to access constraints, but sufficient cores will be extracted from the structures such that informed conclusions can be drawn.

It is clear that the size and shape of all bridge concrete elements i.e. piers, abutments, wingwalls, deck beams and in-situ decks, allows for good compaction and curing, except for bridge parapets, which are narrow and its shape makes it difficult to get full compaction in some of the corners. For deep pours, those generally greater than 2,4m depth, care needs to be exercised such that poker vibrators are sufficiently used to ensure adequate compaction.

The results from the in-situ coring should therefore meet the required durability index parameter if the concrete has been placed in accordance with the specifications.

## 6.3 Discussion

Makeup of the various test samples could influence the results for oxygen permeability, water sorptivity and chloride conductivity if not correctly prepared. The results for the index tests are very sensitive to curing and compaction as well as material properties like aggregate quality, binder type and content. However, if the design mixes have conformed to the targets of the specifications, then the in-situ results should be achieved if the same materials are used as that for the design mix and if correctly prepared.

Chapters 7 and 8 will provide further discussion after analysis of the test results.

# 7 TESTING, ANALYSIS AND EVALUATION OF DURABILITY INDEXES

## 7.1 Introduction

Testing required on the various sites was undertaken jointly by the contractors (on remote sites, the contractors extracted the cores) and the commercial testing firm Contest Concrete Technologies, who are located in Westmead, Durban. Durability Testing was specified under each of the projects for the trial panels and additional cubes. However as in-situ cores were also extracted from the bridges and laboratory testing undertaken, additional costs were incurred for these and paid by SANRAL. Presented in this chapter are the results of these test analysis and evaluation for the various projects.

## 7.2 Linear regression and correlation

Although statistical significance testing could have been used it was decided rather to use correlation based testing to check the closeness of sets of data. In order to correlate data sets, the Pearson product moment correlation coefficient ‘r’ was used. The ‘r’ value is a dimensionless index and ranges inclusively from -1.0 to 1.0. The value indicates the extent of a linear relationship when comparing two sets of data. The coefficient ‘r’ is represented by the following equation:

$$r = \frac{\sum (x-\bar{x})(y-\bar{y})}{\sqrt{\sum (x-\bar{x})^2 \sum (y-\bar{y})^2}} \qquad (7.1)$$

Where

r = correlation coefficient

x and y = two arrays of a sample

x and ÿ = average of two arrays of a sample

When two sets of values (measurement variables) tend to move together— i.e. when high values of one variable are associated with high values of the other, there is a positive correlation (between 0 to 1). Conversely, when smaller values of one variable

is associated with large values of the other, there is a negative correlation (between 0 to -1). When there is no relation between both variables, the correlation will be near to zero (0).
The commonly referred to ‘r-squared’ value can be regarded as the ratio of the variance in y attributable to the variance in x for two sets of data of y and x.
Linear regression trend lines are therefore plotted through the various data points on the graphs that follow.

In addition, for the various test result and sample data tables, the Coefficient of Variation (CoV) is shown, which is regarded as a statistical measure of the dispersion of various data points in a data series spaced around the mean. It is calculated as follows:

$$\text{Coefficient of Variation (\%)} = \frac{\text{Standard Deviation of a sample}}{\text{Mean of a sample}} \times 100 \qquad (7.2)$$

From equation 7.2, the CoV represents the ratio of the standard deviation to the mean. It is a useful statistic for comparing the degree of variation from one data series to another.

## 7.3 Mix designs and trial panels

### 7.3.1 New England Road Interchange Bridge

This was the first site where the new approach of trial panels and additional cubes for durability concrete was incorporated into the specifications and implemented on site. The durability specifications were new to the site staff and therefore the correct testing protocol was not adhered to as was required. Additional cube samples were only taken of certain of the members and therefore a limited comparison was done of in-situ tests.

The trial mixes were designed to achieve the desired target values as shown in Table 7-1 below. The trial panel was constructed and left 24hours in the vertical forms. It was then stripped and a wax emulsion type curing compound was applied within one hour after stripping. Test results obtained showed that both the laboratory and trial panel values were above the minimum target ranges for oxygen permeability and below the maximum for sorptivity. While it was not a requirement under the project for chloride conductivity testing, in-experience by the site staff and consulting engineer with regard to durability testing resulted in the concrete being designed for chloride conductivity limit and tests were undertaken for the laboratory mix and on the trial panels. The results are provided here for completeness only. It must be noted that an average of four tests results were used to obtain the results shown in the table.

**Table 7-1: Laboratory and trial panel results for OPI and Sorptivity at New England Road Bridge**

| TEST PROTOCOL | CONCRETE GRADE | DATE OF CAST | CURING REGIME | ELEMENT/ POSITION | AVERAGE 28 DAY STRENGTH (MPa) | OPI (log value) | | Sorptivity (mm /√hr) | | Chloride Conductivity (mS/cm) | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Ave | CoV (%) | Ave | CoV (%) | Ave | CoV (%) |
| LABORATORY | TARGET | | | | | >9,80 | -- | Ave. of 2 tests | -- | <2,80 | -- |
| | W30/19 | 15.06.2007 | Curing Compound | Substructures | 44.4 | 10.14 | 1.11 | 6.81 | 6.71 | 0.63 | 9.35 |
| | W40/19 | 15.06.2007 | Curing Compound | Deck beams/slab | 53.6 | 10.39 | 1.27 | 4.87 | 5.95 | 0.46 | 9.06 |
| TRIAL PANEL | TARGET | | | | | >9,70 | -- | 1,1 X lab, max =12 | -- | <2,80 | -- |
| | W40/19 | 15.06.2007 | Curing Compound | Precast beams | 53.6 | 10.04 | 1.04 | 7.66 | 14.92 | 0.92 | 11.55 |
| | W40/19 | 15.06.2007 | Curing Compound | Deck slab | 53.6 | 10.07 | 1.49 | 6.40 | 22.54 | 0.77 | 12.01 |

The results show a variability between the laboratory and trial panel tests as would have been expected since the laboratory tests are undertaken under controlled conditions as well as being undertaken with the test cubes. Trial panels were only cast for the decks (W40/19 mix) and not for any of the substructures (W30/19 mix).

The construction of the precast beams (first structural members to be cast) thereafter commenced once the trial panels proved that the in-situ target values could be achieved, as is prescribed by the specifications.

### 7.3.2 Black Mfolozi River Bridge

The construction at this site commenced two months after New England Road Interchange. However this site was unique in that all work was undertaken to maximize local labour. All concrete was manufactured on the site using drum mixers, and the concrete was manufactured to reasonable quality. The site supervision staff consisted of young technical people, who were very eager to get involved with the durability specifications and the construction of the trial panels and manufacture of the additional cubes.

The trial mixes were designed to achieve the desired target values as shown in Table 7-2 below. The test results show that both the laboratory and trial panel values were above the minimum target values for oxygen permeability and below the maximum value for sorptivity. It must be further noted that an average of four results were used to obtain the results shown in the table.

**Table 7-2: Laboratory and trial panel results for OPI and Sorptivity at Black Mfolozi River Bridge**

| TEST PROTOCOL | CONCRETE GRADE | DATE OF CAST | CURING REGIME | ELEMENT/ POSITION | AVERAGE 28 DAY STRENGTH (MPa) | OPI (log value) | | Sorptivity (mm /√hr) | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Ave | CoV (%) | Ave | CoV (%) |
| LABORATORY | TARGET | | | | | >9,80 | -- | Ave. of 2 tests | -- |
| | W30/19 | 20.08.2007 | Wet cured | Substructures | 51.8 | 10.25 | 1.96 | 5.50 | 19.75 |
| | W30/19 | 19/09/2007 | Wet cured | Substructures | 52.1 | 10.19 | 1.13 | 7.96 | 22.10 |
| TRIAL PANEL | TARGET | | | | | >9,70 | -- | 1,1 X lab, max =12 | -- |
| | W40/19 | 2007/10/11 | Curing Compound | Vertical, for pre-cast | 55.5 | 10.06 | 2.62 | 7.56 | 4.00 |
| | W30/19 | 2007/10/10 | Curing Compound | Vertical, for pier 1 | 57.9 | 10.23 | 3.10 | 6.26 | 10.58 |
| | W30/19 | 2008/03/17 | Curing Compound | Horizontal, deck span 3 | 51.8 | 10.37 | 3.03 | 5.79 | 8.44 |
| | W30/19 | 2007/11/12 | Curing Compound | Vertical for abutment | 57.4 | 10.41 | 1.78 | 4.10 | 6.92 |

Durability testing of the W40/19 mix was not done as the contractor was of the opinion that if the W30/19 results met the required targets then it will also have been met on the W40/19 mix. While this was a contentious issue, the contractors was allowed to progress, and prove that durability targets could be met on site. The construction of the precast beams and substructures commenced once the trial panels proved the in-situ values could be achieved.

### 7.3.3 Richmond Road Interchange Bridge

The construction at this site commenced in February 2008. Durability laboratory testing was undertaken by the ready mix supplier for the mix designs. Both the contractor and engineer were for the first time exposed to durability requirements, and both showed enthusiasm in ensuring that all requirements were met. The trial mixes were designed to achieve the desired target values as shown in Table 7-3 below. Actual values show that both the laboratory and trial panel values were within the target range for sorptivity and chloride conductivity. For OPI, the laboratory value obtained was marginal and the contractor carried the risk during the construction of the trial panels which met the requirement for the substructures. It must again be noted that an average of four results was used to obtain the results shown in the table.

**Table 7-3: Laboratory and trial panel values for OPI and Sorptivity at Richmond Road Interchange Bridge**

| TEST PROTOCOL | CONCRETE GRADE | DATE OF CAST | CURING REGIME | ELEMENT/ POSITION | AVERAGE 28 DAY STRENGTH (MPa) | OPI (log value) | | Sorptivity (mm /√hr) | | Chloride Conductivity (mS/cm) | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Ave | CoV (%) | Ave | CoV (%) | Ave | CoV (%) |
| LABORATORY | TARGET | | | | | >9,80 | -- | Ave. of 2 tests | -- | <2,80 | -- |
| | W35/19 | 28..05.2008 | Wet cured | Substructures | 47.0 | 9.38* | 2.77 | 6.97 | 32.80 | 0.39 | 10.48 |
| | W45/19 | 28.05.2008 | Wet cured | Deck | 52.5 | 10.26 | 2.35 | 5.78 | 37.23 | 0.16 | 29.46 |
| TRIAL PANEL | TARGET | | | | | >9,70 | -- | 1,1 X lab, max =12 | -- | <2,80 | -- |
| | W35/19 | 08.05.2008 | Curing Compound | Substructures - Vertical | 43.7 | 9.99 | 0.50 | 4.65 | 8.51 | | |
| | W45/19 | 31.07.2008 | Curing Compound | Decks - Horizontal | 50.6 | 9.70 | 2.58 | 5.53 | 2.06 | 0.21 | 5.50 |
| | W45/19 | 31.07.2008 | Curing Compound | Decks - vertical | 50.6 | 10.53 | 2.79 | 4.84 | 22.65 | 0.17 | 12.31 |
| | W45/19 | 24.10.2008 | Curing Compound | Balustrades - vertical | 50.6 | 10.28 | 3.33 | 6.31 | 13.18 | | |
| | W35/19 | 02.07.2008 | Curing Compound | Substructure columns - | 26.0 | | | | | 0.18 | 15.09 |

Note : * - value below minimum value of 9,80 for laboratory requirement.

The construction of the substructures commenced once the trial panels proved the in-situ values could be achieved.

### 7.3.4 King Shaka International Airport Bridges

The construction at this site commenced in May 2008, with structural works commencing in August 2008. Durability laboratory testing was undertaken by the ready mix supplier for the mix designs and checked by an independent commercial laboratory. The trial mixes were designed to achieve the desired target values as shown in Table 7-4 below. Actual values show that both the laboratory and trial panel values were within the target range. It must be noted that an average of four determinations was used to obtain the results shown in the table.

It is to be noted that although the specifications required both W30 and W40 mixes, the contractor adopted the W40 mix for all the structural elements to reduce the time taken for the mix designs. The construction of the substructures commenced once the trial panels proved the in-situ values could be achieved.

**Table 7-4: Laboratory and trial panel values for OPI and Sorptivity at King Shaka International Airport Bridges**

| TEST PROTOCOL | CONCRETE GRADE | DATE OF CAST | CURING REGIME | ELEMENT/ POSITION | AVERAGE 28 DAY STRENGTH (MPa) | OPI (log value) | | Sorptivity (mm /√hr) | | Chloride Conductivity (mS/cm) | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Ave | CoV (%) | Ave | CoV (%) | Ave | CoV (%) |
| LABORATORY | TARGET | | | | | >9,80 | -- | Ave. of 2 tests | -- | <2,80 | -- |
| | W40/19 | 05.06.2008 | Wet cured | Substructures | 47.3 | 10.52 | 1.47 | 4.27 | 10.42 | 0.22 | 4.30 |
| | W40/19 | 05.06.2008 | Wet cured | Overpass Deck | 47.3 | 10.59 | 0.04 | 3.77 | 6.07 | 0.25 | 11.49 |
| | W60/19 | 02.03.2009 | Wet cured | Ramp E Deck | 80.6 | 10.77 | 2.47 | 6.13 | 8.91 | 0.15 | 12.17 |
| TRIAL PANEL | TARGET | | | | | >9,70 | -- | 1,1 X lab, max =12 | -- | <2,80 | -- |
| | W30/19 | 26.09.08 | Curing Compound | Substructures - Vertical | 50.6 | 10.02 | 2.88 | 5.86 | 16.93 | 0.17 | 9.90 |

## 7.3.5 Mgeni Interchange Bridges

This project has been included only because of comparison of trial panel results with in-situ results and has been included here for completeness. The construction at this site commenced in September 2008, with structural works commencing in May 2009. Durability laboratory testing was undertaken by the ready mix supplier for the mix designs and checked by an independent commercial laboratory. The trial mixes were designed to achieve the desired target values as shown in Table 7-5. Actual values show that both the laboratory and trial panel values were within the target range.

It must be noted that an average of four determinations was used to obtain the results shown in the table. Only mix designs were required for the deck since the sub structures for the widening had already been constructed in the original construction of the interchange. The construction of the deck commenced once the trial panels proved the in-situ values could be achieved.

**Table 7-5: Laboratory and trial panel values for OPI, and Chloride Conductivity at Mgeni Interchange Bridges**

| TEST PROTOCOL | CONCRETE GRADE | DATE OF CAST | CURING REGIME | ELEMENT/ POSITION | AVERAGE 28 DAY STRENGTH (MPa) | OPI (log value) | | Sorptivity (mm /√hr) | | Chloride Conductivity (mS/cm) | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Ave | CoV (%) | Ave | CoV (%) | Ave | CoV (%) |
| LABORATORY | TARGET | | | | | >9,80 | -- | Ave. of 2 tests | -- | <2,80 | -- |
| | W55/19 | 08.10.2008 | Wet cured | Deck | 68.5 | 10.3 | 3.51 | 5.26 | 10.19 | 0.28 | 13.6 |
| TRIAL PANEL | TARGET | | | | | >9,70 | -- | 1,1 X lab, max =12 | -- | <2,80 | -- |
| | W55/19 | 08.10.2008 | Curing Compound | Decks - Horizontal | 68.5 | 10.34 | 0.96 | 5.88 | 10.32 | 0.52 | 25.88 |
| | W55/19 | 08.10.2008 | Curing Compound | Decks - Horizontal | 68.5 | 10.36 | 3.36 | 7.68 | 20.91 | 0.37 | 5.61 |
| | W55/19 | 08.10.2008 | Curing Compound | Decks - vertical | 68.5 | 10.57 | 2.47 | 6.24 | 9.09 | 0.31 | 6.83 |
| | W55/19 | 08.10.2008 | Curing Compound | Decks - vertical | 68.5 | 10.50 | 4.39 | 7.01 | 26.37 | 0.43 | 35.42 |
| | W55/19 | 08.10.2008 | Curing Compound | Decks - vertical | 68.5 | 10.17 | 2.20 | 3.98 | 19.63 | 0.47 | 30.32 |

## 7.4 Elements tested and summary of results

### 7.4.1 New England Road Interchange Bridge

A limited number of test cubes were taken of the precast beams, contrary to the requirements in the specifications. Another major issue on the project was that additional cubes for durability testing on the substructures were only taken on a limited number of casts and only done on two of the culvert casts and one of the pier head pours. A total of 36 additional cube samples were taken from certain of the 32 precast beams on site as well as 22 for the culvert slab and pier head. With regard to in-situ coring and testing, cores were drilled from the edge beams on each of the end spans.

For the substructures, cores were drilled initially at the lower portion of the piers. With the results not meeting the targets for OPI, cores were further drilled 1m above ground level, which again proved unsuccessful and further cores were extracted 2m above ground level. Figure 7-1 below shows the position of the core extractions for durability testing on the structure.

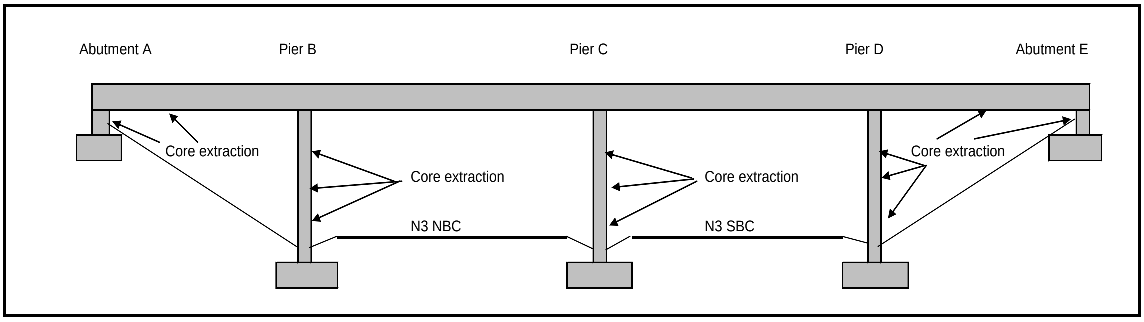



**Figure 7.1 : Position of core extractions at New England Road Bridge**

Coring of the edge beams and upper sections of the piers were done using scaffolding that was set up. Table 7-6 below provides the summarised results from the testing carried out from both the test cubes as well as the in-situ coring. Full determinations are provided under Annexure 3. The sample results are an average of four tests for OPI and two tests for sorptivity.

**Table 7-6: Results of Durability Testing undertaken at New England Road Bridge**

| CONCRETE GRADE | DATE OF CAST | CURING REGIME | ELEMENT/ POSITION | AVERAGE 28 DAY STRENGTH (MPa) | Cubes Cured on site for 28 days | | | | In-situ cores | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | OPI (log value) | | Sorptivity(mm /√hr) | | OPI (log value) | | Sorptivity(mm /√hr) | |
| | | | | | Ave | CoV (%) | Ave | CoV (%) | Ave | CoV | Ave | CoV (%) |
| W30/19 | 02.10.2007 | Curing Compound | Pier 1 -bottom of column | 49.4 | 9.62 | 1.11 | 9.33 | 4.19 | 9.02 | 4.02 | 8.89 | 12.70 |
| W30/19 | 13.09.2007 | Curing Compound | Pier 1 -bottom upstand beam | 50.8 | - | - | - | - | 9.36 | 1.73 | 8.33 | 8.09 |
| W30/19 | 18.09.2007 | Curing Compound | Pier 2 -bottom of column | 52.9 | 9.72 | 1.60 | 7.19 | 8.70 | 9.27 | 2.74 | 8.67 | 14.84 |
| W30/19 | 21.09.2007 | Curing Compound | Pier 2 -bottom upstand beam | 45.3 | - | - | - | - | 9.32 | 5.78 | 7.93 | 14.30 |
| W30/19 | 28.09.2007 | Curing Compound | Pier 3 -bottom of column | 42 | 9.85 | 1.59 | 8.88 | 5.30 | 9.18 | 2.74 | 8.69 | 13.32 |
| W30/19 | 03.09.2007 | Curing Compound | Pier 3 -bottom upstand beam | 40.4 | - | - | - | - | 9.20 | 1.73 | 7.95 | 14.30 |
| W30/19 | 24.10.2007 | Curing Compound | West abutment - wall | 42.7 | 9.72 | 1.60 | 7.19 | 8.70 | 9.16 | 4.02 | 8.13 | 14.84 |
| W30/19 | 17.10.2007 | Curing Compound | West abutment - 1m above ground | 41.9 | 9.72 | 1.60 | 7.19 | 8.70 | 9.15 | 3.89 | 7.80 | 8.09 |
| W30/19 | 02.10.2007 | Curing Compound | Pier 1 - 1m above ground | 49.4 | 9.62 | 1.11 | 9.33 | 4.19 | 9.22 | 3.68 | 9.51 | 12.70 |
| W30/19 | 18.09.2007 | Curing Compound | Pier 2 - 1m above ground | 52.9 | 9.72 | 1.60 | 7.19 | 8.70 | 9.49 | 2.23 | 11.42 | 13.85 |
| W30/19 | 28.09.2007 | Curing Compound | Pier 3 - 1m above ground | 42 | 9.85 | 1.59 | 8.88 | 5.30 | 8.92 | 3.68 | 8.60 | 13.41 |
| W30/19 | 08.10.2007 | Curing Compound | East abutment- 1m above ground | 45.1 | - | - | - | - | 9.28 | 0.30 | 8.79 | 11.50 |
| W40/19 | 17.08.2007 | Curing Compound | Beam [8] (1) Span 1 (South) | 49.1 | 9.76 | 0.77 | 8.87 | 4.36 | 9.02 | 1.27 | 13.77 | 7.00 |
| W40/19 | 21.09.2007 | Curing Compound | Beam [1] (21) Span 1 (North) | 48.5 | 9.51 | 0.49 | 10.24 | 8.60 | 9.28 | 2.80 | 10.10 | 2.26 |
| W40/19 | 04.10.2007 | Curing Compound | Beam [8] (28) Span 4 (South) | 44 | 10.15 | 1.10 | 7.16 | 7.64 | 9.41 | 0.46 | 8.60 | 13.41 |
| W40/19 | 03.10.2007 | Curing Compound | Beam [1] (27) Span 4 (North) | 48.4 | 10.21 | 1.37 | 6.97 | 10.36 | 9.02 | 3.17 | 8.79 | 11.50 |
| W30/19 | 02.10.2007 | Curing Compound | Pier 1-2m above base | 49.4 | 9.62 | 1.11 | 9.33 | 4.19 | 9.23 | 3.60 | 9.93 | 27.86 |
| W30/19 | 18.09.2007 | Curing Compound | Pier 2-2m above base | 52.9 | 9.72 | 1.60 | 7.19 | 8.70 | 9.06 | 2.66 | 9.02 | 5.64 |
| W30/19 | 28.09.2007 | Curing Compound | Pier 3-2m above base | 42 | 9.85 | 1.59 | 8.88 | 5.30 | 8.80 | 0.46 | 9.98 | 12.47 |
| W30/19 | 22.08.2007 | Curing Compound | Deck 2 - Culvert | 43.9 | 9.62 | 1.11 | 9.33 | 4.19 | 9.15 | - | 9.44 | - |
| W30/19 | 22.08.2007 | Curing Compound | Deck 2 -culvert | 43.9 | 9.74 | 1.52 | 8.64 | 11.85 | 9.27 | - | 9.70 | - |
| W40/19 | 22.08.2007 | Curing Compound | Beam 2 | 54.5 | 9.76 | 0.77 | 8.87 | 4.36 | 9.02 | - | 13.77 | - |
| W40/19 | 27.08.2007 | Curing Compound | Beam 3 | 56.5 | 9.51 | 0.49 | 10.24 | 8.60 | 9.28 | - | 10.10 | - |

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| W40/19 | 27.08.2007 | Curing Compound | Beam 4 | 52.8 | 10.15 | 1.10 | 7.16 | 7.64 | *9.41* | - | 8.60 | - |

**Table 7-6 *Continued***

| CONCRETE GRADE | DATE OF CAST | CURING REGIME | ELEMENT/ POSITION | AVERAGE 28 DAY STRENGTH (MPa) | Cubes Cured on site for 28 days | | | | In-situ cores | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | OPI (log value) | | Sorptivity(mm /√hr) | | OPI (log value) | | Sorptivity(mm /√hr) | |
| | | | | | Ave | CoV (%) | Ave | CoV (%) | Ave | CoV | Ave | CoV (%) |
| W30/19 | 28.08.2007 | Curing Compound | Deck 3 - culvert | 57.6 | 9.85 | 1.59 | 8.88 | 5.30 | *8.97* | - | 7.45 | - |
| W40/19 | 28.08.2007 | Curing Compound | Beam 5 | 57.6 | 10.21 | 1.37 | 6.97 | 10.36 | *9.02* | - | 8.79 | - |
| W40/19 | 28.08.2007 | Curing Compound | Beam 6 | 55.4 | 10.05 | 0.94 | 8.81 | 4.86 | *-* | - | - | - |
| W30/19 | 29.11.2007 | Curing Compound | Head 4 - Pier 1 | 54.5 | 9.72 | 1.60 | 7.19 | 8.70 | *9.15* | - | 7.96 | - |
| W40/19 | 10.09.2007 | Curing Compound | Beam 7 | 52 | *9.36* | 1.11 | 11.17 | 13.17 | - | - | - | - |
| Average | | | | | 9.78 | | 8.44 | | *9.17* | | 9.29 | |
| CoV (%) | | | | | 2.26 | | 14.39 | | 1.79 | | 16.85 | |

Note : Results shown in *RED italics* indicate values that have not met the target of > 9,7 for OPI and < 12 for sorptivity.

### 7.4.2 Black Mfolozi River Bridge

Additional cube samples were taken from the 81 precast beams that were cast on site as well as for all the substructure and in-situ decks casts. In addition, the edge beams on each of the 9 spans were cored at the beam yard and tested for OPI and sorptivity. Each of the 10 substructures were cored at the upstream and downstream ends at 1m above ground level and tested. A single location of the in-situ deck concrete was also tested on span 6. Figure 7-2 below shows the position of the core extractions for durability testing on the structure.

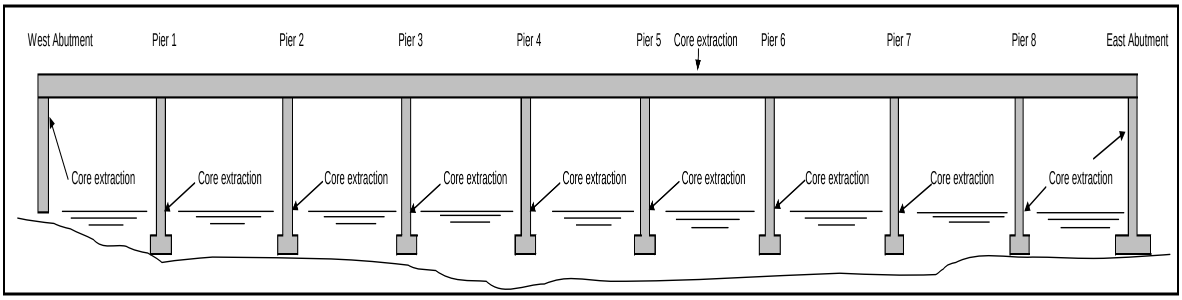



**Figure 7.2: Position of core extractions at Black Mfolozi River Bridge**

Coring of the substructures was done from river bed level during the period when the river level was still very low making access relatively simple. The abutments were cored at 2m above ground level and access was provided using conventional scaffolding. Table 7-7 below provides the summarised results from the testing carried out from both the test cubes as well as the in-situ coring. Full results are provided under Annexure 4.

**Table 7-7: Results of Durability Testing undertaken at Black Mfolozi Bridge**

| CONCRETE GRADE | DATE OF CAST | CURING REGIME | ELEMENT/ POSITION | AVERAGE 28 DAY STRENGTH (MPa) | Cubes Cured on site for 28 days | | | | In-situ cores | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | OPI (log value) | | Sorptivity(mm /√hr) | | OPI (log value) | | Sorptivity(mm /√hr) | |
| | | | | | Ave | CoV (%) | Ave | CoV (%) | Ave | CoV (%) | Ave | CoV (%) |
| W40/19 | 2007/11/15 | 24 h in mould, curing compound sprayed | Pre-Cast beam 34 | 54.1 | 10.51 | 1.14 | 3.62 | 11.33 | | | | |
| W40/19 | 2007/11/15 | 24 h in mould, curing compound sprayed | Pre-Cast beam 35 | 53.2 | 10.40 | 0.41 | 3.64 | 24.48 | | | | |
| W40/19 | 2007/11/19 | 24 h in mould, curing compound sprayed | Pre-Cast beam 36 | 59.8 | 10.04 | 1.97 | 5.18 | 2.18 | | | | |
| W40/19 | 2007/11/19 | 24 h in mould, curing compound sprayed | Pre-Cast beam 37 | 58.1 | 9.91 | 2.21 | 7.32 | 10.63 | 10.03 | 2.26 | 5.42 | 14.61 |
| W40/19 | 2007/11/24 | 24 h in mould, curing compound sprayed | Pre-Cast beam 38 | 56.6 | 10.46 | | 5.29 | 22.46 | | | | |
| W40/19 | 2007/11/24 | 24 h in mould, curing compound sprayed | Pre-Cast beam 39 | 65.0 | 10.32 | 4.32 | 6.49 | 13.29 | | | | |
| W40/19 | 2007/11/27 | 24 h in mould, curing compound sprayed | Pre-Cast beam 40 | 62.9 | 10.44 | 2.98 | 6.43 | 7.59 | | | | |
| W40/19 | 2007/11/29 | 24 h in mould, curing compound sprayed | Pre-Cast beam 42 | 69.1 | 10.60 | 2.74 | 5.30 | 7.34 | | | | |
| W40/19 | 2007/11/29 | 24 h in mould, curing compound sprayed | Pre-Cast beam 43 | 64.8 | 10.54 | 1.14 | 5.69 | 21.87 | | | | |
| W40/19 | 2007/12/04 | 24 h in mould, curing compound sprayed | Pre-Cast beam 44 | 57.2 | 10.00 | 1.20 | 5.48 | 8.65 | | | | |
| W40/19 | 2007/12/04 | 24 h in mould, curing compound sprayed | Pre-Cast beam 45 | 62.5 | 10.15 | 0.84 | 4.77 | 6.08 | 9.88 | | | |
| W40/19 | 2007/12/10 | 24 h in mould, curing compound sprayed | Pre-Cast beam 46 | 57.8 | 10.29 | 1.79 | 5.06 | 9.22 | 9.89 | 5.65 | 5.02 | 2.68 |
| W40/19 | 2007/12/10 | 24 h in mould, curing compound sprayed | Pre-Cast beam 47 | 62.2 | 10.13 | 1.19 | 5.57 | 33.42 | | | | |
| W40/19 | 2008/01/10 | 24 h in mould, curing compound sprayed | Pre-Cast beam 48 | 56.7 | 10.16 | 0.49 | 3.45 | 8.83 | | | | |
| W40/19 | 2008/01/10 | 24 h in mould, curing compound sprayed | Pre-Cast beam 49 | 56.0 | 10.27 | 2.55 | 3.98 | 4.62 | | | | |
| W40/19 | 2008/01/14 | 24 h in mould, curing compound sprayed | Pre-Cast beam 50 | 58.4 | 9.90 | 0.57 | 2.58 | 20.83 | *9.41* | 10.45 | 6.13 | 20.90 |
| W40/19 | 2008/01/14 | 24 h in mould, curing compound sprayed | Pre-Cast beam 51 | 52.1 | 9.82 | 4.54 | 3.79 | 47.39 | | | | |
| W40/19 | 2008/01/18 | 24 h in mould, curing compound sprayed | Pre-Cast beam 52 | 52.6 | 10.14 | 1.12 | 3.42 | 6.83 | | | | |
| W40/19 | 2008/01/18 | 24 h in mould, curing compound sprayed | Pre-Cast beam 53 | 53.3 | 10.08 | | 4.82 | 17.48 | | | | |
| W40/19 | 2008/01/23 | 24 h in mould, curing compound sprayed | Pre-Cast beam 54 | 50.5 | 10.18 | 0.49 | 3.17 | 5.35 | | | | |
| W40/19 | 2008/01/23 | 24 h in mould, curing compound sprayed | Pre-Cast beam 55 | 52.4 | 9.96 | 0.64 | 3.91 | 24.23 | | | | |
| W40/19 | 2008/01/28 | 24 h in mould, curing compound sprayed | Pre-Cast beam 56 | 53.2 | 10.41 | | 3.85 | 11.39 | 9.82 | | 5.89 | |
| W40/19 | 2008/01/28 | 24 h in mould, curing compound sprayed | Pre-Cast beam 57 | 52.3 | 10.30 | 1.78 | 3.17 | 9.81 | | | | |
| W40/19 | 2008/01/31 | 24 h in mould, curing compound sprayed | Pre-Cast beam 58 | 56.4 | *9.59* | 1.54 | 5.26 | 7.35 | | | | - |
| W40/19 | 2008/02/07 | 24 h in mould, curing compound sprayed | Pre-Cast beam 60 | 52.6 | 10.41 | 2.85 | 3.85 | 1.84 | | | | - |
| W40/19 | 2008/02/07 | 24 h in mould, curing compound sprayed | Pre-Cast beam 61 | 50.3 | 10.17 | 1.39 | 3.44 | 12.33 | | | | |
| W40/19 | 2008/02/11 | 24 h in mould, curing compound sprayed | Pre-Cast beam 62 | 53.2 | 9.98 | 1.56 | 4.31 | 19.55 | 10.10 | | 5.68 | |
| W40/19 | 2008/02/11 | 24 h in mould, curing compound sprayed | Pre-Cast beam 63 | 49.6 | 9.85 | | 5.70 | 13.15 | | | | - |
| W40/19 | 2008/02/13 | 24 h in mould, curing compound sprayed | Pre-Cast beam 64 | 51.3 | *9.41* | 0.53 | 6.83 | 9.32 | | | | - |
| W40/19 | 2008/02/13 | 24 h in mould, curing compound sprayed | Pre-Cast beam 65 | 44.9 | *9.56* | 0.59 | 4.00 | 26.02 | | | | - |
| W40/19 | 2008/02/18 | 24 h in mould, curing compound sprayed | Pre-Cast beam 66 | 52.0 | 9.71 | 1.17 | 4.56 | | | | | |
| W40/19 | 2008/02/18 | 24 h in mould, curing compound sprayed | Pre-Cast beam 66 | 52.0 | 9.73 | 2.18 | 4.32 | | | | | - |
| W40/19 | 2008/02/18 | 24 h in mould, curing compound sprayed | Pre-Cast beam 67 | 53.9 | 9.73 | 2.18 | 5.98 | | | | | - |
| W40/19 | 2008/02/21 | 24 h in mould, curing compound sprayed | Pre-Cast beam 68 | 51.3 | *9.27* | 2.20 | 5.22 | 7.38 | | | | - |
| W40/19 | 2008/02/21 | 24 h in mould, curing compound sprayed | Pre-Cast beam 69 | 46.3 | 10.35 | 2.60 | 4.12 | 7.55 | | | | - |
| W40/19 | 2008/02/25 | 24 h in mould, curing compound sprayed | Pre-Cast beam 70 | 52.1 | *9.35* | 2.42 | 4.76 | 6.10 | *9.54* | 2.00 | 5.09 | 14.32 |
| W40/19 | 2008/02/25 | 24 h in mould, curing compound sprayed | Pre-Cast beam 71 | 53.0 | *9.65* | 0.73 | 3.86 | 7.33 | | | | |
| W40/19 | 2008/02/27 | 24 h in mould, curing compound sprayed | Pre-Cast beam 73 | 58.5 | 9.94 | 1.07 | 4.15 | 3.75 | | | | |
| W40/19 | 2008/03/06 | 24 h in mould, curing compound sprayed | Pre-Cast beam 75 | 41.5 | 10.03 | 0.63 | 3.08 | 3.91 | | | | |
| W40/19 | 2008/03/12 | 24 h in mould, curing compound sprayed | Pre-Cast beam 76 | 46.0 | 10.24 | 0.55 | 3.62 | 4.30 | | | | |

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| W40/19 | 2008/03/13 | 24 h in mould, curing compound sprayed | Pre-Cast beam 77 | 47.5 | 10.09 | 2.31 | 4.02 | 10.55 | 9.91 | 2.28 | 6.50 | 0.98 |
| W40/19 | 2008/03/15 | 24 h in mould, curing compound sprayed | Pre-Cast beam 78 | 60.0 | 10.15 | 0.49 | 5.03 | 35.32 | | | | |
| W40/19 | 2008/03/15 | 24 h in mould, curing compound sprayed | Pre-Cast beam 79 | 47.1 | 10.31 | 1.78 | 4.79 | 13.88 | | | | |

## Table 7-7 *Continued*

| CONCRETE GRADE | DATE OF CAST | CURING REGIME | ELEMENT/ POSITION | AVERAGE 28 DAY STRENGTH (MPa) | Cubes Cured on site for 28 days | | | | In-situ cores | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | OPI (log value) | | Sorptivity(mm /√hr) | | OPI (log value) | | Sorptivity(mm /√hr) | |
| | | | | | Ave | CoV (%) | Ave | CoV (%) | Ave | CoV (%) | Ave | CoV (%) |
| W40/19 | 2008/03/28 | 24 h in mould, curing compound sprayed | Pre-Cast beam 80 | 50.8 | *9.66* | 0.73 | 5.54 | 6.52 | | | | |
| W40/19 | 2008/03/28 | 24 h in mould, curing compound sprayed | Pre-Cast beam 81 | 46.0 | 9.90 | 2.22 | 4.00 | 7.42 | | | | |
| W30/19 | 2007/11/12 | 24 h in mould, curing compound sprayed | West abutment wall | 57.4 | 10.03 | 0.55 | 6.21 | 15.84 | 9.96 | 3.30 | 5.05 | 1.96 |
| W30/19 | 25/11/2007 (Sunday) | 24 h in mould, curing compound sprayed | West abutment w/wall | 62.6 | 10.10 | 2.31 | 5.14 | 5.92 | 9.96 | 3.30 | 5.05 | 1.96 |
| W30/19 | 2007/11/27 | 24 h in mould, curing compound sprayed | West abutment upst w/wall | 60.7 | 10.34 | 0.49 | 4.32 | 2.95 | 9.96 | 3.30 | 5.05 | 1.96 |
| W30/19 | 2007/12/11 | 24 h in mould, curing compound sprayed | East abutment base | 50.5 | 10.45 | 1.78 | 4.39 | 16.27 | 9.96 | 3.30 | 3.89 | 1.96 |
| W30/19 | 2008/02/15 | 24 h in mould, curing compound sprayed | East abutment d/s w/wall | 51.7 | 9.96 | 1.69 | 5.78 | 14.97 | 9.96 | 3.30 | 3.89 | 1.96 |
| W30/19 | 2007/10/10 | 24 h in mould, curing compound sprayed | Pier 1 pier wall | 57.9 | | | | | *9.61* | 3.31 | 8.22 | 27.60 |
| W30/19 | 2007/11/23 | 24 h in mould, curing compound sprayed | Pier 2 pier wall | 50.3 | | | | | 9.82 | 2.02 | 8.30 | 29.00 |
| W30/19 | 2008/01/17 | 24 h in mould, curing compound sprayed | Pier 3 pier wall | 47.9 | *9.66* | 1.89 | 3.88 | 38.74 | *9.63* | 2.86 | 8.13 | 35.80 |
| W30/19 | 2008/07/17 | 24 h in mould, curing compound sprayed | Pier 4 pier wall | 40.5 | 9.86 | 4.87 | 6.02 | 32.47 | *9.30* | 2.77 | 10.50 | 6.71 |
| W30/19 | 2008/07/10 | 24 h in mould, curing compound sprayed | Pier 5 pier wall | 31.3 | 10.16 | 0.89 | 3.65 | 14.11 | *9.14* | 2.26 | 12.00 | 17.11 |
| W30/19 | 2008/06/06 | 24 h in mould, curing compound sprayed | Pier 6 pile cap | 32.0 | *9.68* | 0.88 | 4.69 | 23.52 | *9.13* | 5.65 | *13.54* | 17.73 |
| W30/19 | 2008/06/11 | 24 h in mould, curing compound sprayed | Pier 6 pier wall | 32.5 | *9.67* | 1.03 | 6.89 | 14.91 | *9.13* | 10.45 | *13.54* | 8.08 |
| W30/19 | 2008/05/28 | 24 h in mould, curing compound sprayed | Pier 7 pier wall | 30.7 | 9.73 | 1.13 | 4.59 | 13.84 | *8.86* | 4.28 | *13.64* | 8.08 |
| W30/19 | 2008/03/19 | 24 h in mould, curing compound sprayed | Pier 8 pier wall | 43.6 | 10.03 | 0.77 | 6.14 | 21.22 | *9.14* | 2.76 | 11.20 | 11.73 |
| W30/19 | 2008/01/22 | Mist spray + sand | Deck Span 1 | 49.6 | 10.13 | 1.93 | 4.12 | 14.25 | | | | |
| W30/19 | 2008/03/17 | Mist spray + sand | Deck Span 3 | 51.8 | 9.91 | 1.77 | 5.24 | 6.57 | | | | |
| W30/19 | 2008/08/12 | Mist spray + sand | Deck Span 5 | 50.7 | 9.77 | 2.94 | 6.76 | 7.67 | | | | |
| W30/19 | 2008/08/07 | Mist spray + sand | Deck Span 6 | 42.1 | *9.15* | 1.55 | 6.88 | 16.35 | *9.11* | 6.13 | 10.32 | 11.81 |
| W30/19 | 2008/08/04 | Mist spray + sand | Deck Span 7 | 39.0 | 9.86 | 3.16 | 8.75 | 22.88 | | | | |
| W30/19 | 2008/07/30 | Mist spray + sand | Deck Span 8 | 33.3 | *9.13* | 1.49 | 7.67 | 13.05 | | | | |
| W30/19 | 2008/04/24 | Mist spray + sand | Deck Span 9 | 33.6 | 9.92 | 1.31 | 5.62 | 10.61 | | | | |
| W40/19 | 2008/02/29 | in-situ, beneath ground | Pier 4 - pile P1 | 57.9 | *9.39* | 4.97 | 5.33 | 11.80 | | | | |
| W40/19 | 2008/02/25 | in-situ, beneath ground | Pier 4 - pile P2 | 39.3 | *9.31* | 15.27 | 5.24 | 20.13 | | | | |
| W40/19 | 2008/04/05 | in-situ, beneath ground | Pier 4 - pile P3 | 41.6 | 9.86 | 1.72 | 3.97 | 22.09 | | | | |
| W40/19 | 2008/03/05 | in-situ, beneath ground | Pier 5- pile P4 | 42.0 | 10.14 | 2.51 | 2.19 | 20.66 | | | | |
| W40/19 | 2008/03/03 | in-situ, beneath ground | Pier 5- pile P5 | 35.3 | *9.45* | 0.07 | 3.79 | 50.75 | | | | |
| W40/19 | 2008/03/12 | in-situ, beneath ground | Pier 6 - pile P7 | 43.3 | 9.78 | 1.37 | 4.28 | 9.91 | | | | |
| W40/19 | 2008/03/08 | in-situ, beneath ground | Pier 6 - pile P8 | 43.6 | 9.91 | 1.57 | 3.06 | 7.39 | | | | |
| W40/19 | 2008/03/17 | in-situ, beneath ground | Pier 7 - pile P11 | 48.6 | 10.71 | 0.13 | 3.02 | 2.81 | | | | |
| W40/19 | 2008/04/08 | in-situ, beneath ground | Pier 7 - pile P12 | 50.0 | *9.50* | 2.16 | 3.98 | 30.20 | | | | |
| Average | | | | | 9.97 | | 4.77 | | *9.62* | | 7.64 | |
| CoV (%) | | | | | 3.57 | | 26.60 | | 3.98 | | 47.35 | |

Note : Results shown in *RED italics* indicate values that have not met the target of > 9,7 for OPI and < 12 for sorptivity.

The results indicate that the site cured cube values are superior to the in-situ values. While many of the in-situ OPI values are below the target value of 9,7 (indicated in red), the results from the cubes are very close to the target or have passed. The sorptivity values have all passed both for the cubes and in-situ concrete.

### 7.4.3 Richmond road Interchange Bridge

For this project, the amended specifications required that both wet cured and site cured (air cured) samples be taken of all the elements cast. In addition, all the substructures were cored at approximately 2m above ground level and tested. Samples were also extracted from the deck pours where a single pour was done for the bottom slab and webs and one for the top slab. Figure 7-3 below shows the position of the core extractions for durability testing on the structure.

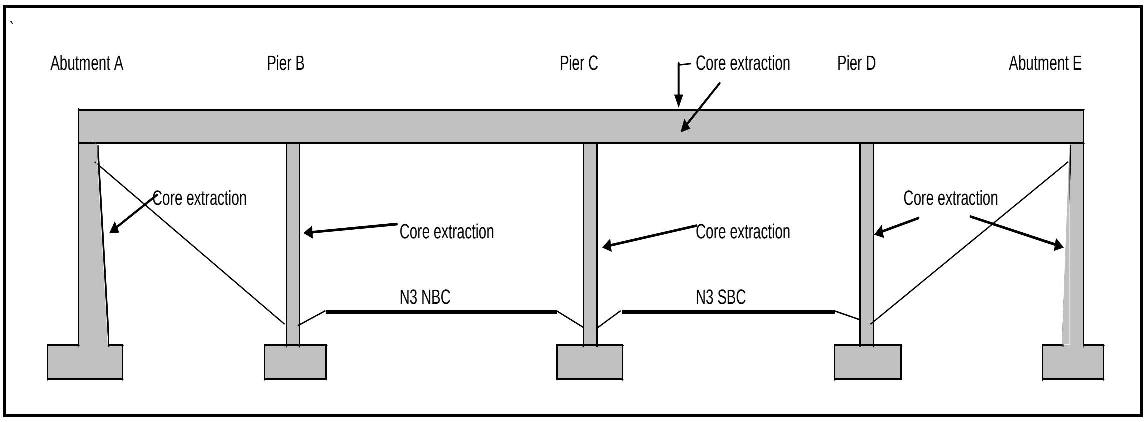



**Figure 7.3: Position of core extractions at Richmond Road Interchange Bridge**

Coring of the substructures was done using scaffolding that was set up. For the decks, the webs were cored while the scaffolding was still in place, while for the top slab, cores were extracted from the top of the deck. Table 7-8 provides the summarised results from the testing carried out from both the test cubes as well as the in-situ coring. Full determination and results are provided under Annexure 5.

The results indicate that the wet cured cube values are superior to the site cured cubes and the in-situ values. On this project, the results were very good, with all of the wet cured OPI values above 9,7, while there were only two results below 9,7 for the air cured cubes and three for the in situ below 9,7 (indicated in red italics). The sorptivity values have all passed both for the wet and air cured cubes as well as for the in-situ concrete.

## Table 7-8: Results of Durability Testing undertaken at Richmond Road Bridge

| CONCRETE GRADE | DATE OF CAST | CURING REGIME | ELEMENT/ POSITION | AVERAGE 28 DAY STRENGTH | Lab Wet(submerged) Cured for 28 days | | | | Cubes Cured on site (curing compound) for 28 days | | | | In-situ cores | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | OPI (log value) | | Sorptivity(mm/√hr) | | OPI (log value) | | Sorptivity(mm/√hr) | | OPI (log value) | | Sorptivity(mm/√hr) | |
| | | | | | Ave | CoV | Ave | CoV | Ave | CoV | Ave | CoV | Ave | CoV | Ave | CoV |
| W35/19 | 16.05.2008 | Curing Compound | A1 & A2 Bases | 46.8 | 10.67 | 3.93 | 3.63 | 6.84 | - | - | - | - | - | | - | |
| W35/19 | 20.05.2008 | Curing Compound | D1 Base | 43.7 | 10.29 | 3.51 | 4.29 | 18.89 | - | - | - | - | - | | - | |
| W35/19 | 20.05.2008 | Curing Compound | Trial Panel | 43.7 | 9.99 | 0.50 | 4.65 | 8.51 | - | - | - | - | - | | - | |
| W35/19 | 21.05.2008 | Curing Compound | D2 Base | 45.3 | 10.19 | 3.92 | 3.89 | 5.60 | - | - | - | - | - | | - | |
| W35/19 | 22.05.2008 | Curing Compound | B2 Base | 46.8 | 10.12 | 5.96 | 3.80 | 3.01 | - | - | - | - | - | | - | |
| W35/19 | 23.05.2008 | Curing Compound | BI Base | 43.1 | 10.23 | 2.76 | 4.75 | 8.12 | 9.90 | 0.92 | 6.87 | 4.14 | - | | - | |
| W35/19 | 26.05.2008 | Curing Compound | A1 Columns (1st Lift) | 45.9 | 10.14 | 4.70 | 4.15 | 14.29 | 9.89 | 1.90 | 8.02 | 20.12 | 9.93 | 7.32 | 6.42 | 11.16 |
| W35/19 | 26.05.2008 | Curing Compound | D1 Columns (1st Lift) | 45.9 | 10.14 | 4.70 | 4.15 | 14.29 | - | - | - | - | 9.82 | 2.57 | 7.68 | 15.20 |
| W35/19 | 27.05.2008 | Curing Compound | C2 Base | 45.9 | 9.87 | 1.43 | 5.11 | 5.49 | - | - | - | - | - | | - | |
| W35/19 | 28.05.2008 | Curing Compound | C1 Base & A2 Column (1st Lift) | 44.1 | 10.43 | 1.20 | 4.59 | 10.42 | - | - | - | - | 9.93 | 7.32 | 6.42 | 11.16 |
| W35/19 | 29.05.2008 | Curing Compound | D2 Column (1stLift) | 42.2 | 10.13 | 2.32 | 4.11 | 6.38 | - | - | - | - | 9.82 | 2.57 | 7.68 | 15.20 |
| W35/19 | 02.06.2008 | Curing Compound | B1 Column (1stLift) | 40.3 | 10.15 | 0.64 | 4.10 | 15.53 | 10.04 | 1.66 | 5.65 | 13.97 | *9.65* | 3.73 | 8.19 | 24.24 |
| W35/19 | 04.06.2008 | Curing Compound | C1 Column (1stLift) | 31.7 | 9.76 | 0.89 | 5.53 | 5.23 | - | - | - | - | *9.23* | 1.47 | 11.07 | 21.55 |
| W35/19 | 05.06.2008 | Curing Compound | Pier D - Wall | 43.2 | 10.48 | 1.00 | 5.20 | 10.77 | 10.48 | 1.00 | 5.20 | 10.77 | - | | - | |
| W35/19 | 06.06.2008 | Curing Compound | B2 Column (1stLift) | 29.7 | 10.04 | 1.66 | 5.65 | 13.97 | 10.04 | 1.66 | 5.65 | 13.97 | - | | - | |
| W35/19 | 10.06.2008 | Curing Compound | C2 Column (1stLift) | 39.5 | 10.43 | 1.34 | 5.63 | 16.56 | 9.90 | 0.92 | 6.87 | 4.14 | - | | - | |
| W35/19 | 11.06.2008 | Curing Compound | Abutment A-Crossbeam (1stLift) | 44.8 | 9.97 | 4.22 | 6.68 | 23.56 | 10.09 | 0.67 | 5.60 | 11.36 | - | | - | |
| W35/19 | 12.06.2008 | Curing Compound | B1 Column (2ndLift) | 37.4 | 9.86 | 4.60 | 6.46 | 22.97 | 10.10 | 7.39 | 9.71 | 1.15 | - | | - | |
| W35/19 | 18.06.2008 | Curing Compound | C1 Column (2ndLift) | 33.8 | 10.13 | 1.76 | 4.86 | 5.65 | 9.97 | 0.73 | 5.50 | 2.62 | - | | - | |
| W35/19 | 19.06.2008 | Curing Compound | Pier B - Wall | 35.5 | 10.28 | 3.01 | 4.78 | 7.69 | 9.93 | 0.47 | 7.01 | 19.37 | - | | - | |
| W35/19 | 23.06.2008 | Curing Compound | Abutment A - Curtain Wall | 43.5 | 11.09 | 4.16 | 3.95 | 20.90 | 10.33 | 0.42 | 4.17 | 3.85 | - | | - | |
| W35/19 | 25.06.2008 | Curing Compound | Pier C2 Column (2ndLift) | 36 | 10.12 | 1.80 | 4.87 | 13.33 | 10.21 | 1.37 | 5.04 | 10.51 | - | | - | |
| W35/19 | 26.06.2008 | Curing Compound | Base E2 | 32.3 | 10.16 | 2.13 | 4.39 | 16.00 | 9.88 | 10.87 | 4.75 | 27.79 | - | | - | |

Note : Results shown in *RED italics* indicate values that have not met the target of > 9,7 for OPI and < 12 for sorptivity.

**Table 7-8 : *Continued***

| CONCRETE GRADE | DATE OF CAST | CURING REGIME | ELEMENT/ POSITION | AVERAGE 28 DAY STRENGTH | Lab Wet(submerged) Cured for 28 days | | | | Cubes Cured on site (curing compound) for 28 days | | | | In-situ cores | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | OPI (log value) | | Sorptivity(mm/√hr) | | OPI (log value) | | Sorptivity(mm/√hr) | | OPI (log value) | | Sorptivity(mm/√hr) | |
| | | | | | Ave | CoV | Ave | CoV | Ave | CoV | Ave | CoV | Ave | CoV | Ave | CoV |
| W35/19 | 27.06.2008 | Curing Compound | B2 Column (2nd Lift) | 35 | 10.16 | 2.17 | 5.56 | 18.33 | 10.09 | 0.93 | 5.52 | 28.99 | - | | - | - |
| W35/19 | 02.07.2008 | Curing Compound | Pier E2 - Column (1stLift) | 26 | 10.78 | 1.14 | 3.89 | 5.25 | *9.65* | 1.18 | 10.51 | 23.31 | *9.54* | 2.20 | 9.82 | 5.55 |
| W35/19 | 02.07.2008 | Curing Compound | Pier B2 Column (3rdLift) | 36.8 | 10.46 | 3.25 | 4.49 | 3.01 | *9.61* | 1.69 | 8.89 | 18.67 | - | | - | - |
| W35/19 | 03.07.2008 | Curing Compound | Pier C Wall | 38.5 | 10.25 | 1.63 | 4.06 | 8.05 | 9.89 | 1.90 | 8.02 | 20.12 | - | | - | - |
| W45/19 | 25.09.2008 | Mist Spray | Deck-Bottom Slab/ webs | 52.4 | 10.69 | 1.92 | 4.13 | 15.75 | 10.84 | 0.39 | 3.80 | 8.76 | 10.83 | 0.90 | 3.59 | 19.69 |
| | | | | | 10.51 | 4.79 | 4.24 | 5.64 | 10.52 | 2.82 | 3.20 | 16.60 | 10.92 | 1.27 | 3.25 | 17.92 |
| | | | | | - | - | - | - | 10.60 | 1.94 | 3.32 | 30.08 | - | | - | - |
| | | | | | - | - | - | - | 10.37 | 1.57 | 4.68 | 13.76 | - | | - | - |
| | | | | | - | - | - | - | 10.19 | 0.35 | 6.50 | 30.46 | - | | - | - |
| W45/19 | 04.10.2008 | Mist Spray | Deck - Top Slab | 51.8 | 10.90 | 1.75 | 5.49 | 12.71 | 10.52 | 6.05 | 5.68 | 2.24 | 10.71 | 1.96 | 2.82 | 13.68 |
| | | | | | - | - | - | - | 10.36 | 7.85 | 5.68 | 8.10 | - | | - | - |
| | | | | | - | - | - | - | 11.19 | 5.18 | 6.55 | 8.10 | - | | - | - |
| | | | | | - | - | - | - | 10.67 | 1.99 | 4.35 | 23.73 | - | | - | - |
| | | | | | - | - | - | - | 10.74 | 4.87 | 5.24 | 16.46 | - | | - | - |
| | | | | | - | - | - | - | 11.54 | 1.29 | 7.54 | 12.29 | - | | - | - |
| Average | | | | | 10.28 | | 4.70 | | 10.27 | | 6.05 | | 10.04 | | 6.69 | |
| CoV (%) | | | | | 3.03 | | 16.63 | | 4.36 | | 29.91 | | 5.77 | | 41.60 | |

Note : Results shown in *RED italics* indicate values that have not met the target of > 9,7 for OPI and < 12 for sorptivity.

### 7.4.4 King Shaka International Airport Bridges

For this project, the amended specifications also required that both wet cured and site cured (air cured) samples be taken of all the elements cast. Due to this project still in the early stages of construction, limited testing has been undertaken thus far. For in-situ coring, these were only done on the pile-caps of the N2 Overpass Bridge and Ramp E Bridge, as shown in Figures 7-4 and 7-5 below.

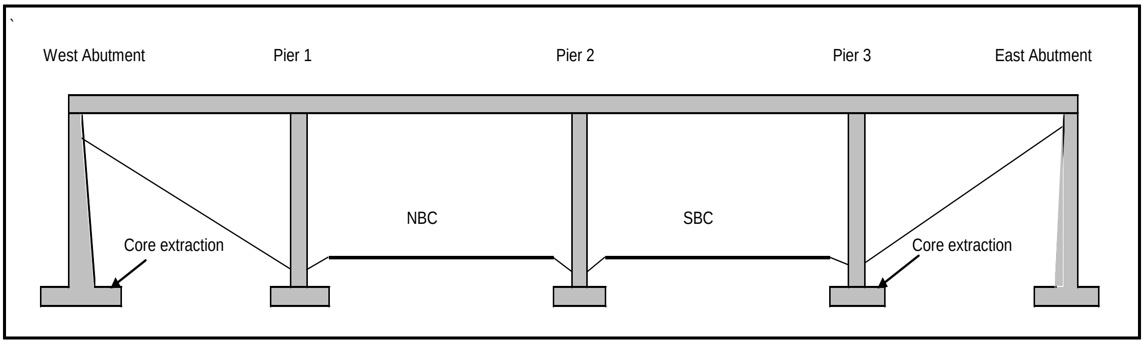



**Figure 7.4 : Positions of limited coring undertaken on the N2 Overpass Bridge**

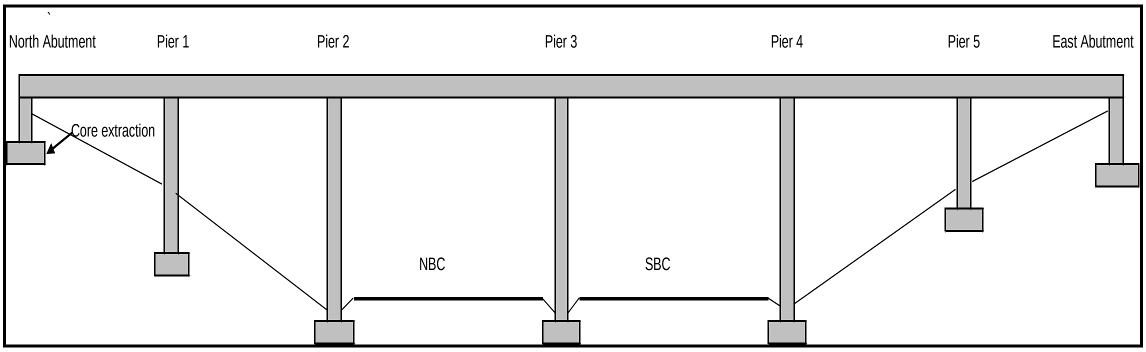



**Figure 7.5: Positions of limited coring undertaken on the N2 Ramp E Bridge**

Coring of the substructures was done at ground level. Although all of the substructures will be backfilled, these were the only elements available to be cored at the time. Further in-situ cores will be taken on the substructures and the decks under the project. Table 7-9 below provides the summarised results from the testing carried out from both the test cubes as well as the in-situ coring. Full determinations and results are provided under Annexure 6. Due to the limited results available, all the graphs plotted in the proceeding sections have been combined for the in-situ, site cured and wet cured cubes.

**Table 7-9: Results of Durability Testing undertaken at King Shaka Airport Bridges**

| CONCRETE GRADE | DATE OF CAST | CURING REGIME | ELEMENT/ POSITION | AVERAGE 28 DAY STRENGTH | Normal Wet Cure | | | | Cubes Cured on site for 28 days | | | | In-situ cores | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | OPI (log value) | | Sorptivity(mm/√hr) | | OPI (log value) | | Sorptivity(mm/√hr) | | OPI (log value) | | Sorptivity(mm/√hr) | |
| | | | | | Ave | CoV | Ave | CoV | Ave | CoV | Ave | CoV | Ave | CoV | Ave | CoV |
| W30/19 | 26.09.08 | Curing Compound | Overpass Bridge - Pier 3 Pilecap South | 50.6 | 10.22 | 3.36 | 4.47 | 20.14 | 10.01 | 2.13 | 8.28 | 16.77 | 9.88 | 3.68 | 6.01 | 4.72 |
| W30/19 | 26.09.08 | Curing Compound | Overpass Bridge - Pier 3 Pilecap North | 50.6 | 10.22 | 3.36 | 4.47 | 20.14 | 10.01 | 2.13 | 8.28 | 16.77 | *9.08* | 9.60 | 6.03 | 18.08 |
| W30/19 | 25.09.08 | Curing Compound | Overpass Bridge - West Abutment Pilecap | 50.5 | -- | -- | -- | -- | -- | -- | -- | -- | 10.12 | 4.36 | 6.80 | 17.23 |
| W30/19 | 28.10.08 | Curing Compound | Ramp E Bridge - North Abutment Pilecap | 48.8 | 10.13 | 3.91 | 5.91 | 45.50 | 9.82 | 1.12 | 4.43 | 21.40 | -- | -- | -- | -- |
| W30/19 | 09.12.08 | Curing Compound | Overpass Bridge - West Abutment | 45.95 | -- | -- | -- | -- | 10.20 | 1.33 | 5.01 | 21.82 | -- | -- | -- | -- |
| W30/19 | 21.01.09 | Curing Compound | Overpass Bridge - Pier 1 , first lift | 32.9 | 10.25 | 0.55 | 5.45 | 12.00 | 10.11 | 0.72 | 5.57 | 22.19 | *9.19* | 7.48 | 9.29 | 3.93 |
| W30/19 | 21.01.09 | Curing Compound | Overpass Bridge - Pier 1 , first lift | 32.9 | 10.25 | 0.55 | 5.45 | 12.00 | 10.11 | 0.72 | 5.57 | 22.19 | *8.86* | 15.98 | 8.82 | 1.23 |
| W30/19 | 15.10.08 | Curing Compound | Overpass Bridge - Pier 3 South | 50.3 | -- | -- | -- | -- | -- | -- | -- | -- | 10.08 | 3.75 | 6.19 | 24.03 |
| W30/19 | 10.10.08 | Curing Compound | Overpass Bridge - Pier 3 North | 42.4 | -- | -- | -- | -- | -- | -- | -- | -- | *9.64* | 2.33 | 8.04 | 7.63 |
| W30/19 | 02.02.2009 | Curing Compound | Ramp C Box Culvert - Panel No. 5 Base | 35.4 | 9.88 | 1.68 | 6.11 | 14.54 | -- | -- | -- | -- | -- | -- | -- | -- |
| W30/19 | 10.02.2009 | Curing Compound | Ramp C Box Culvert - Panel No. 2 Base | 45 | 9.54 | 2.49 | 6.66 | 15.02 | -- | -- | -- | -- | -- | -- | -- | -- |
| W30/19 | 24.02.2009 | Curing Compound | Ramp C Box Culvert - Panel No. 1 Base | -- | 9.45 | 2.32 | 6.01 | 12.37 | -- | -- | -- | -- | -- | -- | -- | -- |
| W40/19 | 18.02.2009 | Curing Compound | Ramp C Box Culvert - Panel No. 3 Walls and Deck | 43.7 | 9.17 | 2.49 | 5.84 | 10.41 | -- | -- | -- | -- | -- | -- | -- | -- |
| Average | | | | | 9.90 | | 5.60 | | 10.04 | | 6.19 | | *9.55* | | 7.31 | |
| CoV (%) | | | | | 4.18 | | 13.12 | | 1.32 | | 27.06 | | 5.32 | | 19.02 | |

*Note : Results shown in RED italics* indicate values that have not met the target of > 9,7 for OPI and < 12 for sorptivity.

## 7.5 Oxygen Permeability results

### 7.5.1 New England Road Interchange Bridge

The results in Table 7-6 shows that while majority of the results from the site cured cubes met the minimum target, the core results drilled from the structure has not met the minimum requirement for all the cores drilled. In-situ cores were drilled at three different locations and all the results proved unsuccessful. Noting that all tests were done on the same batch of concrete, the results of cores from the site air cured test cubes were superior to the in-situ results. The scatter diagram in Figure 7-6 below shows the relationship between in-situ and site (air) cured test cube results.

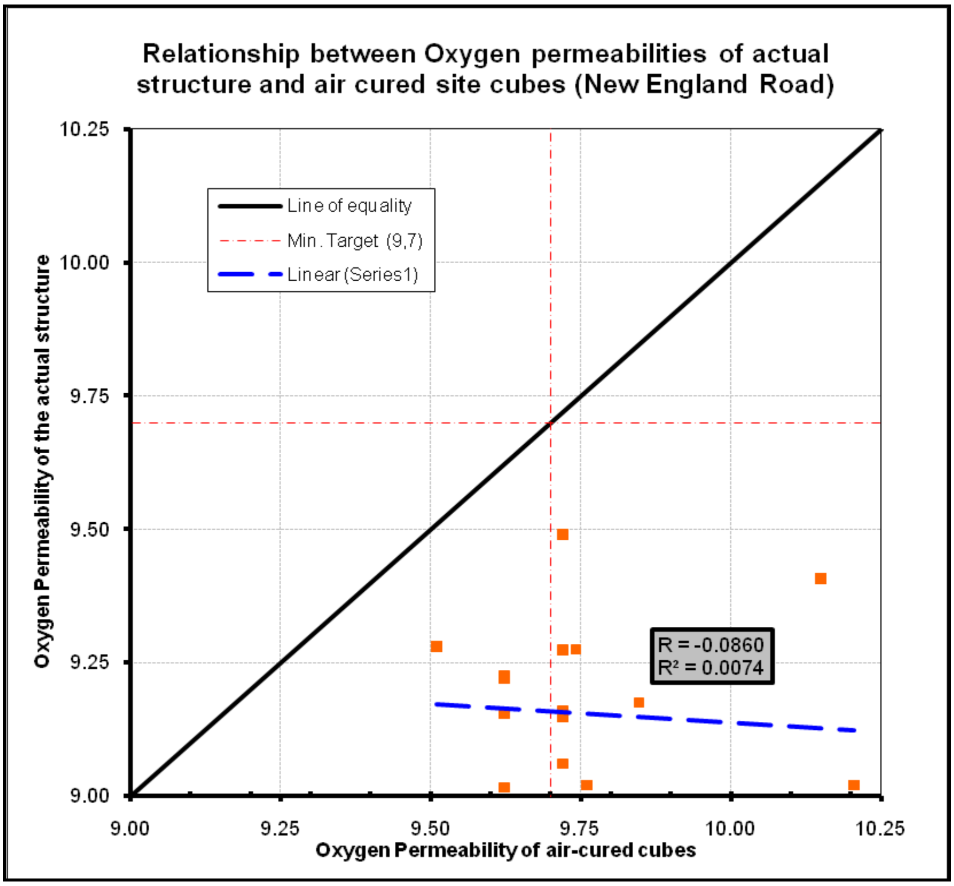
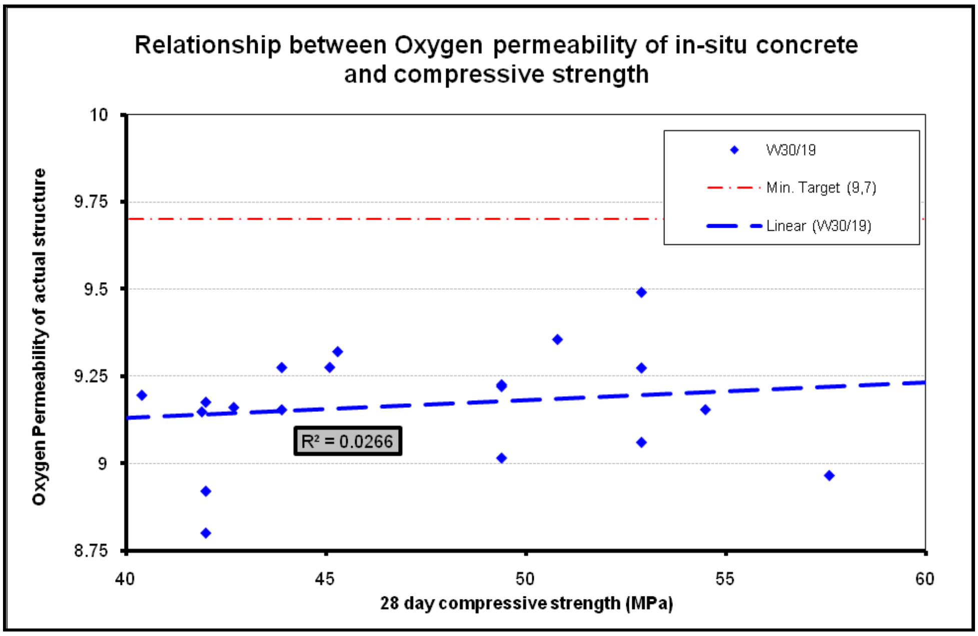


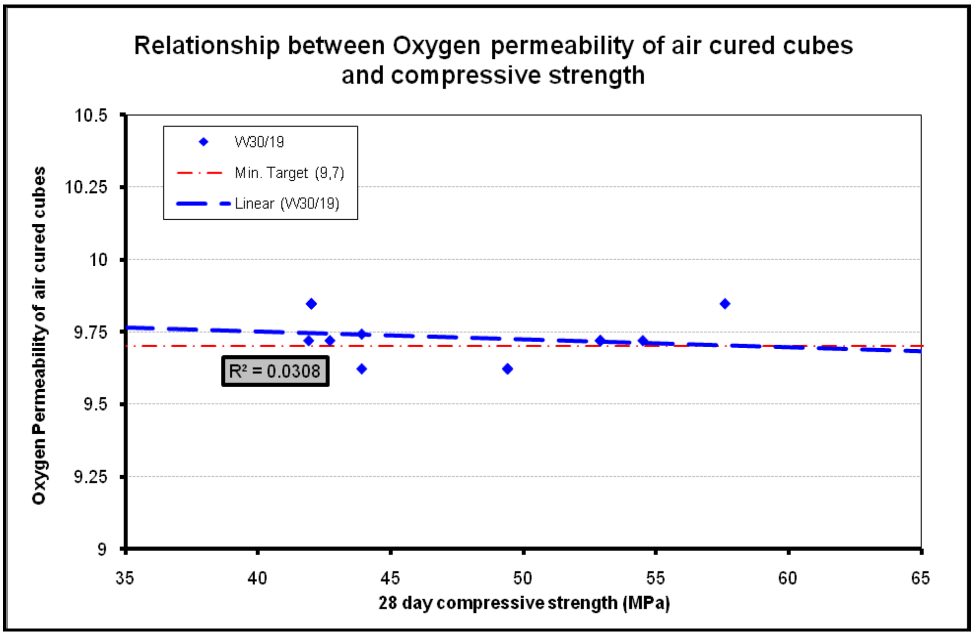

**Figure 7.6 : Relationship of OPI for in-situ versus site (air) cured cube results**

The figure indicates that none of the results fall along the line of equality. All of the results are below the line of equality, meaning that the results are higher for the test cubes than the in-situ. All of the results are below the 9,7 min target line for the in-situ results (horizontal line), while for the site cured cubes, majority of the results are above the min target line of 9,7 (vertical line). The low values of ‘r’ and ‘r2’ of 0,0917 and

0,0084 respectively is an indication that no correlation exists between site and in- situ results. On interrogation it was found that the concrete at this interchange was made with poor quality aggregate and voids in the cores extracted indicated poor compaction. These factors could have influenced in-situ results. The scatter diagram in Figure 7-7 (a) and (b) below shows the relationship between OPI and strength results.




**(a) In situ**




**(b) Site cured cubes**

**Figure 7.7 (a) & (b) : Relationship of OPI versus strength for in-situ and cubes**

The results in Figure 7.7(a) indicates that the in-situ values are all below the minimum target value of 9,7 for the concrete. The results however from the site cured cubes in Figure 7.7 (b) shows that the results are very close to the minimum requirement and majority of the results are higher which indicates that the concrete has met the target value. With regard to relationships of OPI and strength for in-situ concrete, the linear trend line is nearly horizontal indicating large values of compressive strength have little effect on the OPI value. The ‘r-squared’ values are very close to zero indicating no correlation between both these criteria.

### 7.5.2 Black Mfolozi River Bridge

The results in Table 7-7 above shows that majority of the results from the site cured cubes met the minimum target (56 out of 72 sample lots). The core results drilled from the structure indicated on certain of the elements similar results as the cube results. The value of the test result was however superior on the test cubes than the in-situ concrete. There were also many failed results from the in-situ concrete.

The scatter diagram in Figure 7-8 below shows the relationship between in-situ and test cube results for OPI. The diagram indicates that majority of the results do not fall along the line of equality. Majority of the results are below the line, indicating that the results are higher for the cubes than in-situ. There is a equal spread of results above and below the 9,7 min target line for the in-situ results (horizontal line), while for the site cured cubes, majority of the results lie above the min target line of 9,7 (vertical line). For the OPI values when comparing both the cured cubes and in-situ, the ‘r’ correlation value is 0.0173, and ‘r2’ is 0,0003. The values are again very close to zero indicating a poor correlation.

The scatter diagrams in Figure 7-9 (a) and (b) below shows the relationship between OPI and strength results. The results indicate that the in-situ results are spread on either side of the minimum target value of 9,7 for in-situ concrete. The results from the site cured cubes show that majority of the values are very much higher than the minimum requirement indicating the concrete has met the target value. With regard to relationships of OPI and strength, the linear trend line indicates increasing OPI values with increasing strength for both the in-situ values and air cured cubes. while the slopes of the trend lines show a relationship between OPI and strength, the ‘r’ and ‘r2’ values are low and therefore also indicative that durability is not related to strength.

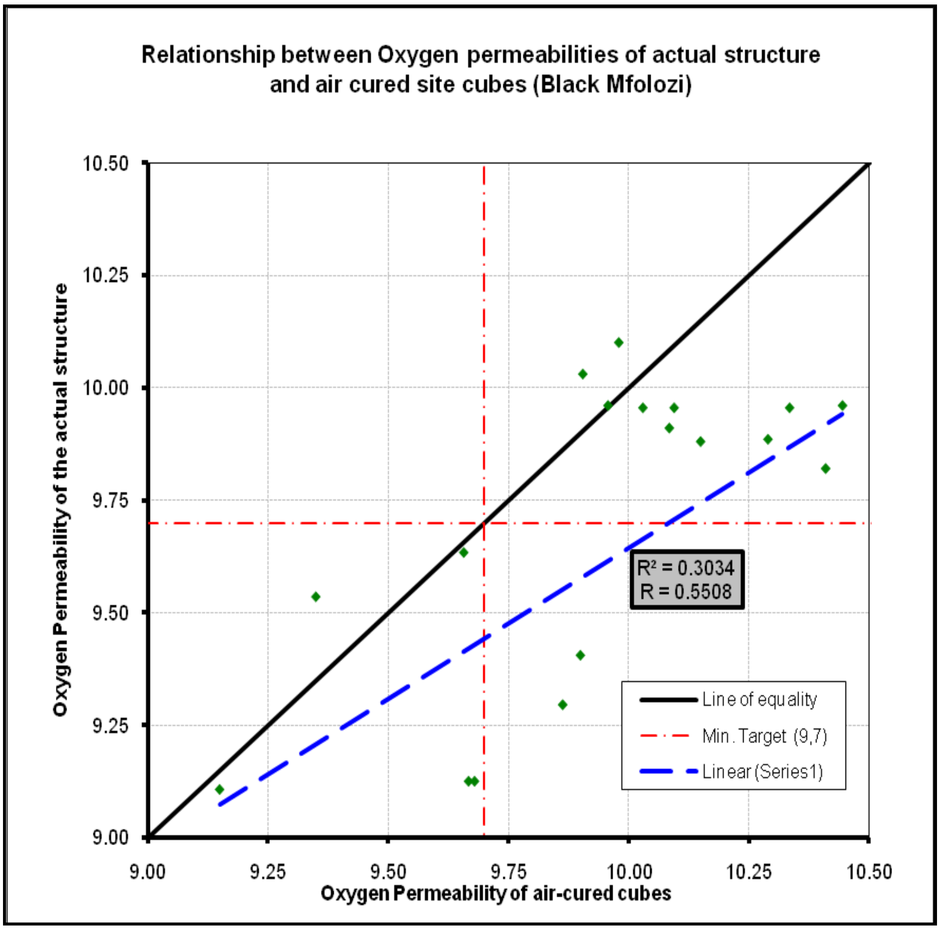
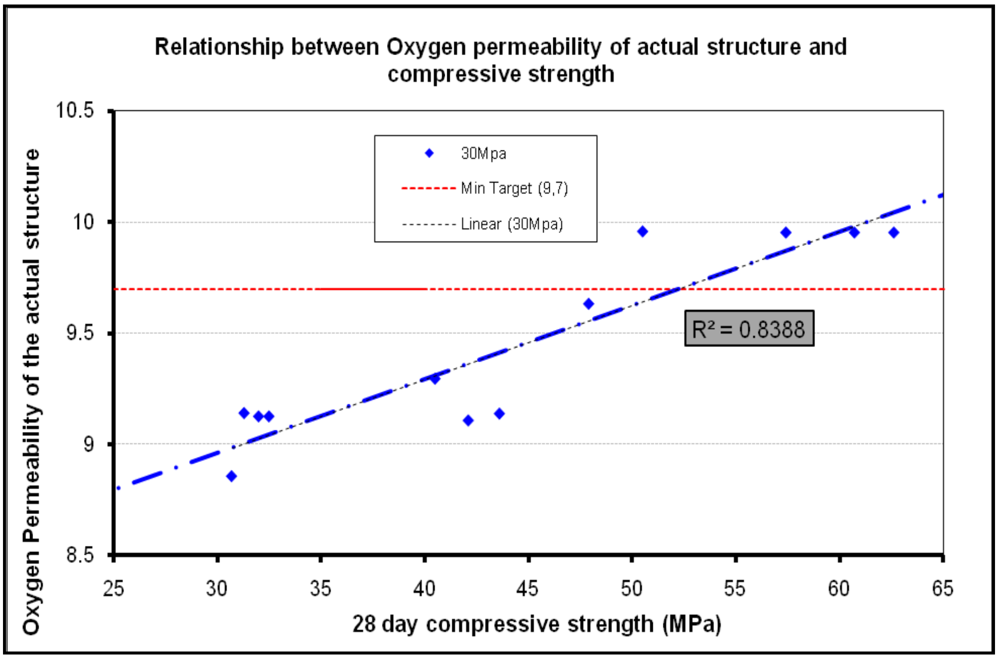
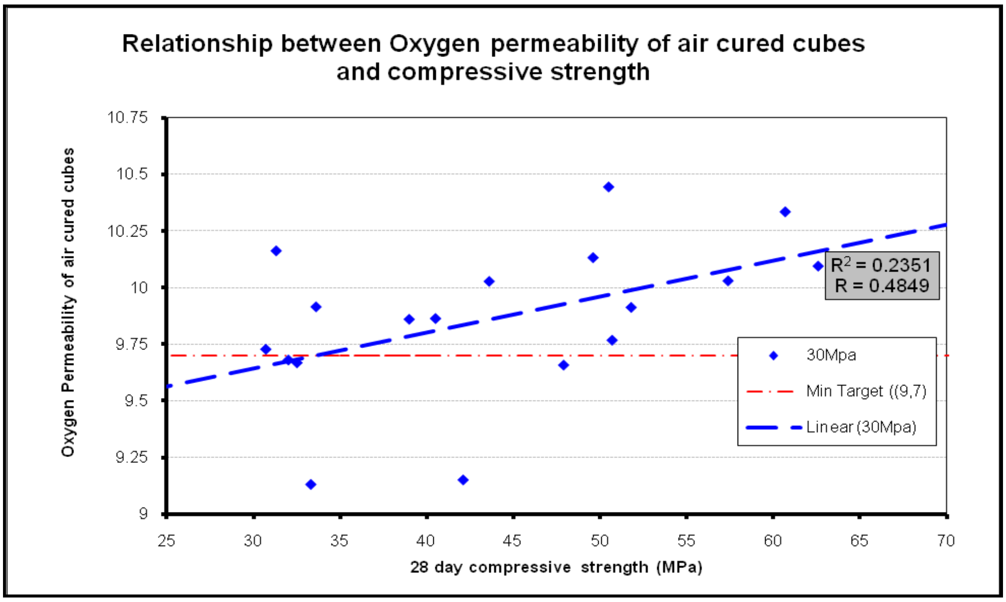



**Figure 7.8: Relationship of OPI for in-situ versus site (air) cured cube results**




**(a) In situ**

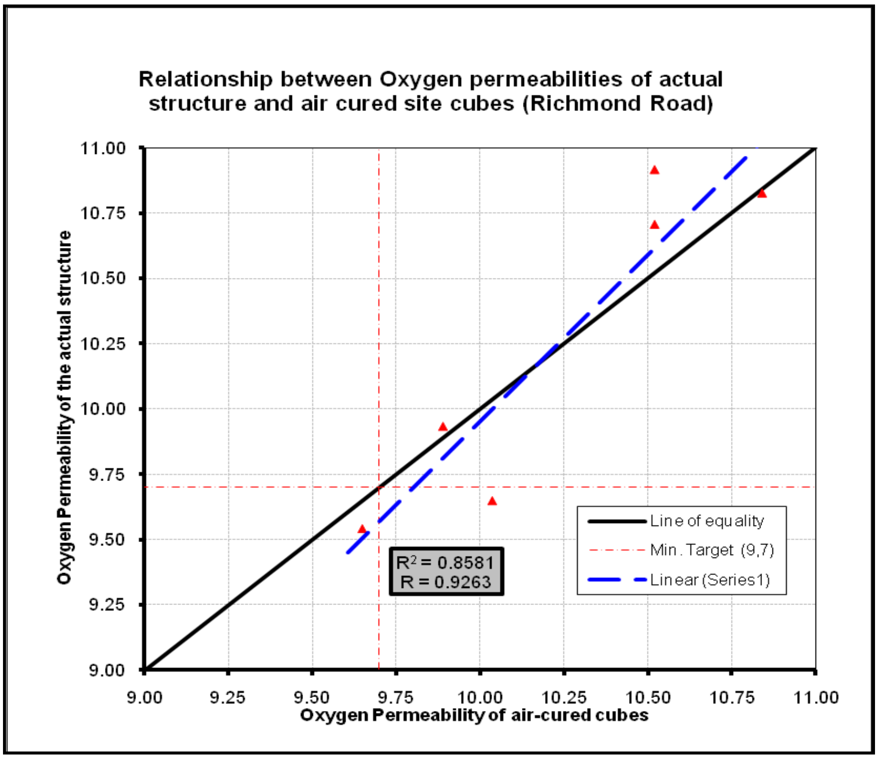



**(b) Site cured cubes**

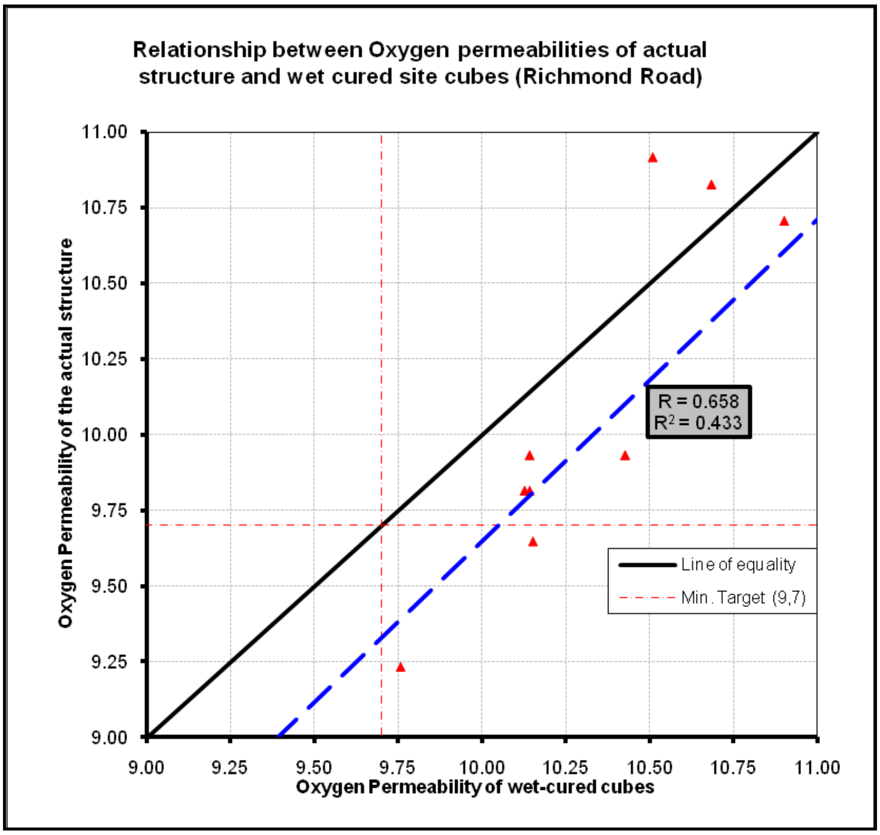
**Figure 7.9 (a) and (b) : Relationship of OPI versus strength for both in-situ and site cured cubes**

### 7.5.3 Richmond road Interchange Bridge

The results in Table 7-8 shows that all of the OPI results from laboratory cured cubes met the minimum target, while two results failed for the piers for the air cured cubes. The core results drilled from the structure however indicated failure on three of the ten samples tested. The values of the test results were superior on both the wet and air cured cube results than the in-situ concrete. The scatter diagrams in Figure 7-10 (a) and (b) shows the relationship between in-situ and both air cured and wet cured test cube results for OPI.




(a) Air cured cubes



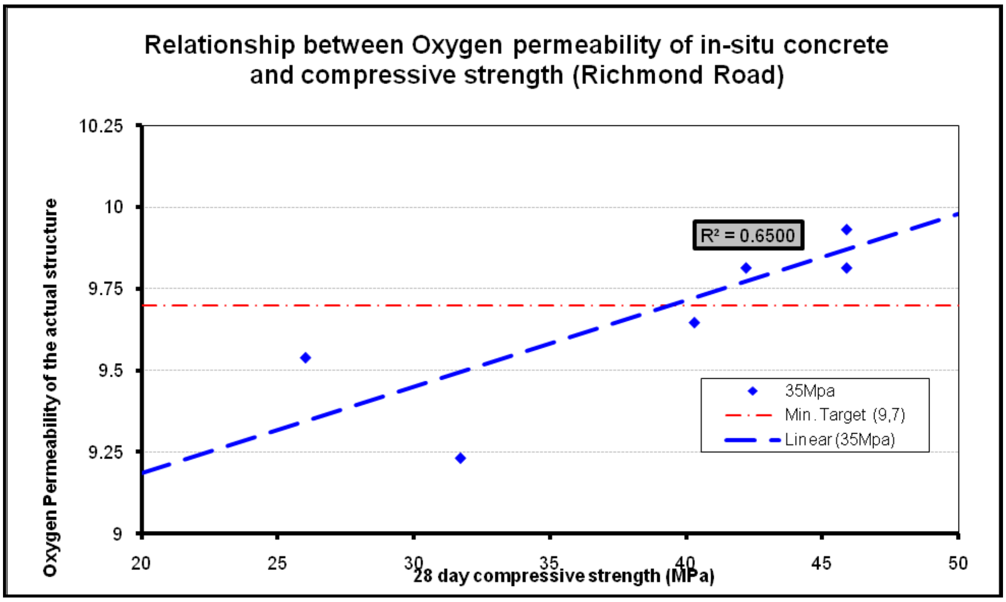

(b) Wet cured cubes

**Figure 7.10 (a) & (b) : Relationship of OPI for in-situ versus site cured / wet cured cubes**

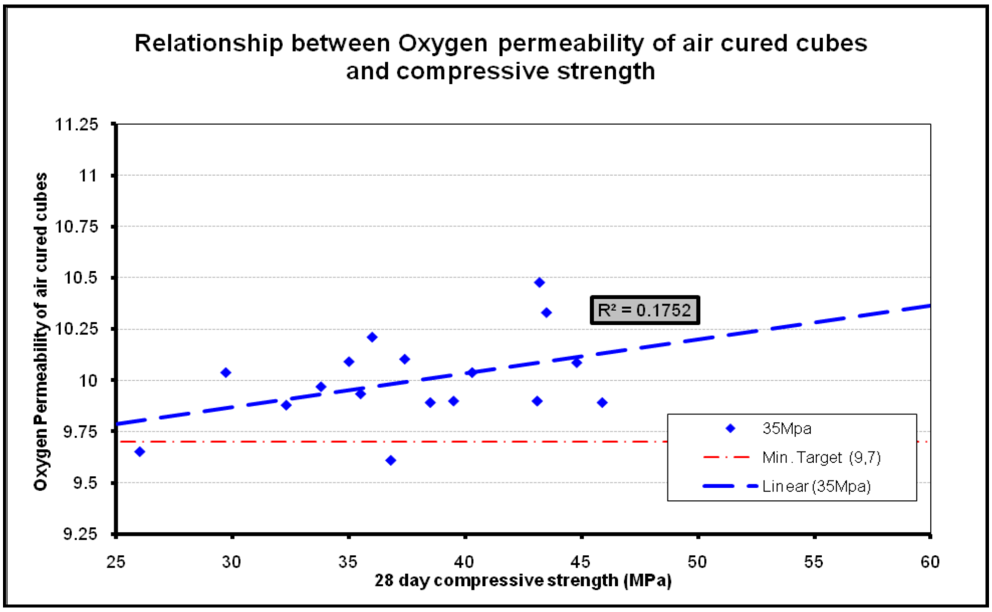
The diagram in Figure 7-10 (a) as well as Table 7-7 shows similar test results from the in-situ concrete and air cured cubes, while for the wet cured cubes (Figure 7-10(b)), the results for the cubes were superior that the in-situ results.

While all of the cube results (both air and wet cured) showed the OPI target being achieved, three of the ten in-situ results (30%) showed failure. When comparing the air cured cubes and in-situ, the ‘r’ correlation value is 0.9263 and ‘$r^2$’ is 0.8581 and for the wet cured cubes, the ‘r’ correlation value is 0.6580, and ‘$r^2$’ is 0,4330. While these values indicate a possible trend, the values for the air cured cubes are superior to the wet cured cubes which indicate that the air cured cubes are closer related to the in-situ values.

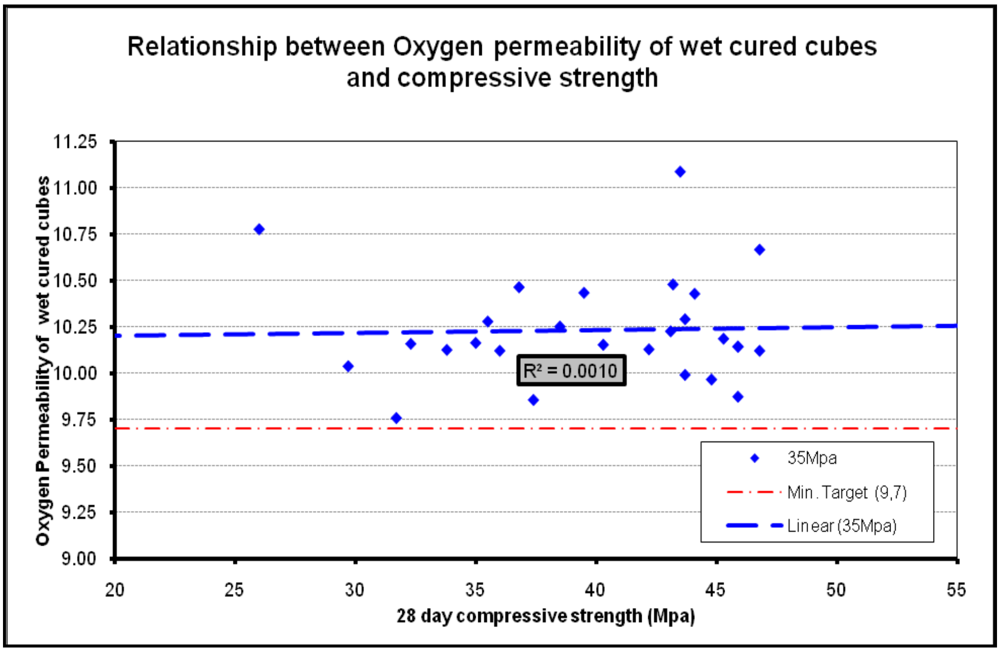
The scatter diagrams in Figure 7-11 (a), (b) and (c) below shows the relationship between OPI and strength results.




**(a) In-situ**




**(b) Air cured cubes**




**(c) Wet cured cubes**

**Figure 7.11 (a), (b) and (c) : Relationship of OPI versus strength for in-situ, site cured and wet cured cubes**

The diagrams indicate that the in-situ results (Figure 7-11(a)) are spread on either side of the minimum target value of 9,7 for in-situ concrete. The results from the air cured cubes show that majority of the results are above the minimum requirement indicating the concrete has met the target value. For the wet cured cubes, the results are above the minimum requirement, and are the highest of all three type test results. With regard to relationships of OPI and strength for the 35MPa concrete, the linear regression trend lines are near horizontal with the ‘r-square’ value being close to zero except for the in-situ results where the line is steeper, which is an indicator of the variability of the OPI results for the in-situ concrete. Due to the limited test values for the 45MPa concrete no trend lines have been drawn.

### 7.5.4 King Shaka International Airport Bridges

The results in Table 7-9 above shows that all of the tests from the site and laboratory cured cubes met the minimum target. The core results drilled from the sub-structures however indicated failure on one of the three samples tested. In addition, the value of the test results was superior on the air cured cube results than the in-situ concrete. The scatter diagram in Figure 7-12 shows the relationship between in-situ and test cube results for OPI.

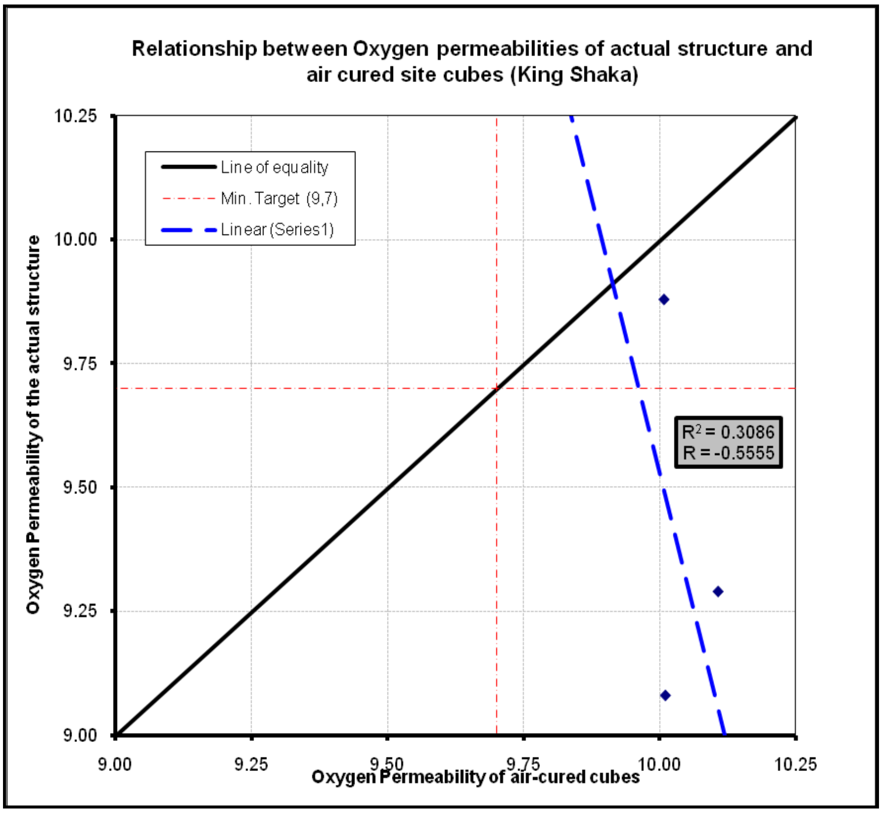



**Figure 7.12: Relationship of OPI for in-situ versus site (air) cured / wet cured cubes**

As can be seen from the diagram in Figure 7-12 there is only limited results due to late commencement of this project. However the graph shows the results being below the line of unity which indicates higher values for the cubes than in-situ. The values of ‘r’ and 'r[2]' are 0,5555 and 0,3086 respectively shows higher values than the other projects but these are only based on a limited number of samples. The scatter diagram in Figure 7-13 below shows the relationship between OPI (air cured cubes) and strength results.

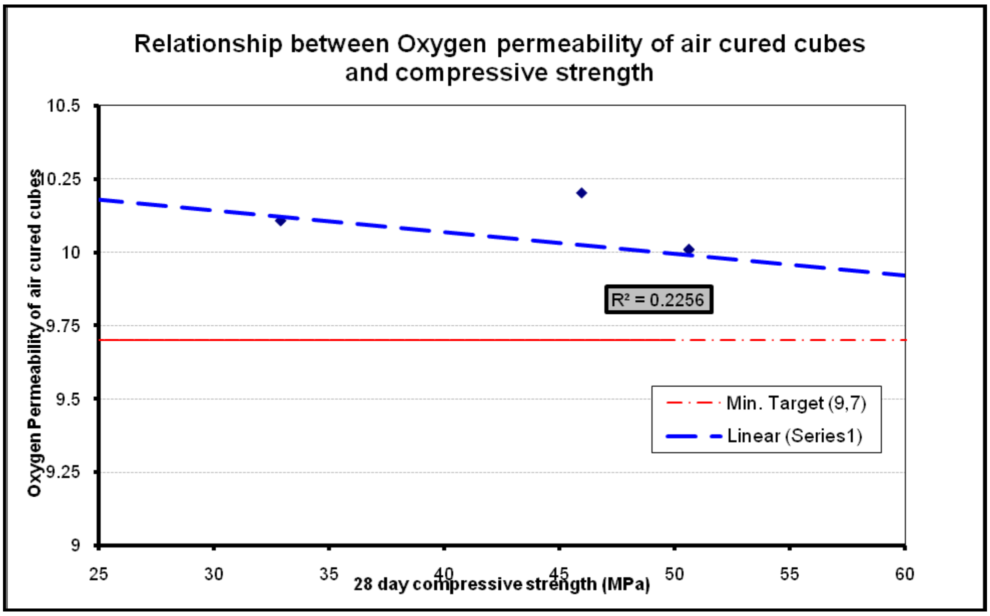
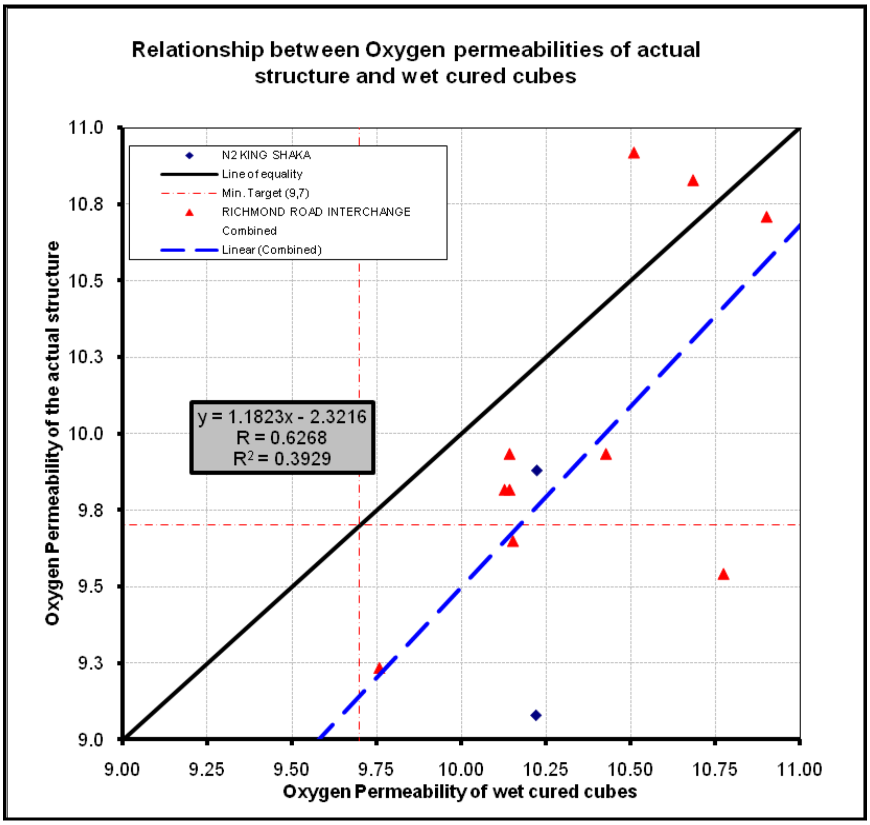



**Figure 7.13: Relationship of OPI versus strength for in-situ and , site cured and wet cured cubes**

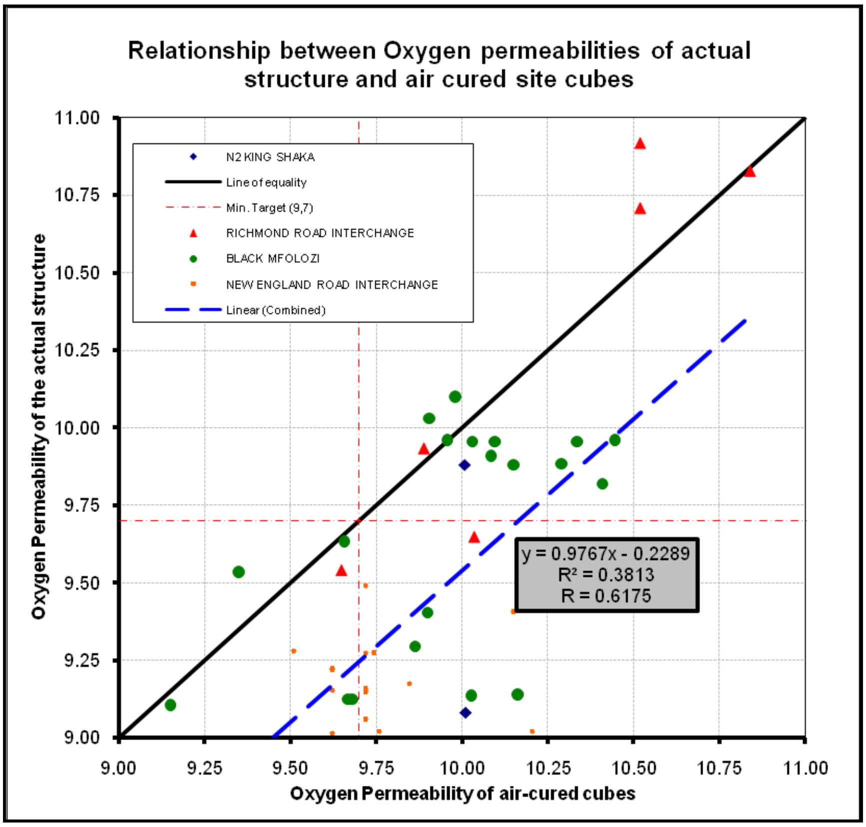
While there are only limited number of test results plotted on the above graphs, the results show that the air cured cube results are above the minimum target value of 9,7. With regard to relationships of OPI and strength, there are too few results to obtain a clear indication of any relationship and therefore the trend lines plotted cannot be used for this purpose.

### 7.5.5 Combined Project cube results

The results from each of the projects for the oxygen permeability tests were combined into common scatter diagrams to examine the overall trend for the in-situ and test cube results. These were plotted and shown in the scatter diagrams of Figure 7-14 (a) and (b) below.




**(a) Wet cured cubes**




**(b) Air cured cubes**

**Figure 7.14 (a) & (b) : Relationship of OPI for in-situ versus air cured / wet cured cubes**

From the diagrams, a trend for each type of curing is evident. For the wet cured cubes, the resulting linear correlation line equation is:

$Y = 1.182X – 2.322$.....................................................................8.1

Where;

Y = oxygen permeability of the structure, and

X = oxygen permeability of wet cured cubes.

For the site cured cubes, the equation is:

$Y = 0.9767X – 0.2289$.....................................................................8.2

Where;

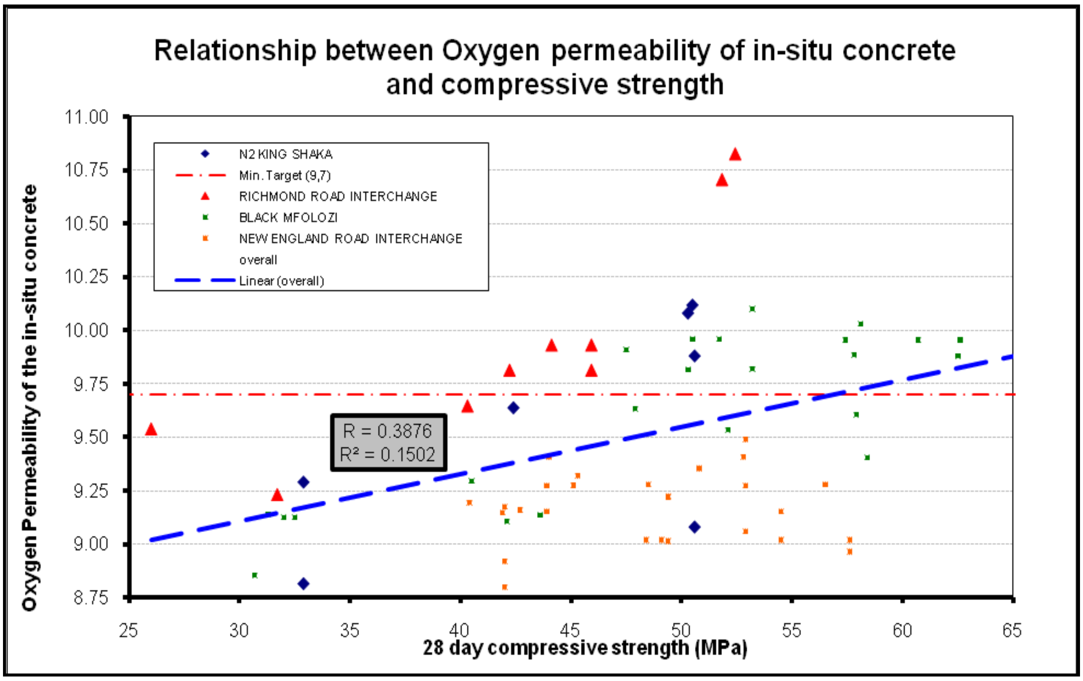
Y = oxygen permeability of the structure, and

X = oxygen permeability of air cured cubes.

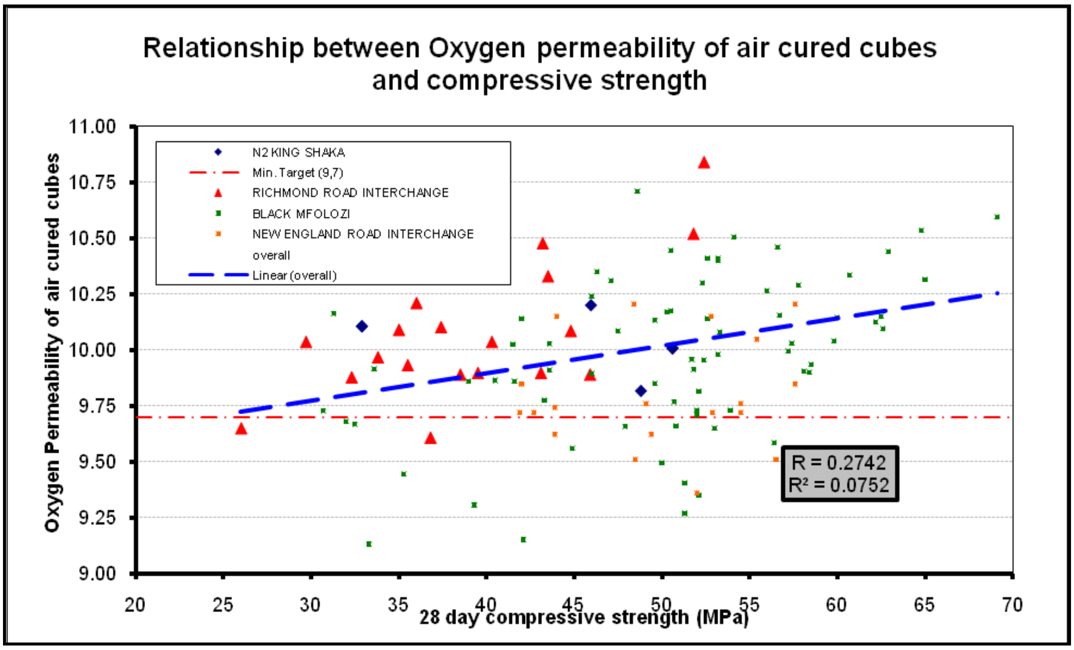
The wet cured cubes indicate a better correlation with an ‘r’ value 0.63, while the air cured cube value is 0.11. It must be noted that the wet cured cube results is from two of the projects only with limited results from the King Shaka Bridge site. With regard to the air cured cube results, all of the projects also showed a reasonable correlation and therefore the combined project results also show a similar trend i.e. an average correlation. Both the slope of wet and air cured cubes is near parallel to the line of equality.

The scatter diagrams in Figure 7-15 (a), (b) and (c) below shows the overall relationship between OPI and strength results.

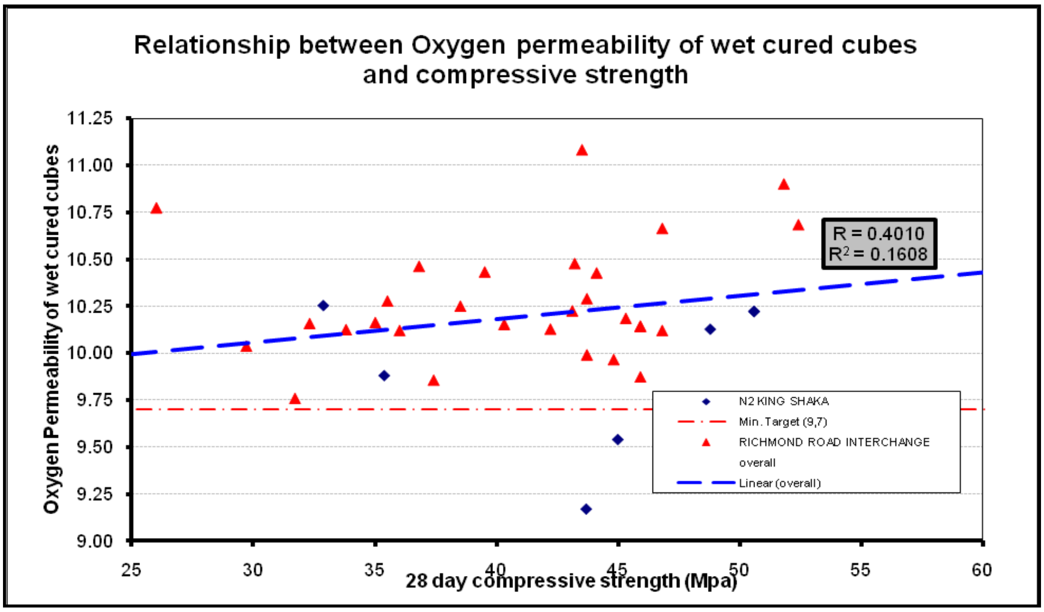
From Figure 7-15 (a) for the in-situ tests, it can be seen that the results are distributed on either side of the minimum target of 9,7, while Figure 7-15 (b) shows majority of the results are above the minimum target. Figure 7-15 (c) shows except for two values, all of the results are above the minimum target line. The value of ‘r-square’ is very low and close to zero indicating a very poor correlation of strength and OPI.




(a) In-situ




(b) Air cured cubes




**(c) Wet cured cubes**

**Figure 7.15 (a), (b) and (c) : Relationship of OPI versus strength for in-situ, air cured and wet cured cubes**

### 7.5.6 Combined Project trial panel results

As discussed under the previous sections, the results of the trial panels for each of the projects were provided in the relevant tables. While the requirement of the specifications was that trial panels be constructed and tested before any construction commences, on two of the projects they were constructed with the same concrete used specifically for certain of the bridge elements. These projects were the King Shaka Airport bridges and the Richmond Road Interchange.

For the Mgeni Interchange Bridges, trial panels were made during the casting of the various decks and this gave a good sample size. A correlation was therefore made of the concrete in the structure and that in the panels. Table 7-11 below provides the test results, while the scatter diagram in Figure 7-16 shows the relationship for oxygen permeability.

**Table 7-10: Oxygen Permeability Results for Trial panels and in-situ concrete**

| Structure | Member | Oxygen Permeability (>9,7) | |
|---|---|---|---|
| | | In-situ | Trial panel |
| King Shaka Bridges | Substructures - Vertical | 10.02 | 9.88 |
| | | 10.12 | 10.02 |
| Richmond Road Bridge | Deck-Bottom Slab/ webs | 10.83 | 10.53 |
| | | 10.92 | 10.53 |
| | Substructures (vertical) | 10.14 | 9.99 |
| Mgeni River Bridges | Median span 2 | 10.68 | 10.30 |
| | Median span 2 | 10.44 | 10.30 |
| | NBC Span 1 | 10.82 | 10.77 |
| | NBC Span 1 | 10.77 | 10.36 |

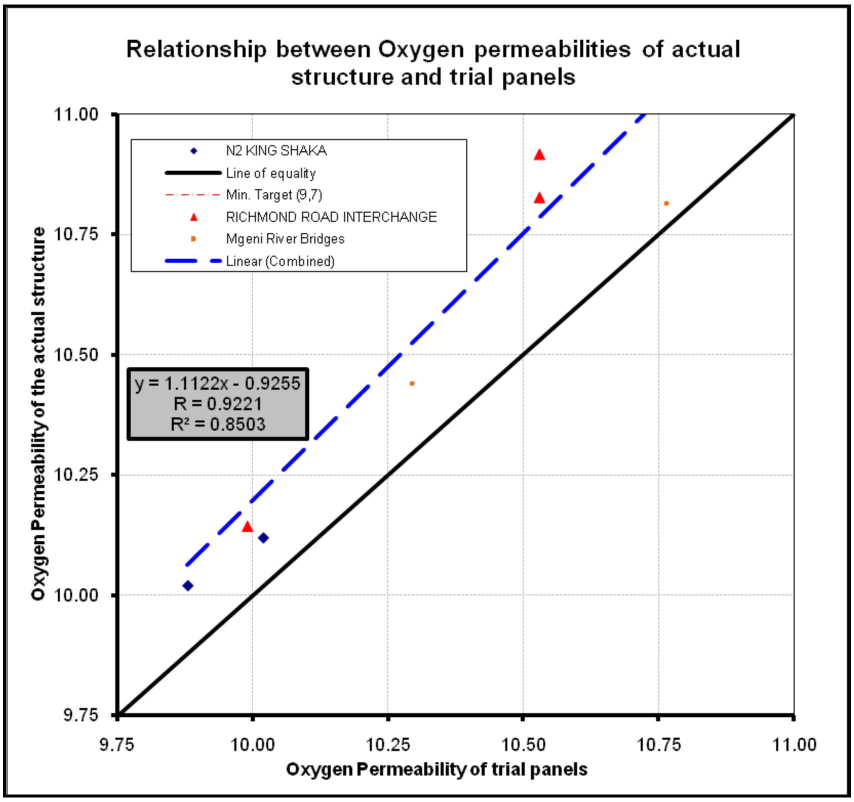



**Figure 7.16: Relationship of OPI for trial panels and in-situ concrete**

From Figure 7.16 for OPI, the resulting linear correlation line equation is:

Y = 1.1122X - 0.9255.....................................................................8.3

Where;

Y = oxygen permeability of the structure, and

X = oxygen permeability of trial panels.

The linear regression line shown in Figure 7.16 closely follows the line of equality and shows a very good correlation with a 'r' value of 0.9221. It should be noted that while it could be argued that correlations should have been done with the Darcy k values, the log values used for OPI were correlated as these are the values generally reported.

## 7.6 Water Sorptivity results

### 7.6.1 New England Road Interchange Bridge

The results in Table 7-6 shows that while the sorptivity target has been met in the test cube and in-situ results, the in-situ results are closer to the target requirement of 12,00. The test cube results are superior to the in-situ results. The scatter diagrams in Figure 7-17 shows the relationship between in-situ and test cube results for both strength concretes.

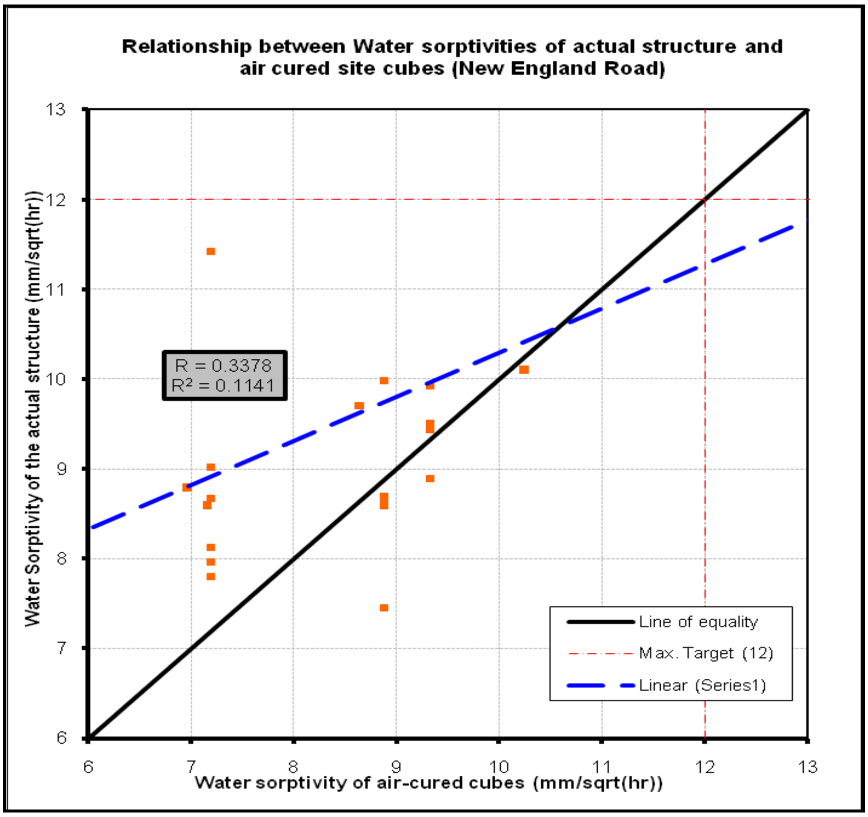
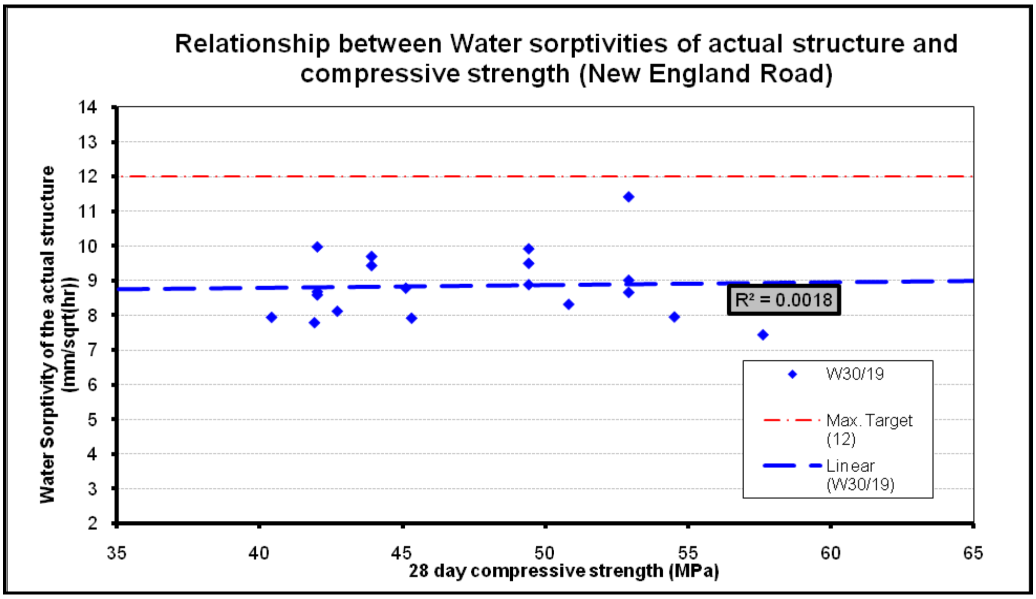


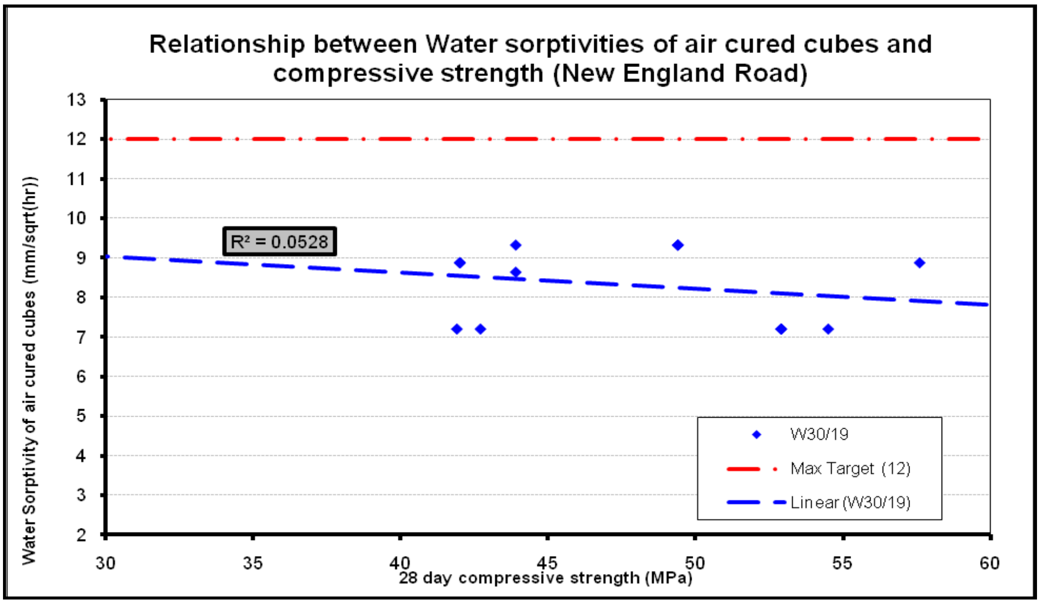

**Figure 7.17: Relationship of Sorptivity for in-situ versus site (air) cured cube results**

The diagram indicates similar results for the in-situ and site cured cubes, with majority of the results being below the maximum specified value of 12,00. The higher values of 'r' and 'r2' of 0,3378 and 0,1141 from the linear regression is an indication that a correlation exists between site cured cubes and in-situ results, although a weak correlation.

The scatter diagrams in Figure 7-18 (a) and (b) below shows the relationship between sorptivity and strength results.




**(a) In-situ**




**(b) Site cured cubes**

**Figure 7.18 (a) & (b) : Relationship of Sorptivity versus strength for both in-situ and site cured cubes**

The results indicate that both the in-situ (except two results) and site (air) cured results are all below the maximum target value of 12,00. With regard to the relationship of sorptivity and strength, the in-situ trend line indicates increasing sorptivity values with increasing strength, except for the 40MPa site cured cube results which indicate decreasing sorptivity values with increasing strength. The gradients of the lines are nearly horizontal, with the 'r-squared' values are very close to zero indicating no correlation between strength and sorptivity. Therefore the scatter and variability of the OPI and sorptivity results indicates that no relationship can be drawn between strength and durability and confirms the conclusions of Gouws et al (2001) that durability is not related to strength.

### 7.6.2 Black Mfolozi River Bridge

The results in Table 7-7 shows that while the sorptivity target has been met in both the test cube and in-situ results (except for three sample lots), the in-situ results are closer to the target requirement i.e. the in-situ results are higher than the cube results. The test cube results are therefore superior to the in-situ results, and can be attributed to the degree of curing and possibly the volume of concrete being compacted in the cube compared to that in the structure. The scatter diagram in Figure 7-19 shows the relationship between in-situ and test cube results for both strength concretes.

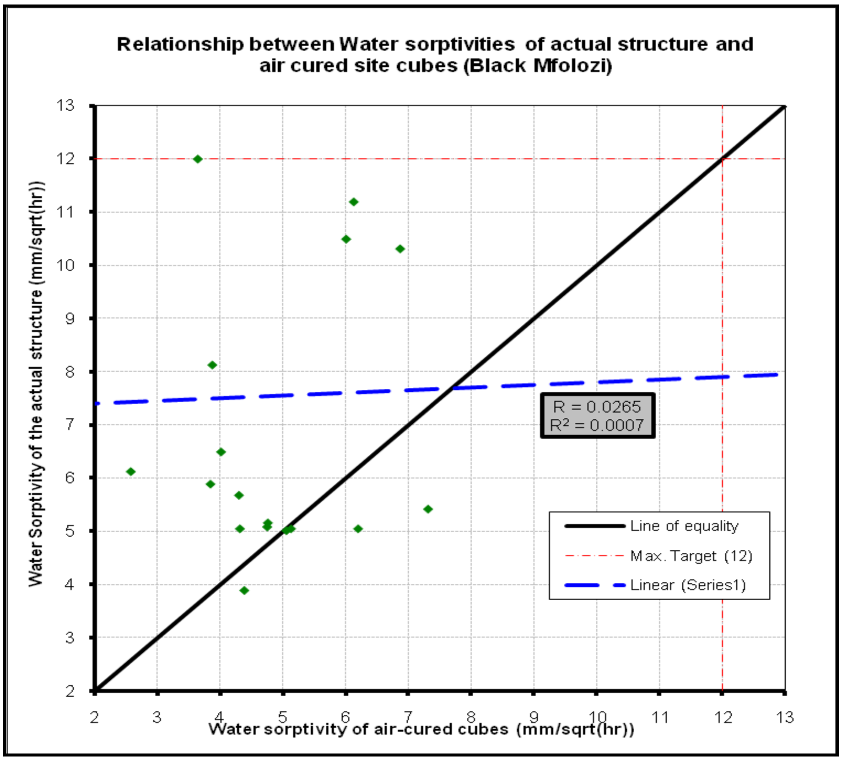
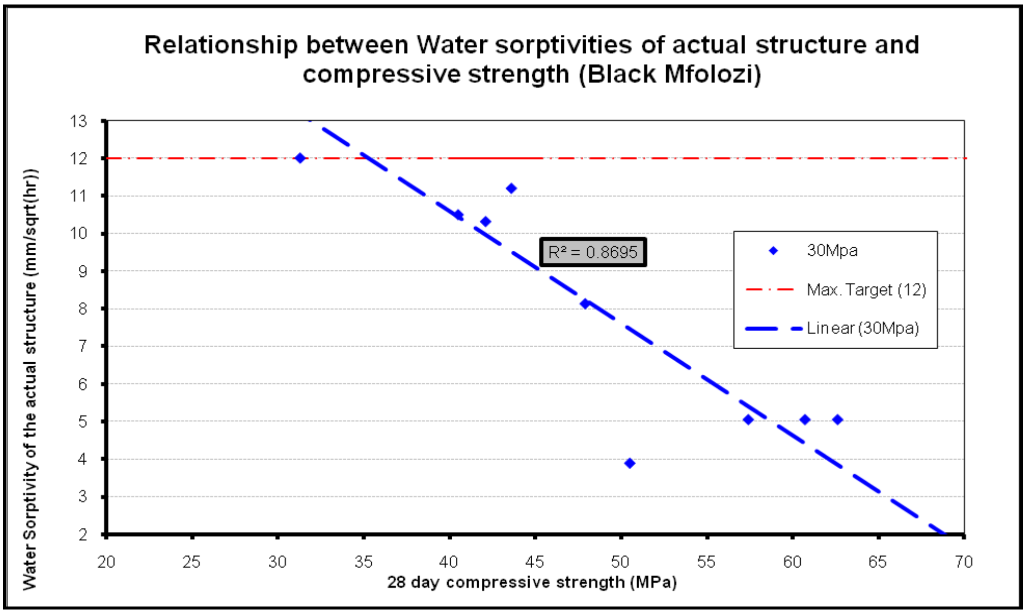



**Figure 7.19: Relationship of Sorptivity for in-situ versus site (air) cured cube results**

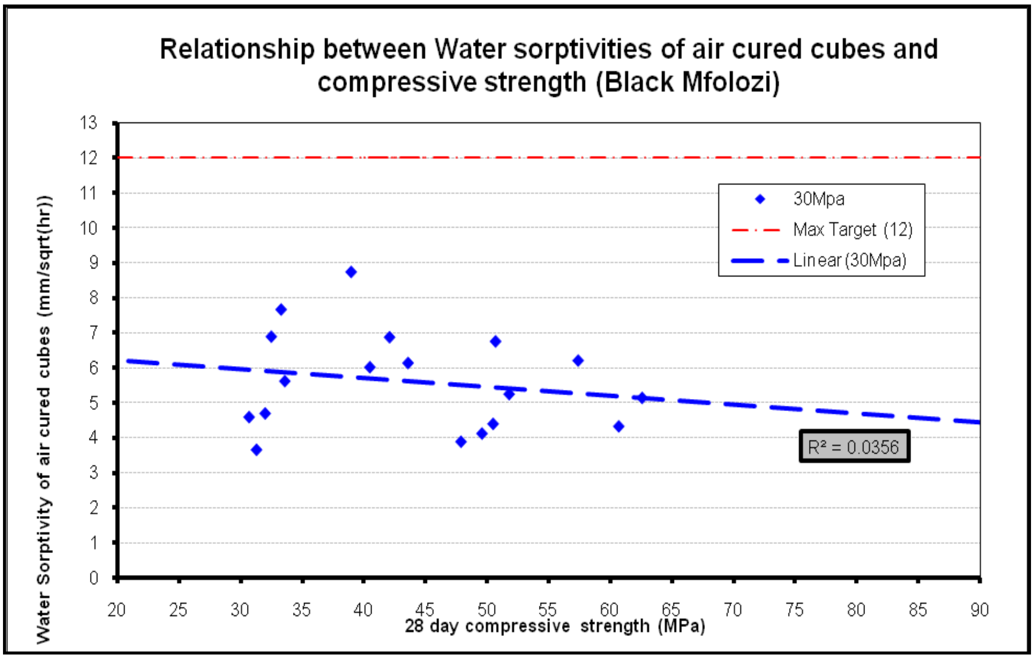
The diagram shows a wide scatter of results. Majority of the results are above the line of equality indicating that the in-situ results are closer to the maximum target of 12 than the cube results. No correlation could be gathered from the results with the value of ‘r’ and ‘$r^2$’ close to zero. For sorptivity, the ‘r’ value is 0,0265 and ‘r2’ is 0.0007. This indicates that a very poor correlation exists between them. The scatter diagrams in Figure 7-20 (a) and (b) below shows the relationship between Sorptivity and strength results.

The results indicate that both the in-situ (apart from three sample lots) and site cured results are all below the maximum target value of 12,00.

With regard to the relationship of sorptivity and strength, the in-situ trend line indicates decreasing sorptivity values with increasing strength (with a steep gradient). This indicates once more that that the trend lines are indicative that durability is not related to strength and again confirms the conclusions of Gouws et al (2001) that durability is not related to strength.




**(a) In-situ**

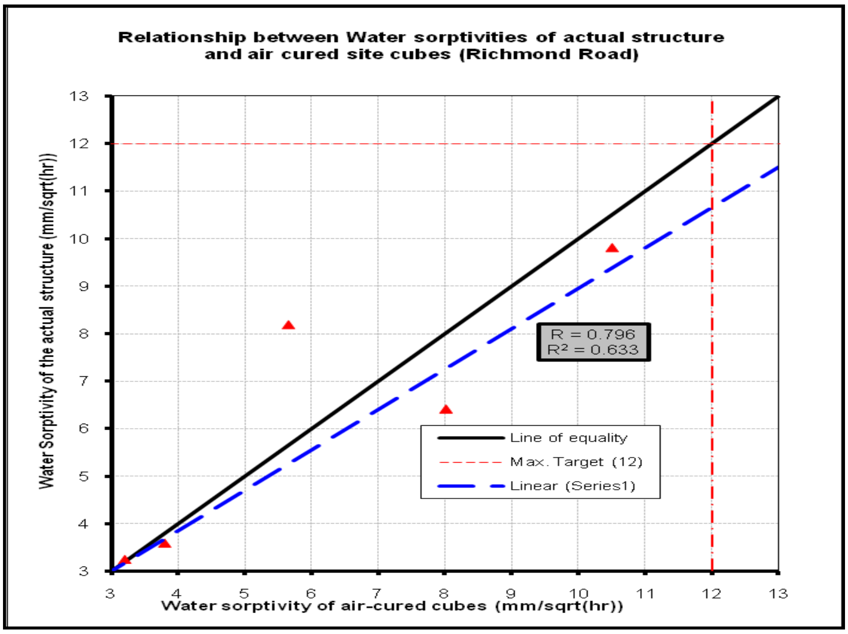



**(b) Site cured cubes**

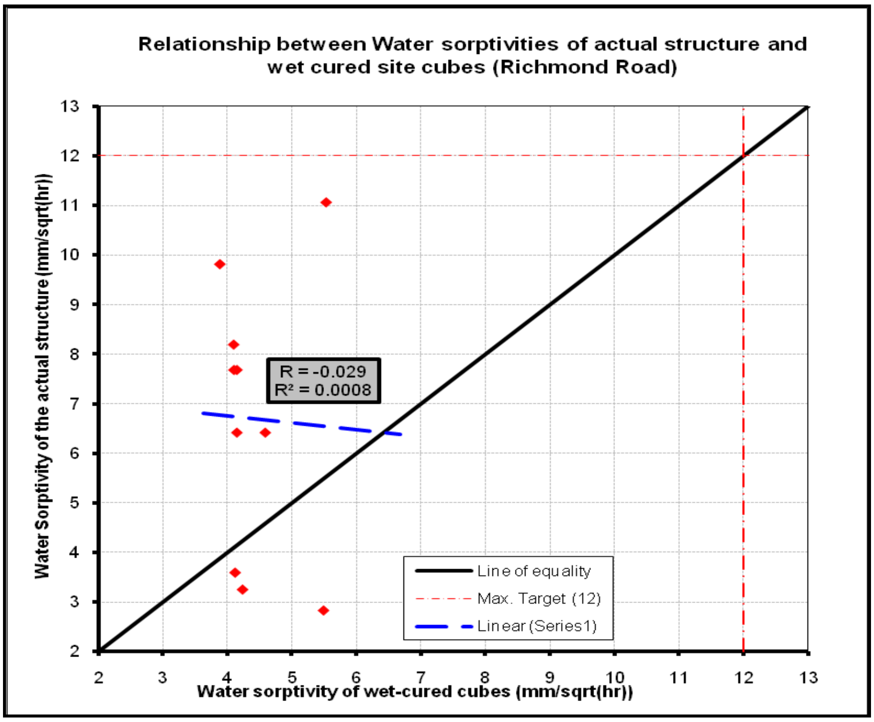
**Figure 7.20 (a) & (b) : Relationship of Sorptivity versus strength for both in-situ and site cured cubes**

### 7.6.3 Richmond road Interchange Bridge

The results in Table 7-8 shows that the sorptivity target has been met in both the wet/air cured cubes and in-situ results, with the in-situ results being higher and closer to the target requirement of 12,0 i.e. the in-situ results are less superior to the cube results.




**(a) Air cured cubes**



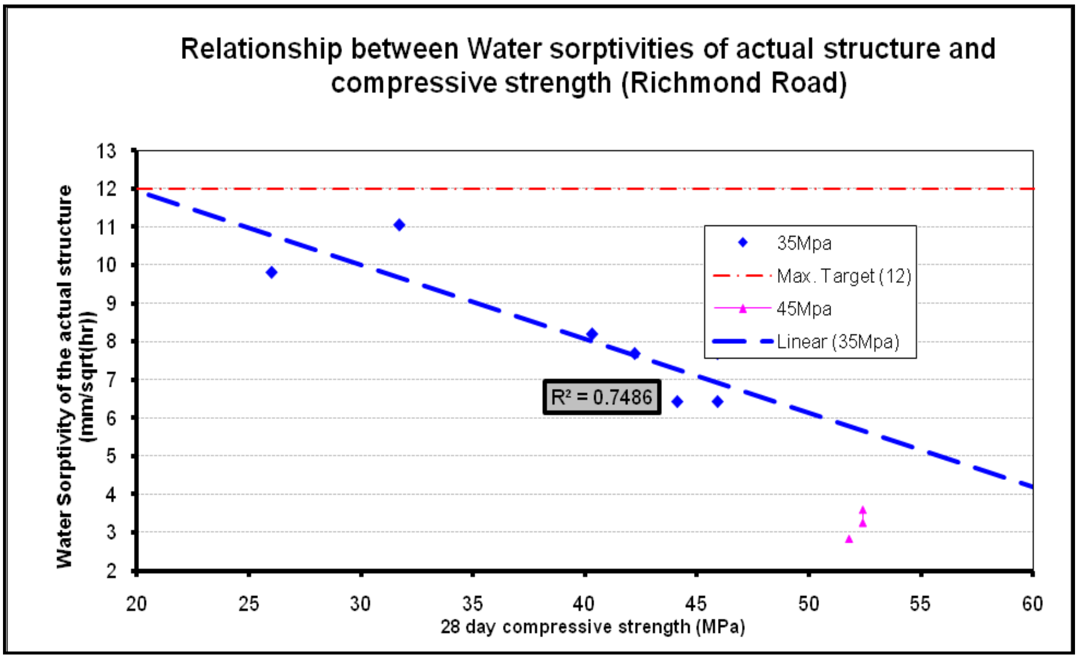

**(b) Wet cured cubes**

**Figure 7.21 (a) & (b) : Relationship of Sorptivity for in-situ versus cured cube results (air and wet cured)**

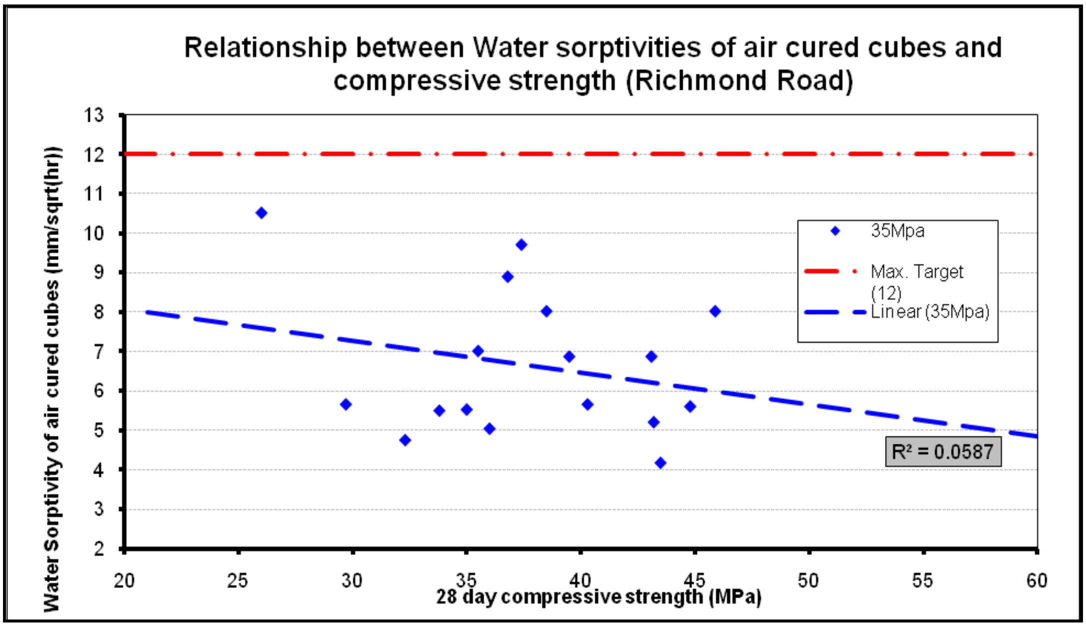
The scatter diagrams in Figure 7-21(a) and (b) shows the relationship between in-situ and test cube results (both site (air) cured and wet cured). The diagrams in Figure 7-21 (a) and (b) shows a wide scatter of results. Majority of the results are above the line of equality indicating that the in-situ results are closer to the maximum target of 12 than the cube results.

For sorptivity values of the air cured cubes and in-situ, ‘r’ is 0.796 and ‘r2’ is 0.633. For the wet cured cubes and in-situ, the ‘r’ value is -0,029 and ‘r2’ is 0.0008. Hence the air cured cube results show a better correlation with the in-situ values.

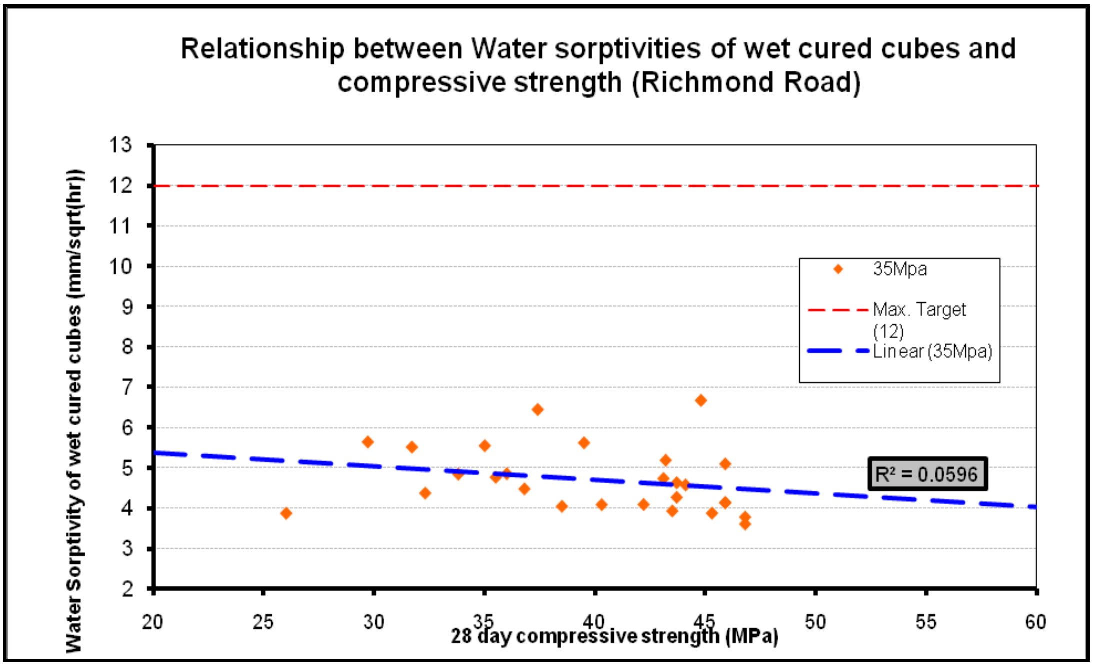
The scatter diagrams in Figure 7-22 (a), (b) and (c) below shows the relationship between sorptivity and strength results.




(a) In-situ




(b) Air cured cubes




**(c) Wet cured cubes**

**Figure 7.22 (a), (b) & (c) : Relationship of Sorptivity versus strength for both in-situ and site cured cubes**

The scatter diagrams indicate that all results are below the maximum target value of 12,00. With regard to the relationship of sorptivity and strength, all three trend lines indicates decreasing sorptivity values with increasing strength, and is near horizontal with the 'r-square' value being close to zero except for the in-situ results where the line is steeper, which is an indicator of the variability of the sorptivity results for the in-situ concrete. This could indicate that sorptivity is very sensitive to curing and compaction.

The trend line has been plotted for the 35MPa concrete only since limited tests were done for the 45MPa concrete.

### 7.6.4 King Shaka International Airport Bridges

The results in Table 7-9 above shows that the sorptivity target has been met in both the site/wet cured test cubes as well as on the in-situ results. Unlike the other projects, the site cured cube results are closer to the maximum limit than the in-situ results, with the wet cubes results being the lowest, indicating the best quality concrete. The scatter diagram in Figures 7-23 and 7-24 below shows the relationship between in-situ and test cube results (both site cured and wet cured).

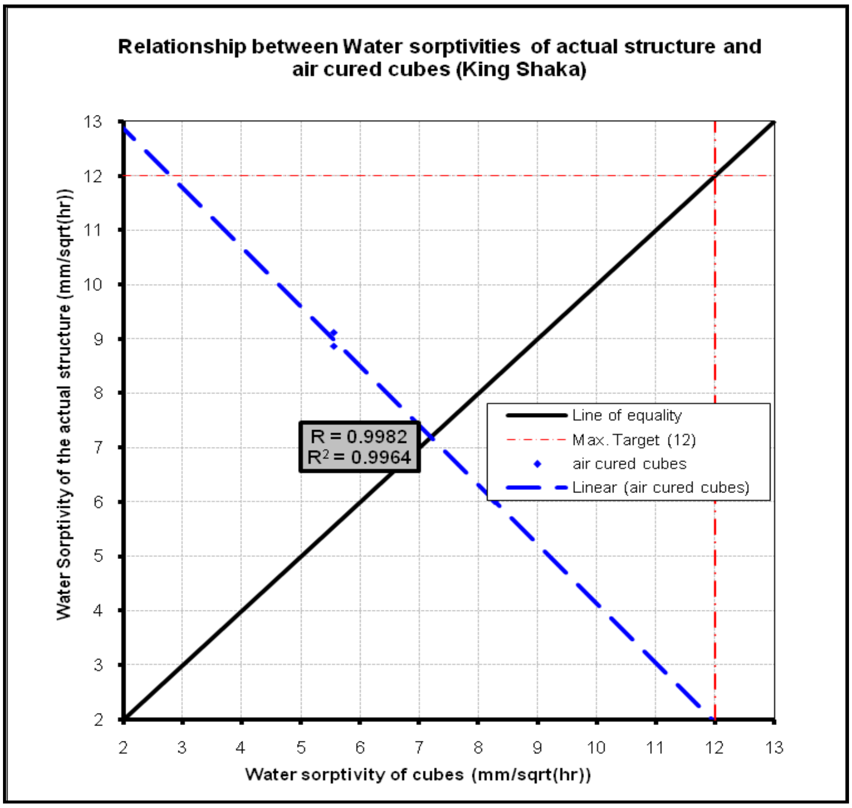



**Figure 7.23: Relationship of Sorptivity for in-situ versus cube results (site cured)**

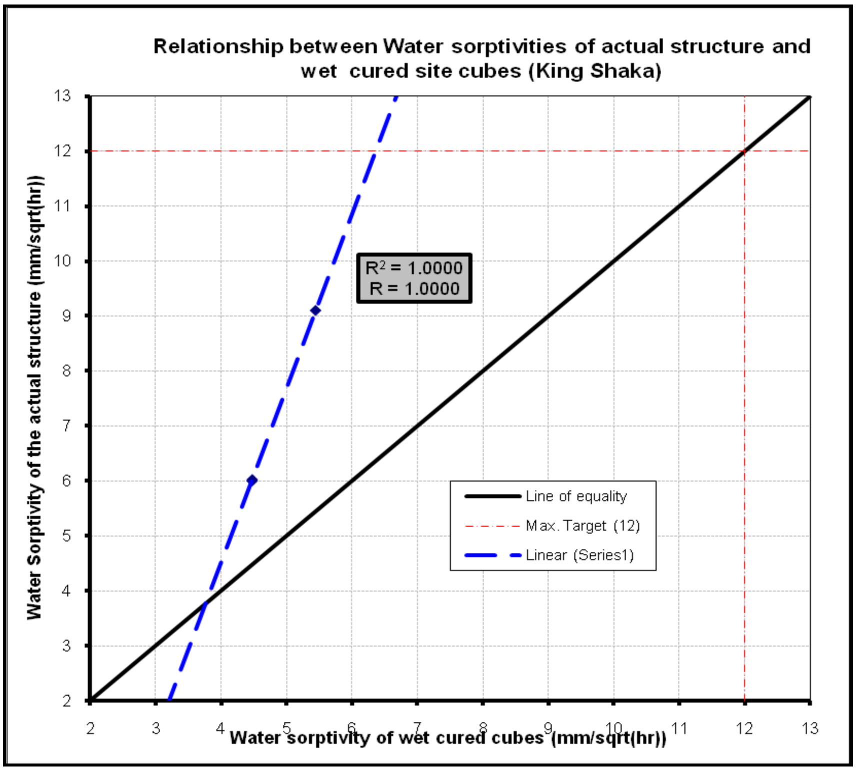



**Figure 7.24: Relationship of Sorptivity for in-situ versus cube results (wet cured)**

Both diagrams shows only three set of results. With the limited number of test results, the correlation provided is not a true reflection of the relationship of in-situ and air cured cubes, although it indicates a very good correlation.

The scatter diagram in Figure 7-25 below shows the relationship between Sorptivity and strength results for wet cured cubes. The graph indicates that all results are below the maximum target value of 12,00 and the trend line shows a poor correlation with a correlation value of only 0,1236. due to the limited results for the air cured results a correlation was not undertaken.

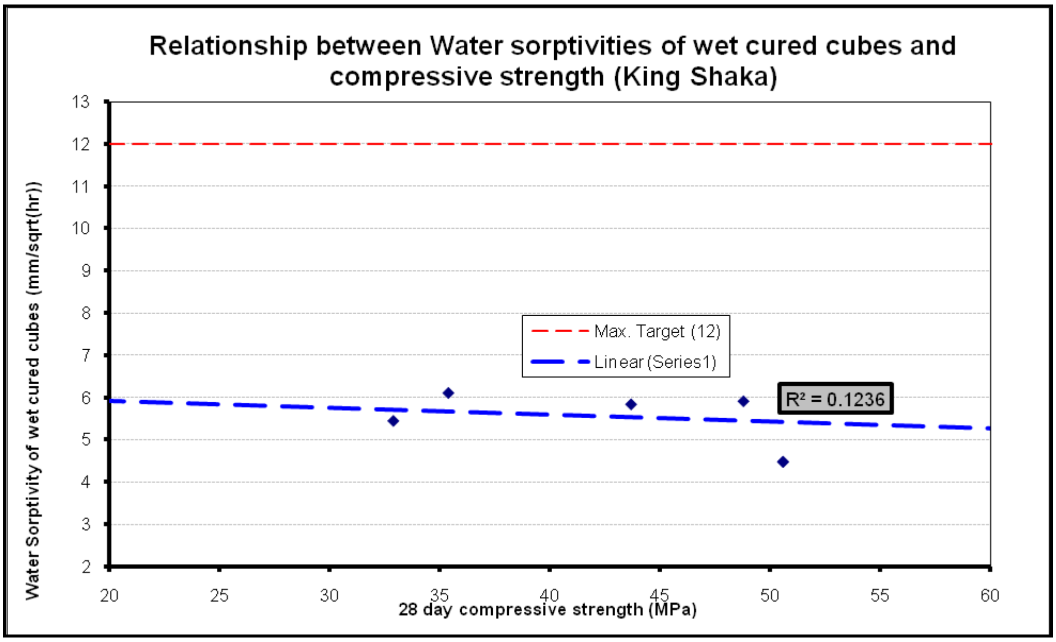
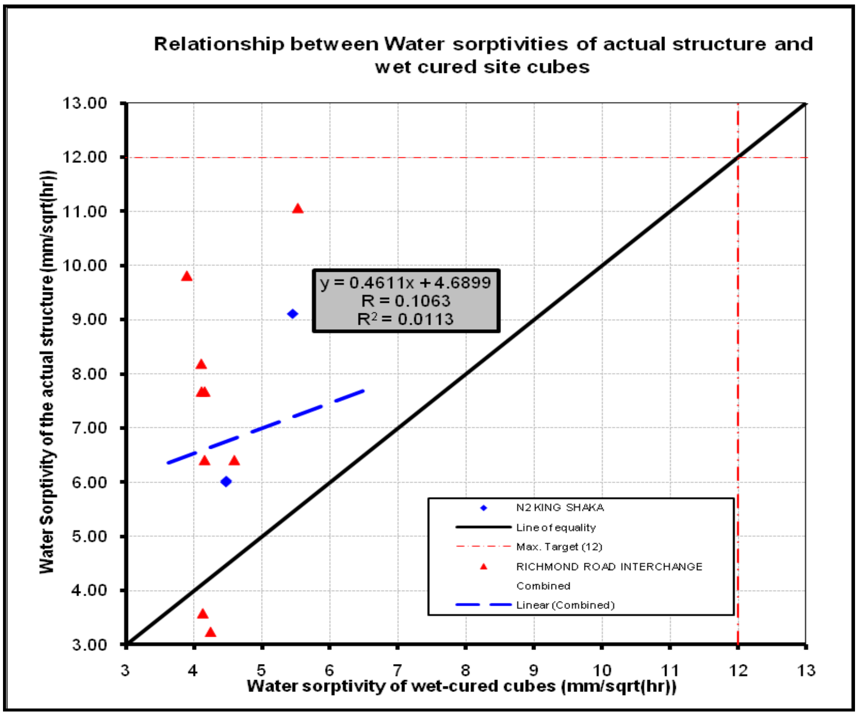


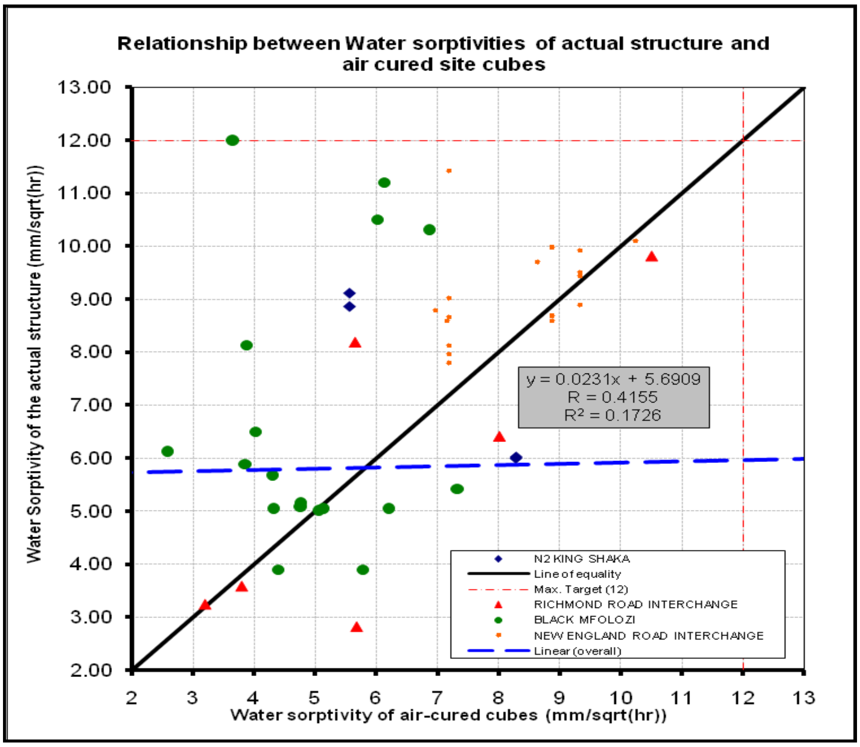

**Figure 7.25: Relationship of Sorptivity versus strength for wet cured cubes and in-situ concrete**

### 7.6.5 Combined Project cube results

Similar to the oxygen permeability tests, the results for sorptivity from wet/site cured cubes and in-situ cores for all the projects were plotted and shown in the scatter diagrams of Figure 7-26 (a) and (b).




(a) Wet cured cubes




(b) Air cured cubes

**Figure 7.26 (a) & (b) : Relationship of Sorptivity for in-situ versus cured cube results (air and wet cured)**

From the diagrams, both linear trend lines are very different to each other, with the wet cured results showing a better correlation than the air cured results, similar to the oxygen permeability results. For the wet cured cubes, the resulting linear correlation line equation is:

Y = 0.461X + 4.690.....................................................................8.4

Where;

Y = sorptivity of the structure, and

X = sorptivity of wet cured cubes.

For the site cured cubes, the equation is:

Y = 0.023X + 5.691.....................................................................8.5

Where;

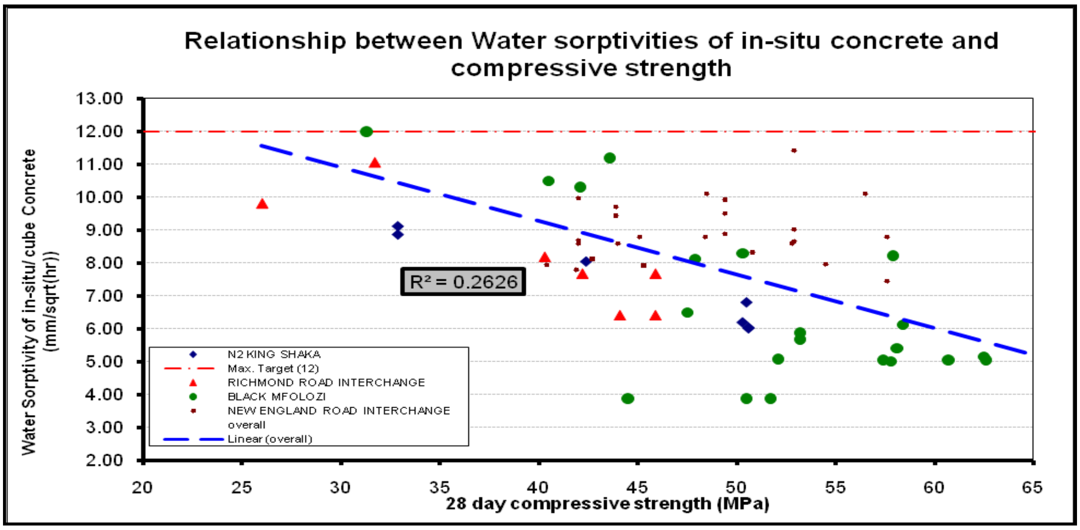
Y = sorptivity of the structure, and

X = sorptivity of air cured cubes.

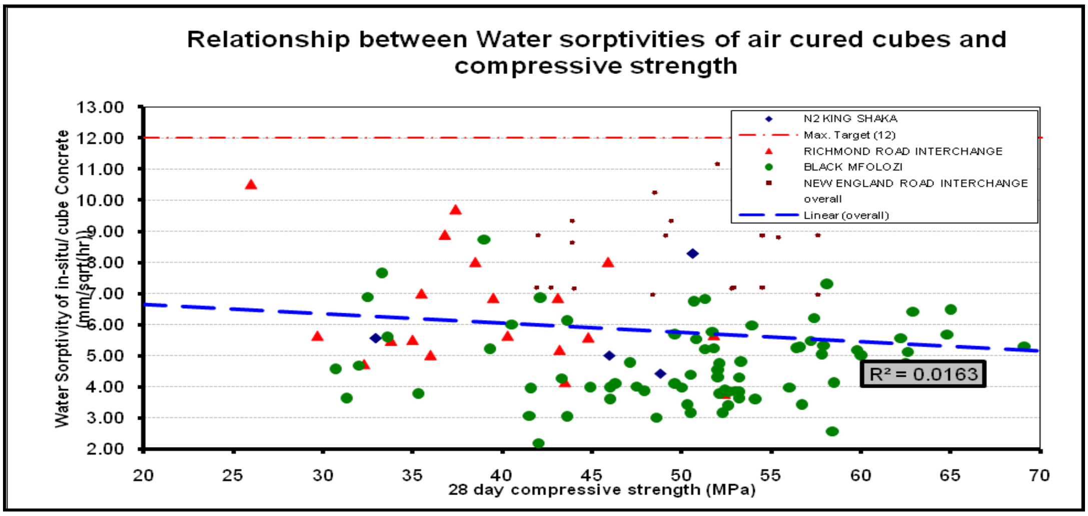
From Figure 7-26 (a) and (b) and the equations 8.4 and 8.5, it is evident that the value of sorptivity for the wet and air cured cubes is higher up to a limit of 8.70 and 5.80 respectively. Thereafter, the in-situ values become higher. Therefore the limiting value of 12 for sorptivity on the structure will result in a much higher value being required in the wet cured cubes, which does not make sense as a poorer quality concrete for the cubes will not result in the maximum value of 12 being obtained in the structure.

As sorptivity is sensitive to curing and conditions where the project is located, the combined graph could indicate that a limiting value of 8,7 is required on the wet cured cubes and similarly a limiting value of 5.80 for the site cured cubes for these particular projects. The overall sorptivity values from all four projects suggest that good quality concrete has been produced as all values were much lower that the recommended maximum of 12,00.

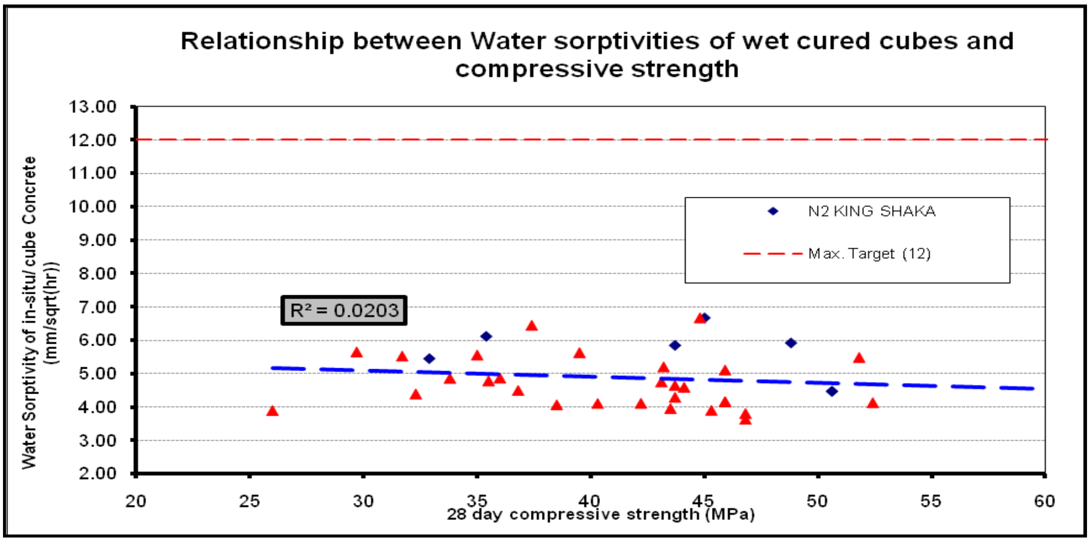
The scatter diagrams in Figure 7-27 (a), (b) and (c) below shows the overall relationship between sorptivity and strength results.




**(a) In-situ**




**(b) Air cured cubes**




**(c) Wet cured cubes**

**Figure 7.27 (a), (b) & (c) : Relationship of Sorptivity versus strength for both in-situ and air cured cubes**

All the scatter diagrams indicate increasing strength with reducing sorptivity values, with the slope of the correlation line the steepest for the in-situ values. Considering each of the figures above, it is clear that the spread of values gets close the to the maximum value of 12 for the in-situ results, while the site cured cubes is lower than the maximum and the wet cured cube results is the lowest of all three. The variability of results from the best cured samples (wet cured) to the in-situ results (affected by curing and compaction) shows that the results of sorptivity is affected by workmanship and that curing may not be as effective on structures as on cubes.

Except for a small proportion of in-situ results, all other results are below the maximum value of 12. The least scatter of results which also showed very low results (average of approximately 5.0) was for the wet cured cubes. This indicates the importance of good controlled curing concrete to ensure long term durability (CSSA, 1991). As was with the OPI results, the value of ‘r-square’ is very low and close to zero indicating a very poor correlation of strength and sorptivity.

### 7.6.6 Combined Project trial panel results

As discussed under section 7.5.6 in the previous section, the results of trial panels here were compared with the in-situ values for sorptivity as was done for oxygen permeability on three of the projects viz. the King Shaka Airport bridges, the Richmond Road Interchange Bridge and the Mgeni Interchange Bridges.

A correlation was therefore made of the concrete in the structure and that in the panels. Table 7-13 below provides the test results, while scatter diagrams in Figure 7-28 shows the relationship for sorptivity oxygen permeability respectively.

From Figure 7,28 for sorptivity, the resulting linear correlation equation is:

$Y = 0.496X + 3.300$.....................................................................8.6

Where;

Y = sorptivity of the structure, and

X = sorptivity of trial panels.

The correlation equation indicates that the value of sorptivity for the trial panels is lower up to a limit of 6,50. Thereafter, the in-situ values become higher, indicating poorer quality concrete. Therefore the limiting value of 12 for sorptivity on the structure will result in a much higher value being required in the trial panels.

**Table 7-11: Sorptivity Results for Trial panels and in-situ concrete**

| Structure | Member | Water Sorptivity (mm /√hr) (<12) | |
|---|---|---|---|
| | | In-situ | Trial panel |
| King Shaka Bridges | Substructures - Vertical | 6.01 | 5.86 |
| | | 6.03 | 5.86 |
| Richmond Road Bridge | Deck-Bottom Slab/ webs | 3.59 | 4.84 |
| | | 3.25 | 4.84 |
| | Substructures (vertical) | 6.42 | 4.65 |
| Mgeni River Bridges | Median span 2 | 5.78 | 3.69 |
| | Median span 2 | 5.78 | 4.30 |
| | NBC Span 1 | 4.61 | 3.37 |
| | NBC Span 1 | 4.61 | 4.16 |

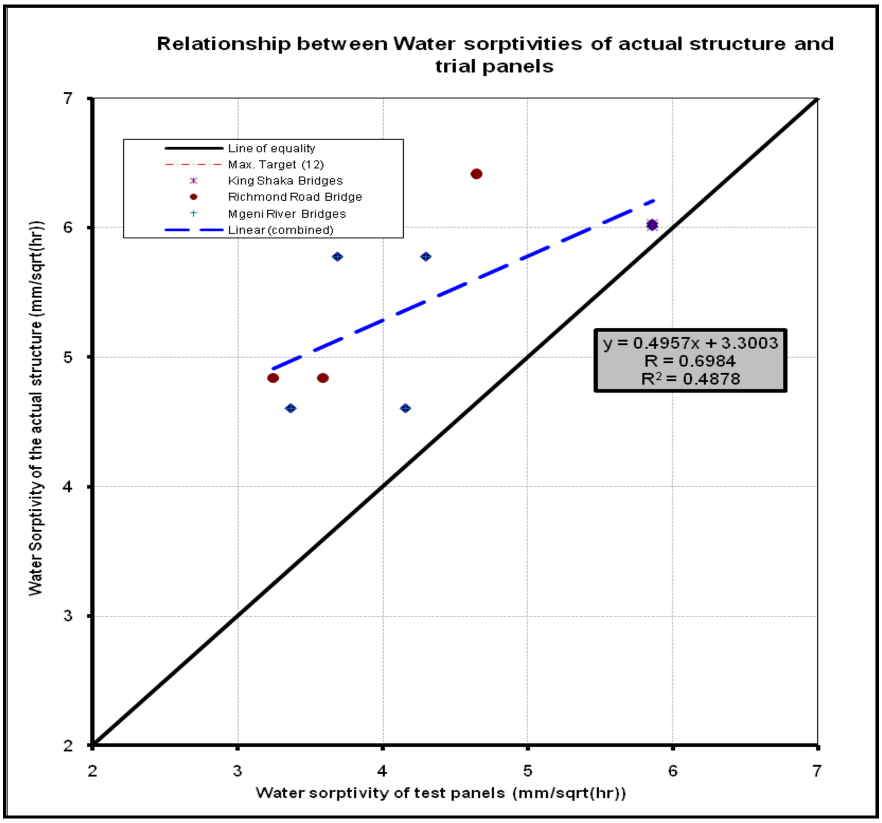



**Figure 7.28: Relationship of Sorptivity for trial panels and in-situ concrete**

The linear regression line shown in Figure 7.28 shows a good correlation with a ‘r’ value of 0.698 compared with any of the linear regression correlation for any of the projects. The values obtained for the trial panels as well as the in-situ concrete is again much lower than the maximum target value of 12, and therefore achieving this in both the structure and the trial panels does not seem to be an issue.

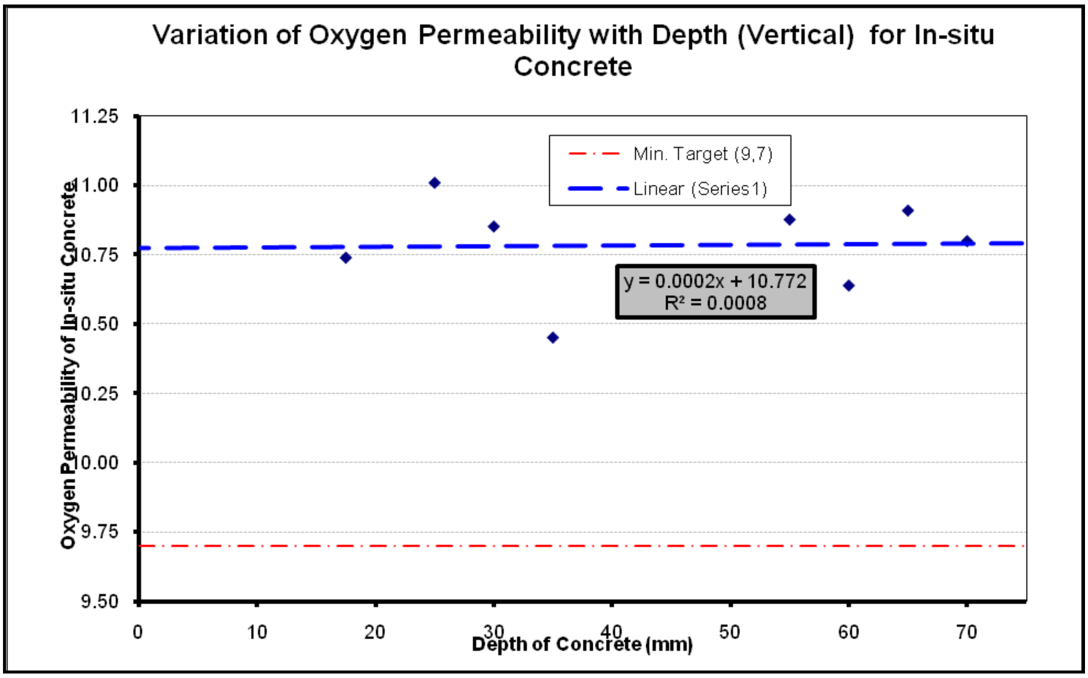
## 7.7 Variation of Durability Indexes with Depth (Vertical)

Bridge decks are always easily accessible for in-situ coring and testing after casting of the concrete i.e. during the curing period and before placement of any waterproofing coatings or asphalt riding surface. It would therefore be possible to core and test the in-situ deck concrete in future specifications. On the Richmond Road Bridge, additional cores were therefore taken to a sufficient depth to check for any variation of oxygen permeability and sorptivity.

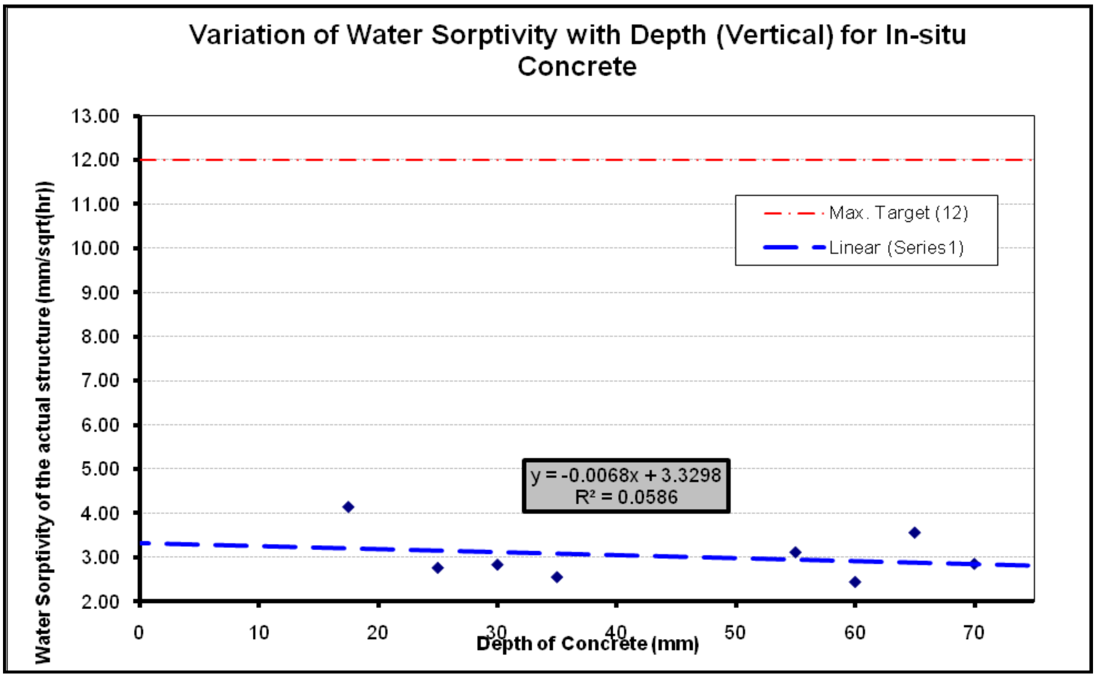
A total of 32 tests were carried out each for oxygen permeability and sorptivity respectively. The rise of bleed water to the surface has an effect on the durability tests on top of concrete elements, especially for deep elements (Gouws et al, 1998); however on decks the majority of concrete pours are not deep but rather wide. Table 7-14 provides details of the test results for oxygen permeability and sorptivity, while scatter diagrams in Figure 7-29 (a) and (b) shows the relationship for sorptivity and oxygen permeability with depth respectively.

**Table 7-12: Oxygen Permeability /Sorptivity Results for various depths in in-situ concrete**

| Concrete Grade | Curing Regime | Element/ Position | Average 28-day Strength | Depth (Midpoint) (mm) | In-situ cores | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | | OPI (log value) | | Sorptivity(mm/√hr) | |
| | | | | | Ave | CoV (%) | Ave | CoV (%) |
| W45/19 | Mist Spray | Decks - Horizontal | 51.8 | 17.5 | 10.74 | 4.05 | 4.14 | 17.43 |
| | | | | 25 | 11.01 | 2.82 | 2.75 | 14.00 |
| | | | | 30 | 10.85 | 2.07 | 2.83 | 7.66 |
| | | | | 35 | 10.45 | 3.83 | 2.54 | 20.17 |
| | | | | 55 | 10.88 | 1.27 | 3.11 | 25.66 |
| | | | | 60 | 10.64 | 2.73 | 2.43 | 27.13 |
| | | | | 65 | 10.91 | 3.07 | 3.56 | 28.75 |
| | | | | 70 | 10.80 | 3.81 | 2.84 | 33.40 |




**(a) Oxygen Permeability**




**(b) Sorptivity**

**Figure 7.29 (a) & (b) : Relationship of Oxygen Permeability and Sorptivity with vertical depth of in-situ concrete**

The scatter diagram for oxygen permeability, i.e. Figure 7-29(a) shows a very small variance with depth, with ‘r’ almost equating to zero at 0.027 and r squared equating to zero, indicating no correlation of oxygen permeability with depth. The linear equation shown indicates that the average value of the sample is almost unchanged with depth.

For the scatter diagram in Figure 7-29(b) of sorptivity versus depth, ‘r’ = -0.242 and r-squared = 0.059, which again indicates no correlation with depth. The values sorptivity are very low in relation to the maximum target value of 12, indicating very good quality concrete. The linear equation shown indicated for sorptivity shows that the average value of the sample slightly improves with depth, but almost negligible.

Therefore in summary, no trend could be determined to indicate inferior quality concrete towards the surface, and therefore bleed water does not seem to influence the durability parameters in this case. The deck thickness for Richmond Road Bridge was 1,35m deep. Majority of bridge decks are in the range of 1,3m to 2,5m. coring from the top of decks could therefore in future be an option to pursue.

## 7.8 Chloride conductivity results

### 7.8.1 Richmond road Interchange Bridge

Chloride conductivity (CC) tests were undertaken during the mix design stage as well on trial panels and in-situ concrete. Due to chloride conductivity being more sensitive to material characteristics than workmanship, the requirements of the specifications are that CC tests be done during the mix design stage and whenever the contractor changes sources of material for the approved mix design. However, poor compaction and curing will also affect the chloride conductivity values. Table 7-15 below shows the results from the trial panels and in-situ cores and which are depicted on the graph of Figure 7-30.

The results show that because of the little effect workmanship has on chloride conductivity, the results are very similar for the trial panels and in-situ. In addition, the uniformity of the in-situ test results proves that none of the concrete material constituents have been varied for the various concrete pours delivered to the site.

**Table 7-13: Chloride Conductivity Results at Richmond Road Bridge**

| Grade | Element | Chloride Conductivity (mS/cm) | |
|---|---|---|---|
| | | Trial panels | In-situ |
| 45/19 | Decks - Horizontal | 0.21 | 0.17 |
| 45/19 | Decks - vertical | 0.17 | 0.15 |
| 45/19 | Decks - vertical | 0.17 | 0.14 |
| 35/19 | Substructure columns - vertical | 0.18 | - |

The scatter diagram in Figure 7-30 shows the relationship between the in-situ results and trial panel results. The value of 'r' and 'r2' are 0.9449 and 0.8929 respectively and indicates a very good correlation between the trial panels and in-situ concrete. It must be noted however, that there is limited number of results to confirm this.

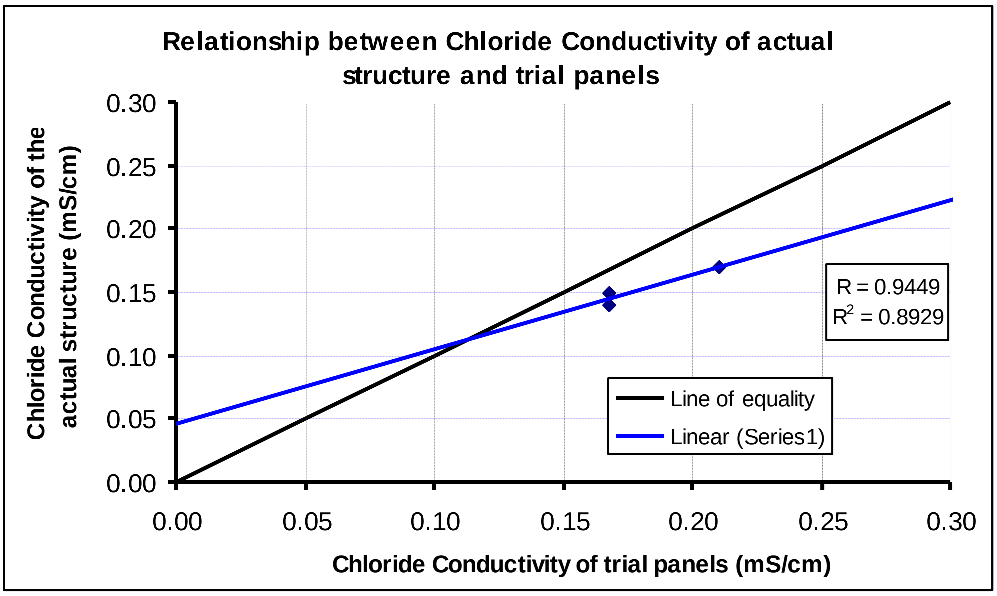



**Figure 7.30: Relationship of Chloride Conductivity for trial panels and in-situ concrete**

### 7.8.2 King Shaka International Airport Bridges

Chloride conductivity (CC) tests were undertaken during the mix design stage and trial panels for the substructures and both deck superstructures. Table 7-16 below shows the results from the mix designs and trial panels for the substructures. The results show that because of the little effect workmanship has on chloride conductivity, the results are very similar for the mix designs and trial panels which have different methods of construction.

**Table 7-14: Chloride Conductivity Results at King Shaka Airport Bridges**

| Grade | Element | Chloride Conductivity (mS/cm) | |
|---|---|---|---|
| | | Trial panels | Mix Design |
| W30/19 | Substructures – Vertical | 0.17 | 0.22 |
| W40/19 | Superstructure – Overpass bridge | 0.25 | Not available |
| W60/19 | Superstructure – Ramp E bridge | 0.15 | Not available |

## 7.9 Closure

The specifications and testing program undertaken on the projects under discussion gave valuable insight into the performance of durability concrete. Much time and effort went into the testing program followed by both the author, the site staff on the various projects as well as the commercial testing facility.

The overall results for the New England Road Bridge show that the sorptivity results passed both for the cubes and in-situ while for the oxygen permeability, the majority of the in-situ results failed although the entire cube results showed concrete passing the requirement. In addition, as expected, the cube results as depicted on the graphs were superior for the cubes than in-situ, indicating that the cubes are not representative of the structure for durability. In terms of strength requirements, the oxygen permeability showed increasing values with increasing strength, while for sorptivity, the graphs had both positive and negative gradients indicating that no clear relationship could be determined.

For the second project, i.e. the Black Mfolozi River Bridge, all of the cube and in-situ results for sorptivity met the maximum target. The results and graphs for the cubes were superior mainly because of the curing regime and compaction employed for the cubes as was evident from the graphs plotted. For the oxygen permeability, the results were again superior on the cubes. In addition, there were certain of the elements that did not meet the minimum requirement which was evident from the both the in-situ and cube results. With regard to strength and durability, the sorptivity values showed decreasing value with increasing strength which is to be expected. For the oxygen permeability, the graph showed increasing values with increasing strength, which again is expected.

The results and graphs for the third project viz. Richmond Road Interchange Bridge showed that all cube results met the requirements for sorptivity and permeability. The wet cured cubes were the most superior followed by the air cured cubes and finally the in-situ results. Certain of the permeability results showed failure for the in-situ concrete. With regard to strength and durability, the sorptivity values showed decreasing value with increasing strength which is to be expected. For the oxygen permeability, the graph showed increasing values with increasing strength, which again is expected. The gradient of the trend lines also gave an indication of the sensitivity of the results for wet cured, air cured and in-situ cured results.

There were very limited test results available for the last project viz. the King Shaka Airport Interchange mainly because of the late start of the project. Nevertheless, the limited results available also followed similarly the trend of the other projects.

Linear regression analysis was undertaken by combining the data from all the projects for the wet cured cubes, air cured cubes and the trial panels and comparing to the in-situ values for both the oxygen permeability and sorptivity values. The wet cured cubes showed a better correlation than the air cured cubes for both indexes, although it was expected that the air cured cubes would provide a better correlation to the in-situ concrete. Of all three test regimes, the trial panels showed the best correlation, and indicate that it can be used to ensure durable concrete is produced in the structure. Due to substantial results from trial panels available from the Mgeni interchange project, it was used in the analysis. Further general comments of the results for sorptivity and oxygen permeability are the following:

- The ineffectiveness of using cubes to predict the durability of the in-situ concrete
- The trial panel results showed the best correlation than the test cubes
- Although there are failures in certain of the in-situ results e.g. New England Road Bridge and Black Mfolozi River Bridge, these were identified as substandard because of the quality of curing and compaction evident on the site
- The trend lines produced of sorptivity versus strength clearly indicated the apparent in-effectiveness of curing which affected the in-situ sorptivity values
- It is noted that high COV values for OPI testing are a matter for concern. could and indicate material variability and this needs to be investigated further.

Tests were also undertaken to check the variability of oxygen permeability and sorptivity with vertical depth of deck. This was done to check if bleed water had any influence on the parameters and whether in future deck could be cored from the top. The results indicates very little variance of permeability and sorptivity with depth.

In Section 8.2.1 and 8.4 of Chapter 8, the overall results of this chapter are critically reviewed. The results from each of the sites based on the concrete quality and location of cores are compared and overall conclusions are drawn. In addition, a comparison is made of the correlations testing between wet cured and site cured cubes as well as cores extracted from the structure.

# 8 EVALUATION AND CRITICAL COMPARISON

## 8.1 General

The dissertation presented in the previous chapters concentrated on addressing three primary issues as follows:

- Compare concrete durability test methods being undertaken internationally, with performance tests and test methods currently adopted by SANRAL;
- Compare concrete performance specifications and testing currently being implemented internationally with specifications currently adopted by SANRAL, including the practicality of construction of trial panels and durability testing on site for quality control;
- Correlate relationships (if any) between results of sorptivity and oxygen permeability values from cubes that are air cured on site and laboratory cured cubes with in-situ results from cores drilled in the structure. Relationship (if any) of compressive strength with sorptivity and oxygen permeability results are also correlated.

Each of these is discussed below. It is to be noted that while many of the comments and recommendations that are provided under this chapter may solely reflect that of the authors, it is in fact made on behalf of SANRAL. The author, who is an employee of SANRAL, is tasked in drafting and revising concrete specifications on its behalf.

## 8.2 Discussion on Durability test methods

### 8.2.1 Current SANRAL Experience

Under the current specifications, four durability tests are undertaken during the construction phase of a bridge structure. During the concrete mix design testing and approval phase, tests are undertaken for sorptivity and oxygen permeability and chloride conductivity (only if structure is located within a very severe or extreme environmental exposure conditions). Targets are set for each of these tests. It seems that due to the special attention that durability concrete mix designs need in order that the index targets

are achieved, currently only a single commercial laboratory is currently capable of undertaking these tests in KwaZulu-Natal. Other commercial laboratories have been approached by SANRAL to set up the equipment and undertake the testing. The major cost is in purchase and setting up of the equipment. There are only a limited number of ready mix suppliers that can undertake the testing at their laboratories. Prior to commencement of construction, core samples are extracted from the trial panels that are cast before any work can commence on the structure, and both the in-situ requirements for sorptivity and oxygen permeability must be achieved.
During the construction phase, additional test cubes are cast purely for coring and testing for the durability criteria required. Tables B8106/1 and 2 provides requirements for sample requirements for the various durability testing criteria, as was highlighted in the previous chapters.

Extensive testing has been undertaken at all three of SANRAL's projects discussed in the previous chapters as well as testing still being undertaken at the King Shaka Airport Bridges. Use has been made of the latest SANRAL requirements and test methods for durability testing. The overall quantum of the tests undertaken as well as the overall summary of the results on each project has been provided under each of the projects in Chapter 7. Apart from the major discrepancy between the in-situ core results and the target requirements for oxygen permeability at New England Road Interchange Bridge, the results for sorptivity and oxygen permeability are fairly consistent. It must also be emphasized that a single commercial laboratory has undertaken all of the durability testing, and therefore the issue of repeatability and reproducibility cannot be ascertained. At the time of completion of this report, another SANAS accredited laboratory was in the process of acquiring the test equipment for all three durability tests in KwaZulu-Natal. Another issue pertinent to the commercial laboratories is that the test equipment is specially designed equipment, and therefore cannot be readily purchased from suppliers of laboratory equipment.

### 8.2.2 International Experience

The South African durability test methods and current research are known of by many of the countries where durability testing and research is ongoing. A comparison has been made with all three South African durability tests with others currently being used in Europe and North America, under the auspices of RILEM. The reference tests used for comparison have been indicated in Table 2-1 in Sub Section 2.3.5 of Chapter 2.

The testing program undertaken by RILEM indicates that both the oxygen permeability and chloride conductivity tests are equally matched if not better suited than the other international tests. The major problem however was with the South African water sorptivity test, and only the RILEM test was successful in differentiating between the mixes used in the RILEM testing program. Previous national testing programs between laboratories indicated that there are certain problems in achieving the desired results with this test.

### 8.2.3 Closure

Currently the Durability Focus Group under the auspices of the Cement and Concrete Institute (C&CI) are in the process of submitting a report to SABS such that all three durability test methods can become SANS standards. SANRAL has adopted the oxygen permeability and chloride conductivity test methods as performance tests where the quality of concrete is subjected to testing to ensure certain targets are met for durable concrete. In addition, the application of a reduction in payment is applied if the durability index requirement is not met for oxygen permeability. The sorptivity test method which initially was used on SANRAL projects as a performance test has since been retracted because of the variability of the results, which is evidenced by the generally high coefficient of variation (CoV). Currently on SANRAL projects, sorptivity is only tested for record purposes to gather data for future research, although it is expected that the values will be within the limits set for the design concrete mix.

## 8.3 Discussion on Durability Specifications

### 8.3.1 Current SANRAL Experience

SANRAL’s current revision to the COLTO standard specifications to ensure durable concrete is constructed is not onerous on contractors to achieve. In fact, very few of SANRAL’s projects over the last five years have shown issues with regard to sub standard structural concrete in terms of durability being produced.

The four ‘C’s to ensure durability i.e. Cover depth, Curing, Compaction and Concrete mix design have been addressed in the specifications. Reduced payments are applied where the measured cover does not meet the requirements, which is based on averages of the surface areas tested. Reduced cover on completed structures is a cause of the majority of the defects e.g. spalling of concrete and cracking. It is therefore a requirement on all SANRAL contracts that cover be checked. Cover depth is currently specified in accordance with Table B6301(provided under Annexure 1), based on the four environmental exposure classes viz. moderate, severe, very severe and extreme, and numerous examples of structural elements within the various minimum cover requirements for each subclass. This is considered too detailed which has been adopted from the previous specifications. Too much emphasis has been placed on the description of structural members and cover requirements for each.

With regard to the various environmental exposure categories as shown in Table B6301(see Annexure 1) , it is recommended that the tables from EN206-1 (Eurocode, 2001) be followed, but expanded. This table is simplistic and could have sub categories. It is therefore proposed to revise the current table under B6301 incorporating the exposure classes with minimum cover requirements. Confusion exists amongst the consulting engineers using SANRAL’s requirements in regard of strength requirements. It was intended that although the characteristic strength is specified in the drawings and schedule of quantities, testing during the mix design process will result in a higher strength being achieved, which will then become the target mean strength for acceptance control requirements. As acceptance testing is based on strength and durability index requirements, it will become unfair to apply a penalty for durability and strength should this be the case; yet the strength is above the characteristic strength. In addition, there is a single target requirement for oxygen permeability nationally for all environmental classes. Drier arid areas in South Africa like the Karoo, are less prone to carbonation and chloride ingress and therefore a different OPI target should be specified. In addition, OPI target is related to cover depth i.e. the deeper the cover, the lower should be the target. A revised Table B6301 has therefore been adopted and is discussed later in this chapter

### 8.3.2 International Experience

Bridge authorities around the world are concerned with the effects of external factors on the long term durability of concrete bridges around the world. Both the USA and the UK have major spending on bridge repairs compared to most other countries. In 2002 alone, a portion between $325 million and $1 billion was spent on repairs to reinforced concrete bridges, the other being on car parks due to deicing salts in the USA (Tullman M, (2007)). A total of approximately $54 billion was required to address bridge deficiencies as was given by the Federal Highways Administration (FHWA). In addition, more than 33% of the US's 600,000 bridges are structurally deficient and the lack of addressing durability criteria during the construction and service life are by far the major reasons for this. In the UK, an estimated amount of £550 million is spent annually for the repair of bridges due to corrosion damage.

The effect of climate change and emissions of carbon dioxide into the atmosphere is also concerning many authorities around the world, including the World Road Association (PIARC), where the author represents South Africa on the technical committee on Road Bridges. One of the themes being focused on is the 'E*ffects of climate change on the design and construction of bridges'*. Increasing levels of $CO_2$ is resulting in many environments which where not prone to carbonation induced corrosion, becoming affected resulting in deterioration of existing structures and more care and diligence required during the design and construction process.

From the survey of the major countries around the world, it seems that both prescriptive and performance based specifications are being used. The use of cementitious extenders is encouraged to ensure the durability is not compromised, although certain countries only allow limited types of extenders to be used. No reasoning is provided in the codes for the choice of certain of the durability tests required. In terms of durability testing being undertaken, only a limited number of countries like the US and Canada undertake in-situ coring and testing after completion of the bridge for sorptivity and permeability. In Europe, the Eurocodes are mandatory and being followed by all European states, with changes specific to each of the country allowed to take place.

### 8.3.3 Closure

The SANRAL specifications have evolved over the years and considering the amount of effort and programs currently available overseas, South Africa is following the correct route with what is being done elsewhere.  Around the world, more emphasis is being placed on concrete durability as researchers and practioners better understand concrete failure due to corrosion and test methods to ensure quality concrete is produced.

## 8.4 Discussion on Durability Correlation testing

### 8.4.1 Individual Project Results

SANRAL has commenced over the last number of years with durability specifications. Five projects located in KwaZulu-Natal were used to undertake correlation testing.  The New England Road Interchange Bridge correlation testing revealed that a relationship exists between cube and in-situ results although the same was not true for oxygen permeability.  In addition, no relationship between strengths and durability could be drawn.  Certain of the testing requirements were however not undertaken due to a lack of experience by the site staff to the specification requirements.  On the Black Mfolozi River Bridge project, similar results from the testing were evident.  Curing and the small size of test cube concrete had a bearing on the results.  The Richmond Road Interchange Bridge project which incorporated wet cured cubes in addition to the air-cured cubes, showed that they were superior to all of the other results.  Similar results were also evident from the King Shaka Airport Interchange project.  The Mgeni Interchange project was used for correlation of the trial panels and in-situ tests and the results proved the value of trial panels where there was a very good correlation.

As was highlighted in the literature survey, wet curing being the ideal form of curing provides the best results for sorptivity and oxygen permeability (Alexander et al, 1994 & Gouws et al, 1998).

### 8.4.2 Combined Project Results

The combined results of all the projects revealed that a good correlation exists for the wet cured test cubes for both oxygen permeability and sorptivity, while the trial panel results provide the best correlation with the in-situ results.

### 8.4.3 Closure

The results of the combined tests of all the projects followed very much the trends of the individual projects. In addition the correlation coefficients calculated showed that the most realistic correlation was for the test cubes and in-situ concrete results. It is clear that durability testing from cores extracted from test cubes provides better results compared to the in-situ concrete. The very small volume of concrete of only 0,003m$^3$ is likely very well compacted using the standard tamping method for cube compaction. In addition, the surface area of each side of 0,023m$^2$ is very small and may be well cured using the standard steel moulds. Equivalent values for OPI for the test cubes were obtained from the linear correlation equation in order to meet the in-situ requirement as required by SANRAL. The results from the trial panels however showed the best correlation compared to the wet and air cured cubes with the in-situ values, although there were limited test sample results. This will therefore require that further correlation testing be undertaken as part of future research before being implemented.

## 8.5 Conclusions from current research

The hypothesis of this dissertation as was outlined in Chapter 1 with regard to the durability of concrete bridges has been adequately fulfilled. The hypothesis was that coring of trial panels and/or test cubes cured on site will replicate results from cores drilled from the structure and therefore can be used to predict durability. With regard to the results of the oxygen permeability index, the linear regression line shown in Figure 7.16 of trial panel results versus in-situ results closely followed the line of equality with a ‘r’ value of 0.9221. This indicates an excellent correlation between the trial panel and in-situ results. Further to this, only oxygen permeability results are used as a performance criteria. With regard to the results of the sorptivity index, the linear regression line as was shown in Figure 7.28 of trial panel results versus in-situ results showed a reasonable correlation with a ‘r’ value of 0.698. The values obtained for the trial panels as well as the in-situ concrete is again much lower than the maximum target value of 12, and therefore achieving this in both the structure and the trial panels did not seem to be an issue.

The literature review that was presented in Chapter 2 gave fundamental reasons for the cause of corrosion in reinforced concrete bridges. The need for durability was highlighted and maintenance problems experienced were discussed. Important was the need to provide background of the durability index tests currently adopted in South Africa and comparisons were made of these tests to other durability tests undertaken internationally. A review was made of previous concrete durability specifications and research undertaken in South Africa and shortcomings with respect to road bridges were presented. A brief summary was also provided of concrete durability specifications used in countries around the world, with a specific review of durability tests being undertaken.

The objectives and methodology of the testing undertaken to test the hypothesis was provided under Chapter 3. Both destructive and non-destructive testing was highlighted which was undertaken under each of the four projects. A review of the current standard and project specifications was performed under Chapter 4. Commentary was provided under each section of the specifications as well as latest design philosophy preferred within the industry.

Chapter 5 provided details of the background (structural details to emphasize type of construction) on each of the projects where testing was undertaken. Criteria for durability testing requirements were also presented. Each contract summary commenced with the location and details of the structural work, description of the environment in which the bridges are located, the durability and strength requirements, and the final concrete mix designs adopted for each. A comparison was made between the durability index targets of the four contracts.

In Chapter 6, limitations as well as discussion was presented under testing of trial panels, concrete cubes and in-situ for the various durability index parameters. All three types of testing methods were undertaken on each of the four contracts. Durability testing results undertaken on each of the four contracts were presented in Chapter 7. The results not only gave guidance on the quality of the concrete produced on each of the sites, but also gave an indication of the type of testing method that would be most representative of the in-situ concrete. The results from each of the sites were discussed and then compared with each other, after which the four contracts were critiqued and evaluated in terms of the testing regime. The differences in test results obtained using the trial panels, test cubes and in-situ coring were compared. Marked differences were discussed with specific references to improving the current specifications adopted by SANRAL. The chapter closed with a summary of the results of the testing. A further section under this chapter considered the combined results from all of the projects reviewed, and again similarities were drawn between the results.

In general, the evaluation highlighted that SANRAL has taken the correct decision in implementing performance specifications for concrete durability as this is being done by all major road authorities around the world. Some of these authorities have gone through major test programs in order that the specifications can be implemented. There is however room for improvement in the current adopted specifications, with specific reference to the environmental exposure classes, strength requirements, durability index limiting values, and durability testing criteria. While data is still being gathered from around the country under SANRAL's contracts, recommendations will be proposed for each of the issues raised above, for consideration to revised specifications being implemented.

It is clear from the evaluation of the combined results that the test cubes for both OPI and Sorptivity provided superior values than in-situ and this was visible from the line graphs that were plotted. On the other hand, the trial panels provided results that more closely followed the in-situ results, although the results were limited. Therefore the hypothesis that coring of trial panels and/or test cubes cured on site will replicate results from cores drilled from the structure and therefore can be used to predict durability, while correctly stated, the results will need to be adjusted for the trial panels as was shown in the relevant tables based on the values chosen by SANRAL.

## 8.6 Recommendations/ requirements for future research

### 8.6.1 Current SANRAL Specifications

The following amendments to the specifications are recommended:

- **Environmental Exposure Classes**

SANRAL has adopted the environmental classes from its previous specifications as shown in Table B6301 (See Annexure 1) of the current specifications. Worldwide, the trend is to rather simplify the number of exposure classes as well as the subclasses, which has been followed by EN206-1 of Eurocode. The environmental classes should be linked to an OPI target value. Table B6301 has therefore been replaced with Table 8.4 which incorporates the format of the EN 206-1 specification but further defines the classes of exposure as well as providing values for OPI, Sorptivity and Chloride Conductivity (saline environment only) for each class of exposure. The table has been developed jointly by SANRAL and the University of Cape Town and shown below.

- **Cover Depth**

With regard to cover depth, the current requirement as shown in Table B6301 is too detailed. Current research suggests that due to the high binder content in durable concrete, cover can be reduced. Otherwise, SANRAL is paying a premium for durable concrete as well as additional cover requirements. It is therefore recommended that the cover requirements be revised as shown in Table 8-4 below where cover depth is linked with both OPI, Sorptivity and Chloride Conductivity (saline environment only) values. This will however need to be considered under future research and testing.

It is to be noted that Table 8-4 is to be provided as a guide only to designers and not incorporated into the specifications. Specifiers will need to consider the least cover specified in order to obtain the durability target values for a structure.

- **Strength Requirements**

The current specifications requires that the *"target mean strength for quality control purposes be based on the mean compressive strength obtained from the mix that* *satisfies both the durability and strength requirements".* Experience has shown that inevitably, strength achieved is based on the durability requirements rather than strength requirements due to a higher binder content. From the contracts where testing has been undertaken, no relationships could be drawn between strength and the durability indexes. Previous research has also indicated that no such relationship exists (Gouws et al, 1998). It will therefore be unfair to penalize a contractor where the durability index has been achieved, but strength fails on the acceptance limit (La) which is based on the mean compressive strength from the mix instead of the characteristic strength. Therefore it is recommended that strength be based on the characteristic strength and the acceptance limit (La) as required by COLTO be based on this.

- **Durability Index Requirements**

Durability index targets should be specified for the different environmental classes because concrete not exposed to a carbonated environment should be treated differently to that exposed to carbonation as well as low humidity areas like the Karoo. Similarly concretes in chloride environments should have more stringent requirements than those in less sensitive environments. It is therefore proposed that as shown in Table 8-4, the various durability index targets for the different environment classes be provided. SANRAL jointly with the University of Cape Town has chosen the OPI, Sorptivity and Chloride Conductivity (saline environment only) targets for the various environments and cover depths based on the durability models that have been developed from the ongoing research at the university. Further research work will be required such that the range of targets provided can be refined in future.

The current specifications exclude Water Sorptivity as an acceptance control test, mainly due to the variability of the results. Results are only recorded during the mix design process and on additional test cubes during the construction stage. However, testing undertaken at the four contracts indicates that the maximum value of 12 is easily achievable, even for the Black Mfolozi River Bridge, which was constructed using labour intensive methods with all concrete batched on site. Further investigation will be required by researchers before this again be introduced in the specifications as a target on site. However, based on the durability models available at the University of Cape Town, recommended and maximum values have been provided in Table 8.4.

- **Durability Testing Requirements**

The program of testing undertaken under the various projects was to confirm SANRAL's need for changing its previous requirements for durability testing by constructing trial panels and test cubes and testing these for water sorptivity, oxygen permeability and certain projects testing for chloride conductivity. With regard to the trial panels, it is recommended that the requirements under the current specifications remain in place. All of the site engineers and site agents representing the consulting engineers and contractors on the projects felt that this was a good method of ensuring that a benchmark is set before construction commences.

## Table 8-1: Concrete Durability Specification Targets (Civil Engineering Structures only)

| Carbonation-Induced Corrosion (from Atmospheric & Industrial) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Design-ation | Description | Condition of Exposure | Description of Exposure | Typical Examples where applicable | Recommended Minimum Cover (mm) | In-situ Durability Index for various Cover Depths within Exposure Condition - 100 Year Life | | | | |
| | | | | | | | OPI (log scale) | | Sorptivity (mm/h) | |
| | | | | | | Cover Depth (mm) | Recommended value | Minimum value | Recommended value | Maximum value |
| XC1a | Low hum. (<50%); exter. conc. sheltered from moisture, arid areas; interior concrete | Mild | Inland dry areas - arid to semi-arid, Karoo etc. Very low (<40%) to low humidity (40% - 50 %). Concrete surfaces not in contact with ground, protected against wetting. | Arid areas, infrequent rain: all exposed members; sides of decks & beams; deck soffits; enclosed surfaces (e.g. interior of box girders); surfaces protected by waterproof cover or permanent formwork not likely to be subjected to weathering; interior members in buildings; | 40 | 40 mm min. cover | N/A | N/A | 10.0 | 12.0 |
| XC1b | Permanently wet or damp | Mild | All areas with access to external or environmental moisture Saturated conditions (RH >95%). Concrete surfaces above ground level kept permanently moist by exposure to water; concrete that never appreciably dries. Concrete surfaces below ground such as piles and buried foundations or abutments kept permanently damp. | Partially submerged and hydraulic structures kept permanently damp; drainage & other elements kept moist; surfaces in contact with permanently damp soil; surfaces kept damp by condensation or moisture; piles (both dry cast and against casings) | 40 | 40 | 9.20 | 9.00 | 10.0 | 12.0 |
| | | | | | | 50 | 9.00 | 9.00 | 10.0 | 12.0 |
| | | | | | | 60 | n/a | n/a | n/a | n/a |
| XC2 | Wet, rarely dry | Moderate | All areas with access to external or environmental moisture Concrete surfaces above ground level kept mostly in moist condition by exposure to water; concrete may occasionally dry for appreciable periods such as when tanks are emptied | Partially submerged and hydraulic or drainage structures kept mostly damp; surfaces in contact with mostly damp soil; surfaces kept mostly damp by condensation or moisture; all wet or mostly damp surfaces which may occasionally dry for limited periods | 40 | 40 | 9.40 | 9.00 | 10.0 | 12.0 |
| | | | | | | 50 | 9.10 | 9.00 | 10.0 | 12.0 |
| | | | | | | 60* | 9.00 | 9.00 | 10.0 | 12.0 |
| | | | | | | 70* | n/a | n/a | n/a | n/a |
| XC3 | Moderate Hum. (50-80%). Ext. conc. sheltered from rain in non-arid areas | Moderate | Near-coastal areas with no chlorides; moist inland areas; adjacent to dams, lakes, major rivers Moderate humidity (50% to 80%), moist climate. Exterior concrete surfaces in moist areas or adjacent to major water bodies, permanently sheltered from rain or direct surface moisture | Moist areas: sides of beams protected from direct rain; deck soffits; enclosed surfaces (e.g. interior of box girders); surfaces protected by waterproof cover or permanent formwork not likely to be subjected to weathering. Consider additional cover at edges of deck at expansion joints, soffits of cantilevers and parapets. | 40 | 40 | 9.40 | 9.00 | 10.0 | 11.0 |
| | | | | | | 50 | 9.10 | 9.00 | 10.0 | 11.0 |
| | | | | | | 60* | 9.00 | 9.00 | 10.0 | 11.0 |
| | | | | | | 70* | n/a | n/a | n/a | n/a |
| XC4 | Cyclic wet and dry | Severe | All areas with access to external or environmental moisture; arid areas excluded Moderate humidity (50% to 80%), moist climate. Concrete surfaces exposed to rain or alternately wet and dry conditions | All exterior surfaces exposed to rain; surfaces where heavy condensation takes place; surfaces alternately wetted and dried by drainage or environmental moisture, such that moisture may penetrate concrete member. | 45 | 40 | 9.60 | 9.20 | 10.0 | 10.0 |
| | | | | | | 50 | 9.30 | 9.00 | 10.0 | 10.0 |
| | | | | | | 60* | 9.10 | 9.00 | 10.0 | 10.0 |
| | | | | | | 70* | 9.00 | 9.00 | 10.0 | 10.0 |

# Table 8-4 : *Continued*

| Chloride-Induced Corrosion (from Groundwater, Seawater & Sea spray) | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Designation | Description | Condition of Exposure | Description of Exposure | Typical Examples where applicable | Recommended Minimum Cover (mm) | In-situ Durability Index for various Cover Depths within Exposure Condition - 100 Year Life | | | | | | |
| | | | | | | Cover Depth (mm) | Chloride Conductivity (mS/cm) Typical Binder Blends | | | | Sorptivity (mm/h) | |
| | | | | | | | 70:30 CEM1:FA | 50:50 CEM1:GGBS | 50:50 CEM1:GGCS | 90:10 CEM1 : CSF | Recommended value | Maximum value |
| XS1 | Exposed to airborne salt but not in direct contact with seawater or inland saline waters | Very Severe | Proven presence of chlorides; generally < 1km from sea, and coastal river valleys (where chlorides are present) and estuaries, or the presence of chlorides proven by experience or testing. This will include inland salt pans or groundwater carrying slats, etc | All exposed and external surfaces subject to significant airborne salt; any surface on which salt can deposit from the air. | 50 | 40 | 1.50 | 1.60 | 2.10 | 0.40 | 10.0 | 12.0 |
| | | | | | | 50 | 2.10 | 2.20 | 2.80 | 0.50 | 10.0 | 12.0 |
| | | | | | | 60 | 2.60 | 2.70 | 3.40 | 0.65 | 10.0 | 12.0 |
| XS2a | Permanently submerged in sea (or saline waters) | Severe | Permanently (or substantially) submerged: in the sea (without heavy wave action); in coastal saline estuaries & rivers; in any aggressive saline waters Concrete surfaces exposed to heavily polluted industrial waters; permanently or substantially submerged or permanantly wet saline conditions (Generally oxygen starved area approximately 1-1,5m below spring type level) | Coastal or other structures permanently submerged in seawater or other aggressive saline waters, including industrially polluted water; surfaces of structures in contact with marshy conditions | 50 | 40 | 1.00 | 1.10 | 1.40 | 0.30 | 10.0 | 11.0 |
| | | | | | | 60 | 1.40 | 1.60 | 2.00 | 0.40 | 10.0 | 11.0 |
| | | | | | | 60 | 1.80 | 2.10 | 2.50 | 0.50 | 10.0 | 11.0 |
| XS2b | XS2a + exposed to abrasion | Extreme | As above, but with heavy wave action; in any aggressive saline waters where abrasion occurs | As above + exposed to abrasion | 60 (Mandatory) | 60 | 1.45 | 1.70 | 2.00 | 0.40 | 10.0 | 11.0 |
| XS3a | Tidal, splash & spray zones | Extreme | Sea or saline estuaries and rivers, but not permanently submerged; tidal zone; and in a spray or splash zone. surfaces exposed to aggressive saline waters, including heavily polluted industrial waters, without being permanently wet. | Coastal or other structures exposed to intertidal, splash, or spray zones, or exposed to other aggressive saline waters, including industrially polluted waters, without being permanently wet; members subject to burying by aeolian sands near coast | 50 | 40 | 0.65 | 0.85 | 1.00 | 0.25 | 10.0 | 10.0 |
| | | | | | | 50 | 1.10 | 1.35 | 1.45 | 0.35 | 10.0 | 10.0 |
| | | | | | | 60 | 1.45 | 1.70 | 2.00 | 0.40 | 10.0 | 10.0 |
| XS3b | XS3a + exposed to abrasion | | As above, but with heavy wave action or where abrasion or erosion can occur | As above + exposed to abrasion | 60 (Mandatory) | 60 | 1.10 | 1.30 | 1.55 | 0.30 | 10.0 | 10.0 |

**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 than 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

It ensures that correct shuttering, compaction and curing takes place, and the results of the durability indexes will prove the quality of the workmanship. This will then become the benchmark during the construction.

The durability index targets set under the specifications for the trial panels have been achieved on all of the projects. It will also not become an issue during the construction should the targets not be achieved, as the contractor will have proved that they are achievable in the trial panels. The only concern was the size of the trial panels which made it difficult to move around the site as well as that coring had to be done on the site. A revised panel size was therefore in need so that after being cast and cured on the site, it could be transported to a commercial testing facility to be cored and tested. The trial panel results matched closely to the in-situ results and from the linear regression plots, a very good correlation was obtained.

With regard to coring and testing of additional test cubes in lieu of coring and testing of actual structure, it is recommended that if the cube testing is to be retained, the target values be adjusted according to Table 7.10. Both the results from air and wet cured cubes were better than the in-situ concrete. In addition, the current requirement in terms of testing frequency is too intense resulting in a costly exercise to prove that the concrete cast has achieved the durability requirements. Testing of the structural elements has however proved otherwise, since on all four projects, the in-situ results were either higher than that for water sorptivity or lower for oxygen permeability than those results from the test cubes indicating poorer quality concrete. In some cases, the results cored from the structure did not meet the requirement as specified yet the results were met with the test cubes. A possible reason for the difference in results is that the test cube is too small for the effects of compaction and curing to have an influence.

While ideally the route that should be followed is to core and test the structural elements that are constructed, there is still the concern of access and long term durability aspects of the structural element. Since the trial panels may be providing more realistic results to the in-situ concrete, it is therefore recommended that the additional cubes for durability testing be replaced by *'test panels'*. A revised size will be required to ensure that the panels can be moved after being cast on the site. In addition, precast elements and tops of bridge decks can be cored and tested in-situ because of easy access and that the tops of these elements are protected when the bridge is commissioned. The results of the testing undertaken on the Richmond Road Interchange Bridge deck did prove that there is no variability in the durability results with depth from the top surface. The following clause is therefore proposed in the specifications:

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 size of the proposed panels has been chosen in discussion with Mr. Jim Horton such that it still retains those same material characteristics of the in-situ concrete as well as that it can be transported to the lab for coring and testing. It is further recommended that SANRAL uses the test panels as a next round of trials and extracts cores from the structure to check for correlation between the test panel and in-situ concrete. This could be done separate to the contractual requirements on a project, and the additional test cubes could be still tested but for the revised values as where indicated in Table 8.1 above.

- **Method Statement for construction of Durable Concrete**

SANRAL requires that a contractor submits a quality assurance program after being awarded a contract. Part of the quality assurance system should therefore include a statement on the method of construction to ensure that all structural concrete is constructed to the required quality to ensure long term durability. This can then be used as a check on the site that the correct procedures are being followed.

### 8.6.2 Further Work

- **Cover Depth Requirements**

As highlighted above in Table 8-4, further work needs to be undertaken by SANRAL to obtain a balance between durability index requirements versus cover requirements for the various environmental classes as proposed.

- **Monitoring of Durability Indexes**

All of the results from SANRAL's sites should be monitored in future to ensure that a database is created for water sorptivity, oxygen permeability and chloride conductivity. This will help in refining the index values as well as possibly revising the binder requirements during tender stage. The use of test panels instead of coring of structures as a means of assessing the quality of concrete should be further assessed from results on the various SANRAL sites before the final decision is taken.

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# 10 ANNEXURE 1 - TABLES EXTRACTED FROM SANRAL SPECIFICATIONS

# Table B6301: Environmental Exposure classes and Minimum Cover Requirements

| Conditions of exposure | ENV CLASS | Description of member/surface to which the cover applies | Min cover (mm) | | | | |
|---|---|---|---|---|---|---|---|
| | | | Class of concrete | | | | |
| | | | 20 | 25 | 30 | 40 | 50 |
| 1. MODERATE<br><br>Concrete surface above ground level and protected against alternately wet and dry conditions caused by water, rain and sea-water spray | XC 1 | 1.1 Surfaces protected by the superstructure, viz. the sides of beams and the undersides of slabs and other surfaces not likely to be moistened by condensation<br>1.2 Surfaces protected by a waterproof cover or permanent formwork not likely to be subjected to weathering or corrosion<br>1.3 Enclosed surfaces | 50 | 45 | 40 | 30 | 30 |
| | | 1.4 Structures/members permanently submerged | 50 | 45 | 40 | 40 | 40 |
| | | 1.5 Transnet Limited structures:<br>i) Surfaces of precast elements not in contact with soil | NA | NA | NA | 30 | 30 |
| | | ii) Surfaces protected by permanent formwork not likely to be subjected to weathering or corrosion | NA | 30 | 30 | 30 | 30 |
| | | iii) Surfaces in contact with ballast | NA | 55 | 50 | 50 | 45 |
| | | iv) All other surfaces | NA | 50 | 40 | 40 | 35 |
| | XC 2 | 1.6 Structures/ members submerged, rarely dry | 50 | 45 | 40 | 40 | 40 |
| 2. SEVERE (Moderate humidity – 60% to 80%)<br><br>Concrete surfaces exposed to hard rain and alternately wet and dry conditions | XC3 | 2.1 All exposed surfaces<br>2.2 Surfaces on which condensation takes place<br>2.3 Surfaces in contact with soil<br>2.4 Surfaces permanently under running water | NA | 50 | 45 | 40 | 40 |
| | | 2.5 Transnet Limited structures<br>i) Surfaces of precast elements not in contact with soil | NA | NA | NA | 40 | 40 |
| | | ii) Surfaces protected by permanent formwork not likely to be subjected to weathering or corrosion | NA | 40 | 40 | 40 | 40 |
| | | iii) Surfaces in contact with ballast | NA | 55 | 50 | 50 | 45 |
| | | iv) All other surfaces | NA | 50 | 40 | 40 | 40 |
| | | 2.6 Cast in situ piles<br>i) Wet cast against casing | 50 | 50 | 50 | 50 | 50 |
| | | ii) Wet cast against soil<br>iii) Dry cast against soil | 75 | 75 | 75 | 75 | 75 |
| 3. VERY SEVERE<br><br>Concrete surfaces exposed to aggressive water, sea water spray or a saline atmosphere | XS1 | 3.1 Exposed to airborne salts:<br>i) < 5km from sea, east of Cape Agulhas or anywhere up river valleys and estuaries up to 15km of coast or locations subject to prevailing winds carrying significant chlorides;<br>ii) < 15km from the sea, west of Cape Agulhas and river valleys and estuaries or locations subject to prevailing winds carrying significant chlorides | NA | NA | NA | 60 | 50 |
| | | | NA | NA | NA | 80 | 80 |
| | | 3.2 Surfaces in rivers polluted by industries<br>3.3 Cast in situ piles, wet cast against casings | | | | | |
| 4. EXTREME<br><br>Concrete surfaces exposed to the abrasive action of sea water or very aggressive water | XS2a | 4.1 Surfaces in contact with sea water of industrially polluted water<br>4.2 Surface in contact with marshy conditions<br>4.3 Structures/ members permanently submerged | NA | NA | NA | 65 | 55 |
| | XS2b | 4.4 Structures/ members permanently submerged and exposed to abrasion | NA | NA | NA | 65 | 55 |
| | XS3a | 4.5 Tidal splash and wetted spray zones | NA | NA | NA | 65 | 55 |
| | XS3b | 4.6 Tidal splash and wetted spray zones and exposed to abrasion | NA | NA | NA | 65 | 55 |

**Table B6402/1 : Selection of cement types for various environmental exposure conditions**

| *Condition of Exposure* | *ENV Class* | *Placing Temperature of Concrete* | *Type of Cement** |
|---|---|---|---|
| *1. MODERATE<br>Concrete surfaces above ground level and protected against alternately wet and dry conditions caused by water, rain and sea-water spray* | *XC1, XC2* | *< 20ºC* | *CEM I<br>CEM II A – S<br>CEM II B – S* |
| | | *20ºC - 30ºC* | *CEM I<br>CEM II A – S<br>CEM II B – S<br>CEM II A – V (or W)<br>CEM II B – V (or W)<br>CEM III A* |
| *2. SEVERE<br>Concrete surfaces exposed to hard rain and alternatively wet and dry conditions* | *XC 3* | *< 20ºC* | *CEM I<br>CEM II A – S<br>CEM II B – S* |
| | | *20ºC - 30ºC* | *CEM I<br>CEM II A – S<br>CEM II B – S<br>CEM II A – V (or W)<br>CEM II B – V (or W)<br>CEM III A* |
| *3. VERY SEVERE<br>Concrete surfaces exposed to aggressive water, sea-water spray or a saline atmosphere* | *XS 1* | *< 20ºC* | *CEM II B – S 42.5<br>CEM III A<br>CEM II B – V 32.5* |
| | | *20ºC - 30ºC* | *CEM II B – S<br>CEM III A<br>CEM II B – V* |
| *4. EXTREME<br>Concrete surfaces exposed to the abrasive action of sea water or very aggressive water* | *XS2a,<br>XS2b,<br>XS3a,<br>XS3b* | *< 20ºC* | *CEM II B – S<br>CEM III A* |
| | | *20ºC - 30ºC* | *CEM II B – S<br>CEM III A* |

**Table B6404/5 : Acceptable ranges for concrete cover**

| *Test No.* | *Description of Test* | *Specified Cover (mm)* | *Acceptance Range* | | | |
|---|---|---|---|---|---|---|
| | | | *Min* | | *Max* | |
| | | | *Overall* | *Individual bar* | *Overall* | *Individual bar* |
| *B8106(g)(iv)* | *Concrete cover to reinforce-ment (mm)* | *30 to 80* | *85% of specified cover* | *75% of specified cover* | *Specified cover + 15mm or where member depth is less than 300mm the limit accepted in writing by Design Engineer.* | *Specified cover + 25mm or where member depth is less than 300mm the limit accepted in writing by Design Engineer.* |

## Table B8106/1 : Minimum Cube/Core samples from additional cubes for durability testing

| *Testing requirement* | *Laboratory curing* | *Site curing & exposure* | *Total* |
|---|---|---|---|
| *a. Chlorides & Sorptivity* | *4+2=6 cores (3 cubes)* | *4+2=6 cores (3 cubes)* | *12 cores (6 cubes)* |
| *b. Oxygen Permeability & Sorptivity* | *4+2=6 cores (3 cubes)* | *4+2=6 cores (3 cubes)* | *12 cores (6 cubes)* |
| *c. Chlorides, Oxygen Permeability & Sorptivity* | *4+4+2= 10 cores (5 cubes)* | *4+4+2= 10 cores (5 cubes)* | *20 cores (10 cubes)* |

## Table B8212/2 : Reduced payments for concrete cover

| *CONCRETE COVER (mm)* | *% of specified cover* | | *PERCENTAGE (%) PAYMENT* |
|---|---|---|---|
| | *Overall* | *Individual bar* | |
| *Full acceptance* | *≥ 85% <(100%+15mm)* | *≥ 75% <(100%+25mm)* | *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%* | *<65% ≥55%* | *70 %* |
| *Rejection* | *<65%* | *<55%* | *Not applicable* |

# ANNEXURE 2 - DETAILS OF PROJECTS WHERE TESTING WAS UNDERTAKEN

**PROJECT 1 : NEW ENGLAND ROAD INTERCHANGE**

Figure A1 below shows the plan view of the new bridge.

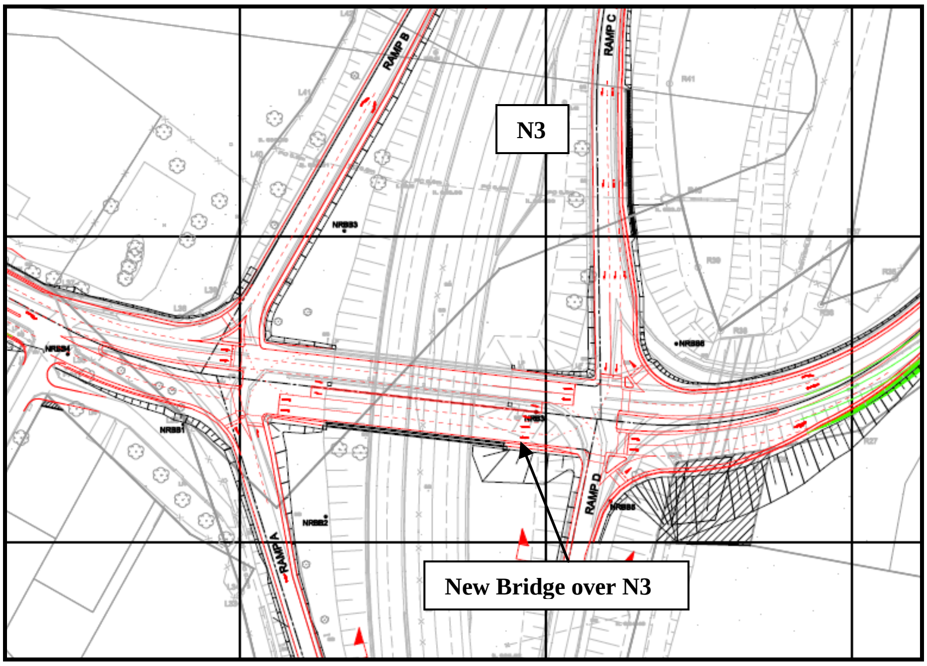


**Figure A1 : Plan View of the New England Road Site (SANRAL Contract N003-003-2005/1, 2007)**

Figure A2 below shows the construction of the piers and abutments.

**Figure A2 : Construction of Substructures (Piers and Abutments) (Source, Author)**


**Figure A3 : Construction of Precast Beams (Source, Author)**

Reinforcement provided at the ends of the beams to control bursting stresses always presents a problem of proper compaction to concrete due to the limited space as can be seen in Figure A3 above. The deck area was very wide as can be seen in Figure A4, and curing was done using a mist spray which proved very effective.


**Figure A4 : Construction of Bridge Deck (Source, Author)**

**PROJECT 2 : BLACK MFOLOZI RIVER BRIDGE**

Figure A5 below shows the plan of the bridge.

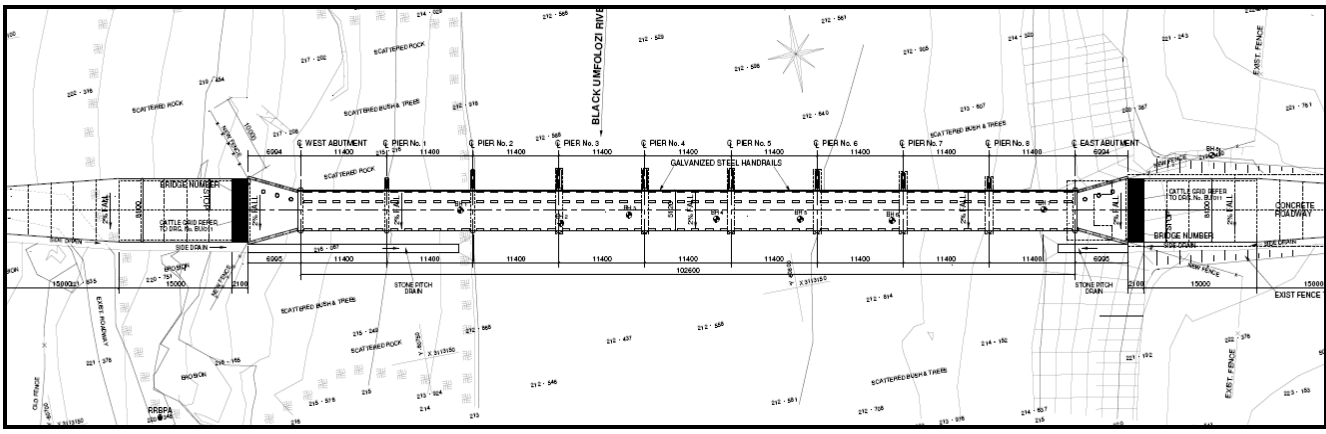


**Figure A5 : Plan View of the Black Mfolozi River Bridge Site (SANRAL Contract P006-032-2007/1, 2007)**

Figure's A6, A7 and A8 show the construction of the substructures, precast beams and in-situ deck. Although labour was used in the mixing of the concrete, it was done to a high standard.

**Figure A6 : Construction of Substructures (Piers and Abutments) (Source, Author)**

The piers were constructed in single lifts and approximately 4,7m high.

**Figure A7 : Construction of Precast Beams (Source, Author)**

All 81 beams were constructed on the site and were designed such that they could be cast in a single stage, with dimensions that made it easier to handle manually.

**Figure A8 : Construction of Bridge Infill and top slab (Source, Author)**

**PROJECT 3 : RICHMOND ROAD INTERCHANGE BRIDGE**

The plan of the bridge is shown in Figure A9 below.

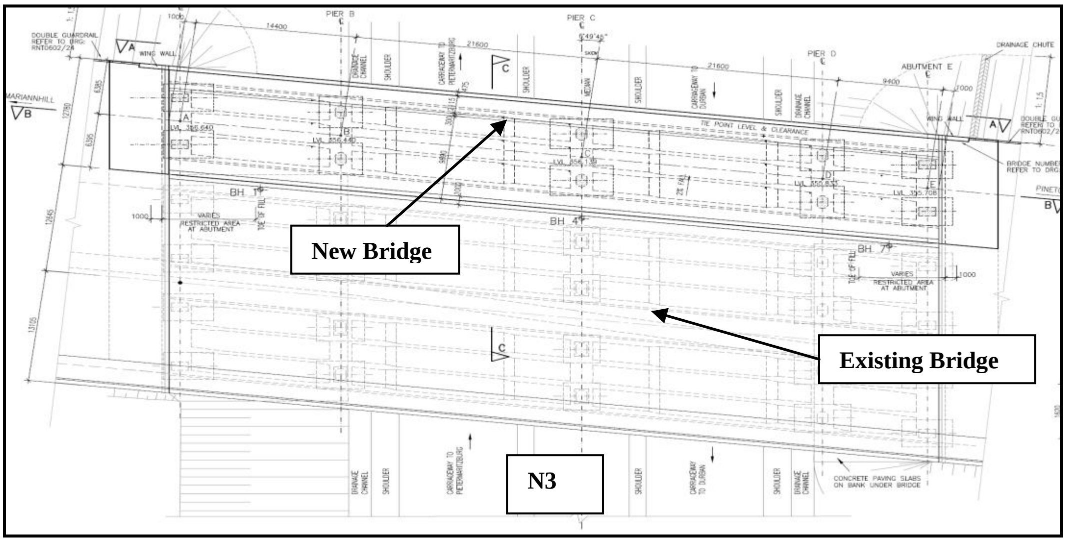


**Figure A9 : Plan View of the Richmond Road Interchange Bridge Site (SANRAL Contract N003-010-2008/1, 2008)**

Figure’s A10 and A11 shows construction of the substructures (piers) and the bridge deck. The deck concrete was designed to be pumped and made compaction easier especially around the bursting reinforcement around the prestress anchorages.


**Figure A10 : Construction of Substructures (Piers and Abutments) (Source, Author)**

**Figure A11 : Construction of Bridge Deck (Source, Author)**

## PROJECT 4 : KING SHAKA INTERNATIONAL AIRPORT INTERCHANGE (KSIA) BRIDGES

The plan of the interchange is show in Figure A12 below.

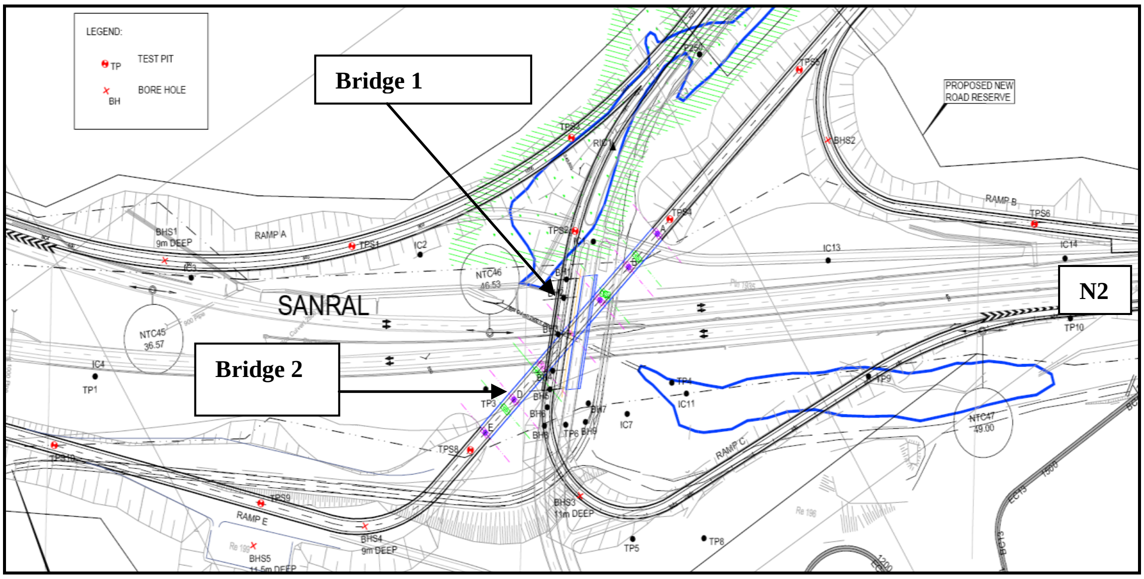


**Figure A12 : Plan View of the King Shaka International Airport Interchange Site (SANRAL Contract N002-260-2005/1, 2008)**

Figure's A13 and A14 shows construction of the Bridge 1 Pier 3, where the finish of the concrete surface was to a high standard. Curing compound was used to cure the concrete surfaces.


**Figure A-13 : Construction of Bridge 1 Pier 3 Pile-cap (Source, Author)**


**Figure A14 : Construction of Bridge 1 Pier3 - 2nd Lift (Source, Author)**

# ANNEXURE 3 - NEW ENGLAND ROAD BRIDGE DURABILITY TESTING RESULTS

**STRUCTURAL CONCRETE DURABILITY RESULTS**
**NEW ENGLAND ROAD INTERCHANGE BRIDGE**

**1 DESIGN MIXES**

| Grade | Description | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity | | | | | | Oxygen Permeability | | | | | | Chloride Conductivity | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV |
| W30 / 19 | Substructure concrete | 15.06.2007 | 20.07.2007 | 20.07.2007 | 44.4 | 6.50 | 7.25 | 6.34 | 7.15 | 6.81 | 6.71 | 10.20 | 10.09 | 10.25 | 10.00 | 10.14 | 1.11 | 0.57 | 0.71 | 0.63 | 0.61 | 0.63 | 9.35 |
| W40 / 19 | Precast beam/Deck concrete | 15.06.2007 | 20.07.2007 | 20.07.2007 | 53.6 | 4.94 | 4.56 | 4.75 | 5.24 | 4.87 | 5.95 | 10.41 | 10.37 | 10.55 | 10.23 | 10.39 | 1.27 | 0.49 | 0.49 | 0.41 | 0.43 | 0.46 | 9.06 |
| | | | | | | | | | | | | | | | | | | | | | | | |

8 8 8

**2 TRIAL PANELS**

| Grade | Cast type | Weather conditions | Curing Regime | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity | | | | | | Oxygen Permeability | | | | | | Chloride Conductivity | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV |
| W40/19 | Precast Beams - VERTICAL | SUNNY | 24 hours in form,curing compound applied when stripped | 06.08.2007 | 08.08.2007 | 28.08.2007 | 53.6 | 7.41 | 9.24 | 7.47 | 6.51 | 7.66 | 14.92 | 10.08 | 9.92 | 10.16 | 9.99 | 10.04 | 1.04 | 0.90 | 1.03 | 0.96 | 0.78 | 0.92 | 11.55 |
| W40/19 | Deck - HORIZONTAL | SUNNY | 24 hours in form,curing compound applied when stripped | 06.08.2007 | 08.08.2007 | 28.08.2007 | 53.6 | 5.58 | 5.56 | 8.07 | | 6.40 | 22.54 | 9.91 | 10.08 | 10.21 | | 10.07 | 1.49 | 0.88 | 0.73 | 0.71 | | 0.77 | 12.01 |

| | Water Sorptivity | | Oxygen Permeability | | Chloride Conductivity | |
|---|---|---|---|---|---|---|
| | 7 | | 7 | | 7 | |
| average (individual) | | 7.03 | | 10.05 | | 0.85 |
| cov | | 12.61 | | 0.21 | | 12.06 |
| average (overall) | | 7.12 | | 10.05 | | 0.86 |
| cov | | 18.83 | | 1.15 | | 14.04 |

**3 IN-SITU (CORES)**

| Grade | Element | Weather conditions | Curing Regime | Core location | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity | | | | | | Oxygen Permeability | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov |
| W30/19 | Pier 3 | Sunny | curing compound | Bottom Upstand | 03.09.2007 | 12.03.2008 | 28.03.2008 | 40.4 | 7.78 | 9.31 | 7.18 | 7.54 | 7.95 | 11.79 | 9.09 | 8.88 | 9.47 | 9.34 | 9.20 | 2.86 |
| W30/19 | Pier 3 | Sunny | curing compound | Columns | 28.09.2007 | 12.03.2008 | 28.03.2008 | 42 | 8.77 | 9.66 | 7.04 | 9.28 | 8.69 | 13.32 | 9.06 | 8.75 | 9.75 | 9.14 | 9.18 | 4.56 |
| W30/19 | Pier 2 | Sunny | curing compound | Bottom Upstand | 21.09.2007 | 12.03.2008 | 28.03.2008 | 45.3 | 9.12 | 6.82 | 8.66 | 7.11 | 7.93 | 14.30 | 8.61 | 9.80 | 9.20 | 9.67 | 9.32 | 5.78 |
| W30/19 | Pier 2 | Sunny | curing compound | Columns | 18.09.2007 | 12.03.2008 | 28.03.2008 | 52.9 | 10.29 | 8.37 | 7.18 | 8.83 | 8.67 | 14.84 | 8.98 | 9.41 | 9.43 | 8..97 | 9.27 | 2.74 |
| W30/19 | Pier 1 | Sunny | curing compound | Bottom Upstand | 13.09.2007 | 12.03.2008 | 28.03.2008 | 50.8 | 8.25 | 8.22 | 7.60 | 9.23 | 8.33 | 8.09 | 9.16 | 9.45 | 9.52 | 9.29 | 9.36 | 1.73 |
| W30/19 | Pier 1 | Sunny | curing compound | Columns | 02.10.2007 | 12.03.2008 | 28.03.2008 | 49.4 | 10.53 | 8.02 | 8.30 | 8.72 | 8.89 | 12.70 | 8.66 | 9.23 | 9.41 | 8.76 | 9.02 | 4.02 |
| W30/19 | West Abutment | Sunny | curing compound | Wall | 24.10.2007 | 12.03.2008 | 28.03.2008 | 42.7 | 8.06 | 7.94 | 6.89 | 9.62 | 8.13 | 13.85 | 8.96 | 8.90 | 9.68 | 9.10 | 9.16 | 3.89 |
| | | | | | | | | | | | | | | | | | | | | |
| W30/19 | West Abutment | Sunny | curing compound | 1m above ground | 17.10.2007 | 04.10.2008 | 17.10.2008 | 41.9 | 8.46 | 9.00 | 7.13 | 6.60 | 7.80 | 14.37 | 9.03 | 8.95 | 9.20 | 9.41 | 9.15 | 2.23 |
| W30/19 | Pier 1 | Sunny | curing compound | 1m above ground | 02.10.2007 | 04.10.2008 | 17.10.2008 | 49.4 | 9.79 | 9.22 | | | 9.51 | 4.24 | 9.46 | 8.98 | | | 9.22 | 3.68 |
| W30/19 | Pier 2 | Sunny | curing compound | 1m above ground | 18.09.2007 | 04.10.2008 | 17.10.2008 | 52.9 | 12.79 | 10.05 | | | 11.42 | 16.97 | 9.51 | 9.47 | | | 9.49 | 0.30 |
| W30/19 | Pier 3 | Sunny | curing compound | 1m above ground | 28.09.2007 | 04.10.2008 | 17.10.2008 | 42 | 9.41 | 7.78 | | | 8.60 | 13.41 | 8.84 | 9.00 | | | 8.92 | 1.27 |
| W30/19 | East Abutment | Sunny | curing compound | 1m above ground | 08.10.2007 | 04.10.2008 | 17.10.2008 | 45.1 | 7.72 | 8.34 | 9.03 | 10.08 | 8.79 | 11.50 | 9.50 | 9.50 | 9.04 | 9.06 | 9.28 | 2.80 |
| | | | | | | | | | | | | | | | | | | | | |
| W40/19 | Beam [8] (1) Span 1 (South) | Sunny | curing compound | Within 500mm under bottom of web | 17.08.2007 | 12.12.2008 | 16.01.2009 | 49.1 | 13.37 | 14.29 | 12.63 | 14.80 | 13.77 | 7.00 | 9.19 | 9.33 | 8.73 | 8.83 | 9.02 | 3.17 |
| W40/19 | Beam [8] (1) Span 1 (South)-retest on 2nd 30mm slice | Sunny | curing compound | Within 500mm under bottom of web | 17.08.2007 | 12.12.2008 | 16.01.2009 | 49.1 | | | | | | | | | 9.60 | 9.06 | 9.33 | 4.09 |
| W40/19 | Beam [1] (21) Span 1 (North) | Sunny | curing compound | Within 500mm under bottom of web | 21.09.2007 | 12.12.2008 | 16.01.2009 | 48.5 | 10.10 | 9.82 | 10.11 | 10.38 | 10.10 | 2.26 | 9.36 | 9.56 | 8.97 | 9.23 | 9.28 | 2.66 |
| W40/19 | Beam [8] (28) Span 4 (South) | Sunny | curing compound | Within 500mm under bottom of web | 04.10.2007 | 12.12.2008 | 16.01.2009 | 44 | 12.67 | 11.40 | 11.28 | 13.33 | 12.17 | 8.19 | 9.40 | 9.39 | 9.37 | 9.47 | 9.41 | 0.46 |
| W40/19 | Beam [1] (27) Span 4 (North) | Sunny | curing compound | Within 500mm under bottom of web | 03.10.2007 | 12.12.2008 | 16.01.2009 | 48.4 | 15.25 | 13.00 | 10.67 | 12.01 | 12.73 | 15.17 | 8.76 | 9.43 | 9.41 | 9.44 | 9.26 | 3.60 |
| W40/19 | Beam [1] (27) Span 4 (North)-retest on 2nd 30mm slice | Sunny | curing compound | Within 500mm under bottom of web | 03.10.2007 | 12.12.2008 | 16.01.2009 | 48.4 | | | | | | | 9.33 | | | | 9.33 | |
| W30/19 | Pier 1-2m above base | Sunny | curing compound | 2m above ground | 02.10.2007 | 12.12.2008 | 16.01.2009 | 49.4 | 7.97 | 11.88 | | | 9.93 | 27.86 | 9.46 | 8.99 | | | 9.23 | 3.60 |
| W30/19 | Pier 1-retest on 2nd 30mm slice | Sunny | curing compound | 2m above ground | 02.10.2007 | 12.12.2008 | 16.01.2009 | 49.4 | | | | | | | 9.51 | | | | 9.51 | |
| | Pier 2-2m above base | Sunny | curing compound | 2m above ground | 18.09.2007 | 12.12.2008 | 16.01.2009 | 52.9 | 8.66 | 9.38 | | | 9.02 | 5.64 | 9.38 | 8.74 | | | 9.06 | 5.00 |
| W30/19 | Pier 3-2m above base | Sunny | curing compound | 2m above ground | 28.09.2007 | 12.12.2008 | 16.01.2009 | 42 | 10.86 | 9.10 | | | 9.98 | 12.47 | 8.49 | 9.11 | | | 8.80 | 4.98 |

| | Water Sorptivity | | | Oxygen Permeability | | |
|---|---|---|---|---|---|---|
| average (individual) | | 9.60 | | | 9.22 | |
| cov | 64 | 18.11 | 19 | 67 | 1.90 | 22 |
| average (overall) | | 9.57 | | | 9.21 | |
| cov | | 21.66 | | | 3.27 | |

0

**4 TEST CUBES (curing compound/ air cured)**

24 hours in moulds. Curing Compound sprayed 1 hour after stripping. Cured at bridge site.

| Grade | Element | Weather conditions | Curing Regime | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity | | | | | | Oxygen Permeability | | | | | | Chloride Conductivity | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov |
| W30/19 | Deck 2 - Culvert | Sunny | 24 | 22.08.2007 | 10.08.2007 | 10.08.2007 | 43.9 | 9.84 | 9.22 | 9.36 | 8.90 | 9.33 | 4.19 | 9.47 | 9.66 | 9.72 | 9.64 | 9.62 | 1.11 | 1.66 | 2.17 | 1.80 | 1.68 | 1.83 | 12.94 |
| W30/19 | Deck 2 -culvert | Overcast | Hours in | 22.08.2007 | 10.09.2007 | 10.09.2007 | 43.9 | 8.69 | 7.55 | 10.00 | 8.32 | 8.64 | 11.85 | 9.60 | 9.95 | 9.71 | 9.71 | 9.74 | 1.52 | 1.72 | 1.54 | 2.04 | 1.68 | 1.75 | 12.11 |
| W40/19 | Beam 2 | Sunny | moulds. | 22.08.2007 | 10.09.2007 | 10.09.2007 | 54.5 | 8.55 | 9.02 | 8.55 | 9.34 | 8.87 | 4.36 | 9.68 | 9.74 | 9.86 | 9.76 | 9.76 | 0.77 | 2.28 | 2.13 | 2.38 | 2.01 | 2.20 | 7.41 |
| W40/19 | Beam 3 | Sunny | Curing | 27.08.2007 | 19.09.2007 | 19.09.2007 | 56.5 | 9.41 | 9.65 | 11.32 | 10.59 | 10.24 | 8.60 | 9.53 | 9.56 | 9.45 | 9.50 | 9.51 | 0.49 | 1.08 | 1.21 | 1.26 | 1.25 | 1.20 | 6.91 |
| W40/19 | Beam 4 | Overcast | Compound | 27.08.2007 | 19.09.2007 | 19.09.2007 | 52.8 | 7.11 | 6.43 | 7.72 | 7.38 | 7.16 | 7.64 | 10.13 | 10.24 | 10.23 | 10.00 | 10.15 | 1.10 | 1.07 | 1.01 | 0.98 | 0.95 | 1.00 | 5.11 |
| W30/19 | Deck 3 - culvert | Overcast | Sprayed | 28.08.2007 | 25.09.2007 | 25.09.2007 | 57.6 | 9.03 | 8.50 | 9.48 | 8.51 | 8.88 | 5.30 | 9.71 | 9.73 | 10.04 | 9.91 | 9.85 | 1.59 | 0.95 | 1.00 | 1.15 | 0.89 | 1.00 | 11.14 |
| W40/19 | Beam 5 | Sunny | 1 hour | 28.08.2007 | 25.09.2007 | 25.09.2007 | 57.6 | 7.74 | 6.98 | 7.14 | 6.00 | 6.97 | 10.36 | 10.26 | 10.14 | 10.37 | 10.05 | 10.21 | 1.37 | 0.67 | 0.90 | 0.84 | 0.84 | 0.81 | 12.20 |
| W40/19 | Beam 6 | Sunny | after | 28.08.2007 | 25.09.2007 | 25.09.2007 | 55.4 | 8.49 | 8.57 | 9.43 | 8.74 | 8.81 | 4.86 | 10.05 | 10.03 | 10.17 | 9.94 | 10.05 | 0.94 | 0.92 | 0.74 | 0.71 | 0.90 | 0.82 | 13.19 |
| W40/19 | Head 4 - Pier 1 | Overcast | stripping. | 29.11.2007 | 21.12.2007 | 21.12.2007 | 54.5 | 8.07 | 6.62 | 6.92 | 7.15 | 7.19 | 8.70 | 9.70 | 9.72 | 9.54 | 9.92 | 9.72 | 1.60 | | | | | | |
| W40/19 | Beam 7 | Sunny | Cured at | 10.09.2007 | 09.10.2007 | 09.10.2007 | 52 | 9.43 | 12.90 | 11.66 | 10.69 | 11.17 | 13.17 | 9.39 | 9.23 | 9.34 | 9.48 | 9.36 | 1.11 | 1.37 | 1.88 | 1.03 | 1.35 | 1.41 | 24.97 |

| | Water Sorptivity | | | Oxygen Permeability | | | Chloride Conductivity |
|---|---|---|---|---|---|---|---|
| average (individual) | | 8.73 | | | 9.80 | | |
| cov | 40 | 15.57 | 10 | 40 | 2.79 | 10 | 36 |
| | 24 | | | 24 | | | 24 |
| average (overall) | 12 | 8.73 | | 12 | 9.80 | | 12 |
| cov | | 16.83 | | | 2.88 | | |

# ANNEXURE 4 - BLACK MFOLOZI RIVER BRIDGE DURABILITY TESTING RESULTS

**STRUCTURAL CONCRETE DURABILITY RESULTS**
**BLACK MFOLOZI RIVER BRIDGE**

1 DESIGN MIXES

| Grade | Description | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV | Oxygen Permeability Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| W30/19 | Wet Cured | 2007/08/20 | 2007/09/19 | 2007/09/19 | | 6.75 | 4.89 | 4.85 | | 5.50 | 19.75 | 10.04 | 10.26 | 10.44 | | 10.25 | 1.96 |
| | | | | | | | | | | | | | | | | | |
| W30/19 | Wet Cured | 2007/09/19 | 2007/09/19 | 2007/09/19 | | 9.99 | 6.88 | 7.01 | | 7.96 | 22.10 | 10.08 | 10.18 | 10.31 | | 10.19 | 1.13 |
| | | | | | | | | | | | 6 | | | | | | 6 |

2 TRIAL PANELS

| Grade | Cast type | Weather conditions | Curing Regime | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV | Oxygen Permeability Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| W40/19 | Vertical, for pre-cast beams 14-15 | Cloudy, bit of sun | 24 h in forms, curing compound sprayed 1h after stripping | 2007/10/11 | 2007/11/20 | 2007/11/20 | 55.5 | 7.11 | 7.64 | 7.75 | 7.73 | 7.56 | 4.00 | 10.26 | 10.17 | 9.67 | 10.13 | 10.06 | 2.62 |
| W30/19 | Vertical, for pier 1 | Cloudy, dry+windy | 24 h in forms, curing compound sprayed 1h after stripping | 2007/10/10 | 2007/11/20 | 2007/11/20 | 57.9 | 5.56 | 6.33 | 6.01 | 7.13 | 6.26 | 10.58 | 9.77 | 10.48 | 10.27 | 10.39 | 10.23 | 3.10 |
| W30/19 | Horizontal, deck span 3 (pier 2-3) | Cloudy, but dry | 24 h in forms, curing compound sprayed 1h after stripping | 2008/03/17 | 2008/07/18 | 2008/09/10 | 51.8 | 6.5 | 5.51 | 5.69 | 5.44 | 5.79 | 8.44 | 9.92 | 10.43 | 10.5 | 10.64 | 10.37 | 3.03 |
| W30/19 | Vertical for abutment walls | Sunny | 24 h in forms, curing compound sprayed 1h after stripping | 2007/11/12 | 2008/07/18 | 2008/09/10 | 57.4 | 3.85 | 4.32 | 4.37 | 3.86 | 4.10 | 6.92 | 10.15 | 10.44 | 10.45 | 10.59 | 10.41 | 1.78 |

| | | | | | | |
|---|---|---|---|---|---|---|
| average (individual) | 5.93 | 16 | | average (individual) | 10.27 | 16 |
| cov | 24.12 | 4 | | cov | 1.55 | 4 |
| average (overall) | 5.93 | | | average (overall) | 10.27 | |
| cov | 22.66 | | | cov | 2.77 | |

3 TEST CUBES (AIR cured)

| Grade | Element | Weather conditions | Curing Regime | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov | Oxygen Permeability Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| W40/19 | Pre-Cast beam 34 | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2007/11/15 | 2008/02/25 | 2008/02/25 | 54.1 | 3.91 | 3.33 | | | 3.62 | 11.33 | 10.59 | 10.42 | | | 10.51 | 1.14 |
| W40/19 | Pre-Cast beam 35 | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2007/11/15 | 2008/02/25 | 2008/02/25 | 53.2 | 4.27 | 3.01 | | | 3.64 | 24.48 | 10.43 | 10.37 | | | 10.40 | 0.41 |
| W40/19 | Pre-Cast beam 36 | Sunny, hot | 24 h in moulds, curing compound sprayed 1h after | 2007/11/19 | 2008/02/25 | 2008/02/25 | 59.8 | 5.1 | 5.26 | | | 5.18 | 2.18 | 9.9 | 10.18 | | | 10.04 | 1.97 |
| W40/19 | Pre-Cast beam 37 | Sunny, hot | 24 h in moulds, curing compound sprayed 1h after | 2007/11/19 | 2008/02/25 | 2008/02/25 | 58.1 | 6.77 | 7.87 | | | 7.32 | 10.63 | 10.06 | 9.75 | | | 9.91 | 2.21 |
| W40/19 | Pre-Cast beam 38 | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2007/11/24 | 2008/02/25 | 2008/02/25 | 56.6 | 4.45 | 6.13 | | | 5.29 | 22.46 | 10.46 | 10.46 | | | 10.46 | 0.00 |
| W40/19 | Pre-Cast beam 39 | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2007/11/24 | 2008/02/25 | 2008/02/25 | 65.0 | 5.88 | 7.1 | | | 6.49 | 13.29 | 10.63 | 10 | | | 10.32 | 4.32 |
| W40/19 | Pre-Cast beam 40 | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2007/11/27 | 2008/02/25 | 2008/02/25 | 62.9 | 6.08 | 6.77 | | | 6.43 | 7.59 | 10.22 | 10.66 | | | 10.44 | 2.98 |
| W40/19 | Pre-Cast beam 42 | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2007/11/29 | 2008/02/25 | 2008/02/25 | 69.1 | 5.57 | 5.02 | | | 5.30 | 7.34 | 10.8 | 10.39 | | | 10.60 | 2.74 |
| W40/19 | Pre-Cast beam 43 | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2007/11/29 | 2008/02/25 | 2008/02/25 | 64.8 | 6.57 | 4.81 | | | 5.69 | 21.87 | 10.62 | 10.45 | | | 10.54 | 1.14 |
| W40/19 | Pre-Cast beam 44 | cloudy, few drops | 24 h in moulds, curing compound sprayed 1h after | 2007/12/04 | 2008/02/25 | 2008/02/25 | 57.2 | 5.81 | 5.14 | | | 5.48 | 8.65 | 10.08 | 9.91 | | | 10.00 | 1.20 |
| W40/19 | Pre-Cast beam 45 | cloudy, few drops | 24 h in moulds, curing compound sprayed 1h after | 2007/12/04 | 2008/02/25 | 2008/02/25 | 62.5 | 4.97 | 4.56 | | | 4.77 | 6.08 | 10.09 | 10.21 | | | 10.15 | 0.84 |
| W40/19 | Pre-Cast beam 46 | Overcasted | 24 h in moulds, curing compound sprayed 1h after | 2007/12/10 | 2008/02/25 | 2008/02/25 | 57.8 | 5.39 | 4.73 | | | 5.06 | 9.22 | 10.42 | 10.16 | | | 10.29 | 1.79 |
| W40/19 | Pre-Cast beam 47 | Overcasted | 24 h in moulds, curing compound sprayed 1h after | 2007/12/10 | 2008/02/25 | 2008/02/25 | 62.2 | 4.25 | 6.88 | | | 5.57 | 33.42 | 10.21 | 10.04 | | | 10.13 | 1.19 |
| W40/19 | Pre-Cast beam 48 | Cloudy with sun | 24 h in moulds, curing compound sprayed 1h after | 2008/01/10 | 2008/02/25 | 2008/02/25 | 56.7 | 3.23 | 3.66 | | | 3.45 | 8.83 | 10.12 | 10.19 | | | 10.16 | 0.49 |
| W40/19 | Pre-Cast beam 49 | Cloudy with sun | 24 h in moulds, curing compound sprayed 1h after | 2008/01/10 | 2008/02/25 | 2008/02/25 | 56.0 | 3.85 | 4.11 | | | 3.98 | 4.62 | 10.45 | 10.08 | | | 10.27 | 2.55 |
| W40/19 | Pre-Cast beam 50 | Overcasted | 24 h in moulds, curing compound sprayed 1h after | 2008/01/14 | 2008/02/25 | 2008/02/25 | 58.4 | 2.96 | 2.2 | | | 2.58 | 20.83 | 9.94 | 9.86 | | | 9.90 | 0.57 |
| W40/19 | Pre-Cast beam 51 | Overcasted | 24 h in moulds, curing compound sprayed 1h after | 2008/01/14 | 2008/02/25 | 2008/02/25 | 52.1 | 5.06 | 2.52 | | | 3.79 | 47.39 | 9.5 | 10.13 | | | 9.82 | 4.54 |
| W40/19 | Pre-Cast beam 52 | Cloudy with sun | 24 h in moulds, curing compound sprayed 1h after | 2008/01/18 | 2008/03/19 | 2008/04/02 | 52.6 | 3.58 | 3.25 | | | 3.42 | 6.83 | 10.06 | 10.22 | | | 10.14 | 1.12 |
| W40/19 | Pre-Cast beam 53 | Cloudy with sun | 24 h in moulds, curing compound sprayed 1h after | 2008/01/18 | 2008/03/19 | 2008/04/02 | 53.3 | 4.22 | 5.41 | | | 4.82 | 17.48 | 10.08 | Damaged Slice | | | 10.08 | 0.00 |
| W40/19 | Pre-Cast beam 54 | Cloudy with wind | 24 h in moulds, curing compound sprayed 1h after | 2008/01/23 | 2008/03/19 | 2008/04/02 | 50.5 | 3.05 | 3.29 | | | 3.17 | 5.35 | 10.14 | 10.21 | | | 10.18 | 0.49 |
| W40/19 | Pre-Cast beam 55 | Cloudy with wind | 24 h in moulds, curing compound sprayed 1h after | 2008/01/23 | 2008/03/19 | 2008/04/02 | 52.4 | 4.58 | 3.24 | | | 3.91 | 24.23 | 10 | 9.91 | | | 9.96 | 0.64 |
| W40/19 | Pre-Cast beam 56 | Cloudy with sun | 24 h in moulds, curing compound sprayed 1h after | 2008/01/28 | 2008/03/19 | 2008/04/02 | 53.2 | 3.54 | 4.16 | | | 3.85 | 11.39 | Outlier | 10.41 | | | 10.41 | 0.00 |
| W40/19 | Pre-Cast beam 57 | Cloudy with sun | 24 h in moulds, curing compound sprayed 1h after | 2008/01/28 | 2008/03/19 | 2008/04/02 | 52.3 | 3.39 | 2.95 | | | 3.17 | 9.81 | 10.43 | 10.17 | | | 10.30 | 1.78 |
| W40/19 | Pre-Cast beam 58 | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/01/31 | 2008/03/19 | 2008/04/02 | 56.4 | 5.58 | 4.72 | 5.49 | 5.23 | 5.26 | 7.35 | 9.46 | 9.75 | 9.46 | 9.67 | 9.59 | 1.54 |
| W40/19 | Pre-Cast beam 60 | Cloudy with sun + wind | 24 h in moulds, curing compound sprayed 1h after | 2008/02/07 | 2008/03/19 | 2008/04/02 | 52.6 | 3.8 | 3.9 | | | 3.85 | 1.84 | 10.2 | 10.62 | | | 10.41 | 2.85 |
| W40/19 | Pre-Cast beam 61 | Cloudy with sun + wind | 24 h in moulds, curing compound sprayed 1h after | 2008/02/07 | 2008/03/19 | 2008/04/02 | 50.3 | 3.74 | 3.14 | | | 3.44 | 12.33 | 10.27 | 10.07 | | | 10.17 | 1.39 |
| W40/19 | Pre-Cast beam 62 | Cloudy with sun + wind | 24 h in moulds, curing compound sprayed 1h after | 2008/02/11 | 2008/03/19 | 2008/04/02 | 53.2 | 4.9 | 3.71 | | | 4.31 | 19.55 | 9.87 | 10.09 | | | 9.98 | 1.56 |
| W40/19 | Pre-Cast beam 63 | Cloudy with sun + wind | 24 h in moulds, curing compound sprayed 1h after | 2008/02/11 | 2008/03/19 | 2008/04/02 | 49.6 | 5.17 | 6.23 | | | 5.70 | 13.15 | 9.85 | 9.85 | | | 9.85 | 0.00 |
| W40/19 | Pre-Cast beam 64 | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2008/02/13 | 2008/03/19 | 2008/04/02 | 51.3 | 7.28 | 6.38 | | | 6.83 | 9.32 | 9.37 | 9.44 | | | 9.41 | 0.53 |
| W40/19 | Pre-Cast beam 65 | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2008/02/13 | 2008/03/19 | 2008/04/02 | 44.9 | 4.73 | 3.26 | | | 4.00 | 26.02 | 9.6 | 9.52 | | | 9.56 | 0.59 |
| W40/19 | Pre-Cast beam 66 | Sunny with few clouds | 24 h in moulds, curing compound sprayed 1h after stripping, cured in open at | 2008/02/18 | 2008/03/19 | 2008/04/02 | 52.0 | 4.56 | | | | 4.56 | #DIV/0! | 9.63 | 9.79 | | | 9.71 | 1.17 |
| | | | | 2008/02/18 | 2008/03/19 | 2008/04/02 | 52.0 | 4.32 | 4.32 | | | 4.32 | 0.00 | 9.88 | 9.58 | | | 9.73 | 2.18 |
| W40/19 | Pre-Cast beam 67 | Sunny with few clouds | 24 h in moulds, curing compound sprayed 1h after | 2008/02/18 | 2008/03/19 | 2008/04/02 | 53.9 | 5.98 | | | | 5.98 | #DIV/0! | 9.88 | 9.58 | | | 9.73 | 2.18 |
| W40/19 | Pre-Cast beam 68 | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/02/21 | 2008/03/19 | 2008/04/02 | 51.3 | 4.93 | 4.87 | 5.41 | 5.67 | 5.22 | 7.38 | 9.23 | 9.37 | 9.47 | 9.00 | 9.27 | 2.20 |
| W40/19 | Pre-Cast beam 69 | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/02/21 | 2008/06/24 | 2008/07/29 | 46.3 | 3.9 | 4.34 | | | 4.12 | 7.55 | 10.16 | 10.54 | | | 10.35 | 2.60 |
| W40/19 | Pre-Cast beam 70 | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2008/02/25 | 2008/06/24 | 2008/07/29 | 52.1 | 4.96 | 4.55 | | | 4.76 | 6.10 | 9.51 | 9.19 | | | 9.35 | 2.42 |
| W40/19 | Pre-Cast beam 71 | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2008/02/25 | 2008/06/24 | 2008/07/29 | 53.0 | 3.66 | 4.06 | | | 3.86 | 7.33 | 9.7 | 9.6 | | | 9.65 | 0.73 |
| W40/19 | Pre-Cast beam 73 | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2008/02/27 | 2008/06/24 | 2008/07/29 | 58.5 | 4.26 | 4.04 | | | 4.15 | 3.75 | 9.86 | 10.01 | | | 9.94 | 1.07 |
| W40/19 | Pre-Cast beam 75 | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/03/06 | 2008/06/24 | 2008/07/29 | 41.5 | 2.99 | 3.16 | | | 3.08 | 3.91 | 9.98 | 10.07 | | | 10.03 | 0.63 |
| W40/19 | Pre-Cast beam 76 | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/03/12 | 2008/06/24 | 2008/07/29 | 46.0 | 3.73 | 3.51 | | | 3.62 | 4.30 | 10.28 | 10.2 | | | 10.24 | 0.55 |
| W40/19 | Pre-Cast beam 77 | Heavily overcasted | 24 h in moulds, curing compound sprayed 1h after | 2008/03/13 | 2008/06/24 | 2008/07/29 | 47.5 | 3.72 | 4.32 | | | 4.02 | 10.55 | 10.25 | 9.92 | | | 10.09 | 2.31 |
| W40/19 | Pre-Cast beam 78 | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2008/03/15 | 2008/06/24 | 2008/07/29 | 60.0 | 6.28 | 3.77 | | | 5.03 | 35.32 | 10.11 | 10.18 | | | 10.15 | 0.49 |
| W40/19 | Pre-Cast beam 79 | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2008/03/15 | 2008/06/24 | 2008/07/29 | 47.1 | 4.32 | 5.26 | | | 4.79 | 13.88 | 10.18 | 10.44 | | | 10.31 | 1.78 |
| W40/19 | Pre-Cast beam 80 | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/03/28 | 2008/06/24 | 2008/07/29 | 50.8 | 5.79 | 5.28 | | | 5.54 | 6.52 | 9.71 | 9.61 | | | 9.66 | 0.73 |
| W40/19 | Pre-Cast beam 81 | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/03/28 | 2008/06/24 | 2008/07/29 | 46.0 | 3.79 | 4.21 | | | 4.00 | 7.42 | 9.74 | 10.05 | | | 9.90 | 2.22 |
| W30/19 | West abutment: abutment wall | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2007/11/12 | 2008/02/25 | 2008/02/25 | 57.4 | 6.9 | 5.51 | | | 6.21 | 15.84 | 9.91 | 10.15 | | | 10.03 | 1.69 |
| W30/19 | West abutment: down stream wing wall | Sunny with few clouds | 24 h in moulds, curing compound sprayed 1h after | 25/11/2007 (Sunday) | 2008/02/25 | 2008/02/25 | 62.6 | 5.35 | 4.92 | | | 5.14 | 5.92 | 10.11 | 10.08 | | | 10.10 | 0.21 |
| W30/19 | West abutment: up stream wing wall | Drizzling | 24 h in moulds, curing compound sprayed 1h after | 2007/11/27 | 2008/02/25 | 2008/02/25 | 60.7 | 4.23 | 4.41 | | | 4.32 | 2.95 | 10.44 | 10.23 | | | 10.34 | 1.44 |
| W30/19 | East abutment: base | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2007/12/11 | 2008/02/25 | 2008/02/25 | 50.5 | 4.92 | 4.11 | 5.02 | 3.51 | 4.39 | 16.27 | 10.36 | 10.74 | 10.36 | 10.32 | 10.45 | 1.89 |
| W30/19 | East abutment: downstream wing wall | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2008/02/15 | 2008/03/19 | 2008/04/02 | 51.7 | 5.43 | 5.05 | 5.61 | 7.03 | 5.78 | 14.97 | 9.69 | 10.3 | 10.04 | 9.8 | 9.96 | 2.72 |
| W30/19 | East abutment: upstream wing wall | Sunny with few clouds | 24 h in moulds, curing compound sprayed 1h after | 2008/02/18 | | | 53.2 | The durability cube was made (WU-7) and left at the structure | | | | | | | | | | | |
| W30/19 | Pier 1 pier wall | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2007/10/10 | | | 57.9 | No test cubes for durability were made | | | | | | | | | | | |
| W30/19 | Pier 2 pier wall | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2007/11/23 | | | 50.3 | No test cubes for durability were made | | | | | | | | | | | |
| W30/19 | Pier 3 pier wall | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2008/01/17 | 2008/03/19 | 2008/04/02 | 47.9 | 4.93 | 2.54 | 5.41 | 2.64 | 3.88 | 38.74 | 9.88 | 9.68 | 9.65 | 9.42 | 9.66 | 1.95 |
| W30/19 | Pier 4 pier wall | Cloudy, but dry vibration table | 24 h in moulds, curing compound sprayed 1h after | 2008/07/17 | 2008/09/17 | 2008/10/04 | 40.5 | 7.11 | 8.2 | 4.16 | 4.59 | 6.02 | 32.47 | 9.31 | outlier | 10.11 | 10.17 | 9.86 | 4.87 |
| W30/19 | Pier 5 pier wall | Sunny with few clouds | 24 h in moulds, curing compound sprayed 1h after | 2008/07/10 | 2008/09/17 | 2008/10/04 | 31.3 | 3.7 | 2.91 | 4.07 | 3.91 | 3.65 | 14.11 | 10.15 | 10.24 | 10.04 | 10.22 | 10.16 | 0.89 |
| W30/19 | Pier 6 **pile cap** | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/06/06 | 2008/07/14 | 2008/07/14 | 32.0 | 5.47 | 3.91 | | | 4.69 | 23.52 | 9.74 | 9.62 | | | 9.68 | 0.88 |
| W30/19 | Pier 6 pier wall | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/06/11 | 2008/07/14 | 2008/07/14 | 32.5 | 7.42 | 6.28 | 8.05 | 5.81 | 6.89 | 14.91 | 9.64 | 9.67 | 9.56 | 9.8 | 9.67 | 1.03 |
| W30/19 | Pier 7 pier wall | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/05/28 | 2008/07/14 | 2008/07/14 | 30.7 | 5.13 | 3.94 | 5.13 | 4.14 | 4.59 | 13.84 | 9.57 | 9.82 | 9.74 | 9.78 | 9.73 | 1.13 |
| W30/19 | Pier 8 pier wall | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/03/19 | 2008/06/24 | 2008/07/29 | 43.6 | 5.18 | 4.91 | 6.84 | 7.61 | 6.14 | 21.22 | 10.12 | 9.95 | 10.06 | 9.98 | 10.03 | 0.77 |
| W30/19 | In-situ concrete deck span 1 west abut - pier 1 | Sunny with few clouds | 24 h in moulds, curing compound sprayed 1h after | 2008/01/22 | 2008/03/19 | 2008/04/02 | 49.6 | 4.76 | 3.36 | 4.03 | 4.31 | 4.12 | 14.25 | 10.02 | 10.4 | 10.15 | 9.96 | 10.13 | 1.93 |
| W30/19 | In-situ concrete deck span 2 pier 1 - pier 2 | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2007/12/06 | | | 61.5 | Results lost | | | | | | | | | | | |
| W30/19 | In-situ concrete deck span 3 pier 2 - pier 3 | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2008/03/17 | 2008/06/24 | 2008/07/29 | 51.8 | 4.98 | 4.93 | 5.64 | 5.42 | 5.24 | 6.57 | 9.99 | 10 | 9.65 | 10.01 | 9.91 | 1.77 |
| W30/19 | In-situ concrete deck span 4 pier 3 - pier 4 | Sunny vibration table | 24 h in moulds, curing compound sprayed 1h after | 2008/08/15 | | | 51.2 | Results lost | | | | | | | | | | | |
| W30/19 | In-situ concrete deck span 5 pier 4 - pier 5 | Sunny vibration table | 24 h in moulds, curing compound sprayed 1h after | 2008/08/12 | 2008/09/17 | 2008/10/04 | 50.7 | 6.44 | 7.09 | 6.2 | 7.29 | 6.76 | 7.67 | 9.37 | 9.77 | 10.04 | 9.89 | 9.77 | 2.94 |
| W30/19 | In-situ concrete deck span 6 pier 5 - pier 6 | Sunny vibration table | 24 h in moulds, curing compound sprayed 1h after | 2008/08/07 | 2008/09/17 | 2008/10/04 | 42.1 | 6.08 | 7.67 | | | 6.88 | 16.35 | 9.25 | 9.05 | | | 9.15 | 1.55 |
| W30/19 | In-situ concrete deck span 7 pier 6 - pier 7 | Sunny + windy vibration table | 24 h in moulds, curing compound sprayed 1h after | 2008/08/04 | 2008/09/17 | 2008/10/04 | 39.0 | 9.15 | 11.38 | 7.6 | 6.85 | 8.75 | 22.88 | 9.64 | 10.08 | | | 9.86 | 3.16 |
| W30/19 | In-situ concrete deck span 8 pier 7 - pier 8 | Sunny + windy vibration table | 24 h in moulds, curing compound sprayed 1h after | 2008/07/30 | 2008/09/17 | 2008/10/04 | 33.3 | 7.19 | 8.26 | 6.51 | 8.71 | 7.67 | 13.05 | 9.1 | 9.24 | 9.23 | 8.95 | 9.13 | 1.49 |
| W30/19 | In-situ concrete deck span 9 pier 8 - east abut | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/04/24 | 2008/06/24 | 2008/07/29 | 33.6 | 4.73 | 6.01 | 5.91 | 5.81 | 5.62 | 10.61 | 10.06 | 9.99 | 9.81 | 9.8 | 9.92 | 1.31 |
| W40/19 | Pier 4 - pile P1 | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/02/29 | 2008/06/24 | 2008/07/29 | 57.9 | 6.07 | 5.53 | 5.11 | 4.59 | 5.33 | 11.80 | 8.91 | 9.08 | 9.87 | 9.7 | 9.39 | 4.97 |
| W40/19 | Pier 4 - pile P2 | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2008/02/25 | 2008/06/24 | 2008/07/29 | 39.3 | 5.98 | 4.49 | | | 5.24 | 20.13 | 8.3 | 10.31 | | | 9.31 | 15.27 |
| W40/19 | Pier 4 - pile P3 | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/04/05 | 2008/06/24 | 2008/07/29 | 41.6 | 4.59 | 3.35 | | | 3.97 | 22.09 | 9.74 | 9.98 | | | 9.86 | 1.72 |
| W40/19 | Pier 5- pile P4 | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2008/03/05 | 2008/07/14 | 2008/07/14 | 42.0 | 1.87 | 2.51 | | | 2.19 | 20.66 | 10.32 | 9.96 | | | 10.14 | 2.51 |
| W40/19 | Pier 5- pile P5 | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/03/03 | 2008/06/24 | 2008/07/29 | 35.3 | 2.43 | 5.15 | | | 3.79 | 50.75 | 9.45 | 9.44 | | | 9.45 | 0.07 |
| W40/19 | Pier 6 - pile P7 | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/03/12 | 2008/07/14 | 2008/07/14 | 43.3 | 3.98 | 4.58 | | | 4.28 | 9.91 | 9.87 | 9.68 | | | 9.78 | 1.37 |
| W40/19 | Pier 6 - pile P8 | Sunny | 24 h in moulds, curing compound sprayed 1h after | 2008/03/08 | 2008/06/24 | 2008/07/29 | 43.6 | 2.9 | 3.22 | | | 3.06 | 7.39 | 9.8 | 10.02 | | | 9.91 | 1.57 |
| W40/19 | Pier 7 - pile P11 | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2008/03/17 | 2008/06/24 | 2008/07/29 | 48.6 | 2.96 | 3.08 | | | 3.02 | 2.81 | 10.72 | 10.7 | | | 10.71 | 0.13 |
| W40/19 | Pier 7 - pile P12 | Cloudy, but dry | 24 h in moulds, curing compound sprayed 1h after | 2008/04/08 | 2008/06/24 | 2008/07/29 | 50.0 | 3.13 | 4.83 | | | 3.98 | 30.20 | 9.64 | 9.35 | | | 9.50 | 2.16 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 178 | average (individual) | 4.77 | 73 | 175 | average (individual) | 9.97 | 73 | |
| 89 | cov | 26.60 | | 87.5 | cov | 3.57 | | |
| | average (overall) | 4.92 | | | average (overall) | 9.93 | | |
| | cov | 29.08 | 176.5 | | cov | 4.08 | | |

4 IN-SITU (CORES)

| Grade | Element | Weather conditions | Curing Regime | Core location | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov | Oxygen Permeability Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| W30/19 | Pier 1 - pier wall west side | Cloudy, but dry | 24 h in forms, curing compound sprayed 1h after stripping | 1m above ground level, north facing | 2007/10/10 | 2008/11/01 | 2008/11/25 | 57.9 | 8.67 | 6.84 | 11.23 | 6.15 | 8.22 | 27.60 | 9.96 | 9.22 | 9.74 | 9.5 | 9.61 | 3.31 |
| W30/19 | Pier 2- pier wall west side | Cloudy, but dry | 24 h in forms, curing compound sprayed 1h after stripping | 1m above ground level, north facing | 2007/11/23 | 01/11/2008 | 25/11/2008 | 50.3 | 11.45 | 8.37 | 5.63 | 7.74 | 8.30 | 29.00 | cracked | 9.99 | 9.6 | 9.86 | 9.82 | 2.02 |
| W30/19 | Pier 3- pier wall east side | Cloudy, but dry | 24 h in forms, curing compound sprayed 1h after stripping | 1m above ground level, north facing | 2008/01/17 | 01/11/2008 | 25/11/2008 | 47.9 | 6.9 | 5.12 | 8.54 | 11.96 | 8.13 | 35.80 | cracked | 9.95 | 9.5 | 9.45 | 9.63 | 2.86 |
| W30/19 | Pier 4- pier wall west side | Cloudy, but dry | 24 h in forms, curing compound sprayed 1h after stripping | 1m above ground level, north facing | 2008/07/17 | 01/11/2008 | 25/11/2008 | 40.5 | 10.34 | 9.92 | 10.21 | 11.52 | 10.50 | 6.71 | 9.79 | 9.45 | 9.14 | 8.8 | 9.30 | 4.56 |
| W30/19 | Pier 5- pier wall east side | Sunny with few clouds | 24 h in forms, curing compound sprayed 1h after stripping | 1m above ground level, north facing | 2008/07/10 | 01/11/2008 | 25/11/2008 | 31.3 | 9.99 | 13.32 | 10.53 | 14.17 | 12.00 | 17.11 | 9.27 | 9.27 | 8.89 | 9.13 | 9.14 | 1.96 |
| W30/19 | Pier 6- pier wall west side | Sunny | 24 h in forms, curing compound sprayed 1h after stripping | 1m above ground level, north facing | 2008/06/11 | 01/11/2008 | 25/11/2008 | 32.5 | 12.21 | 10.95 | 14.72 | 16.26 | 13.54 | 17.73 | 9 | 9.93 | 8.65 | 8.92 | 9.13 | 6.11 |
| W30/19 | Pier 7- pier wall east side | Sunny | 24 h in forms, curing compound sprayed 1h after stripping | 1m above ground level, north facing | 2008/05/28 | 01/11/2008 | 25/11/2008 | 30.7 | 14.49 | 13.62 | 12.09 | 14.36 | 13.64 | 8.08 | 8.98 | 9.08 | 9.07 | 8.29 | 8.86 | 4.28 |
| W30/19 | Pier 8- pier wall west side | Sunny | 24 h in forms, curing compound sprayed 1h after stripping | 1m above ground level, north facing | 2008/03/19 | 01/11/2008 | 25/11/2008 | 43.6 | 12.53 | 10.29 | 12.1 | 9.87 | 11.20 | 11.73 | 8.83 | 9.09 | 9.19 | 9.44 | 9.14 | 2.76 |
| W30/19 | East Abutment; base | Sunny | 24 h in forms, curing compound sprayed 1h after stripping | 1m above ground level, north facing | 2007/12/11 | 2008/03/07 | 2008/03/07 | 50.6 | 2.57 | 4.51 | 5.08 | 3.4 | 3.89 | 28.87 | 10.15 | crack found in disc | 9.58 | 10.15 | 9.96 | 3.30 |
| W30/29 | West Abutment; wall | Sunny | 24 h in forms, curing compound sprayed 1h after stripping | 2m above ground level, north facing | 2007/11/12 | 2008/03/07 | 2008/03/07 | 57.4 | 4.98 | 5.12 | | | 5.05 | 1.96 | 9.76 | 10.15 | | | 9.96 | 2.77 |
| W40/19 | Deck-Span 4 (pier 3 - pier 4) Beam **37** | Sunny, hot | 24 h in forms, curing compound sprayed 1h after stripping | web | 2007/11/19 | 2008/07/18 | 2008/09/10 | 58.1 | 5.98 | 4.86 | | | 5.42 | 14.61 | 10.19 | 9.87 | | | 10.03 | 2.26 |
| W40/19 | Deck-Span 4 (pier 3 - pier 4) Beam **46** | Overcasted | 24 h in forms, curing compound sprayed 1h after stripping | web | 2007/12/10 | 18/07/2008 | 10/09/2008 | 57.8 | 4.92 | 5.11 | | | 5.02 | 2.68 | 9.49 | 10.28 | | | 9.89 | 5.65 |
| W40/19 | Deck-Span 4 (pier 3 - pier 4) Beam **50** | Overcasted | 24 h in forms, curing compound sprayed 1h after stripping | web | 2008/01/14 | 2008/05/07 | 2008/05/22 | 58.4 | 7.03 | 5.22 | | | 6.13 | 20.90 | 8.71 | 10.1 | | | 9.41 | 10.45 |
| W40/19 | Deck-Span 4 (pier 3 - pier 4) Beam **70** | Cloudy, but dry | 24 h in forms, curing compound sprayed 1h after stripping | web | 2008/02/25 | 18/07/2008 | 10/09/2008 | 52.1 | 4.57 | 5.6 | | | 5.09 | 14.32 | 9.67 | 9.4 | | | 9.54 | 2.00 |
| W40/19 | Deck-Span 5 (pier 4 - pier 5) Beam **31** | Cloudy, drizzling | 24 h in forms, curing compound sprayed 1h after stripping | web | 2007/11/09 | 18/07/2008 | 10/09/2008 | 61.8 | 4.26 | 6.15 | | | 5.21 | 25.68 | 9.83 | 9.87 | | | 9.85 | 0.29 |
| W40/19 | Deck-Span 5 (pier 4 - pier 5) Beam **77** | Heavily overcasted | 24 h in forms, curing compound sprayed 1h after stripping | web | 2008/03/13 | 18/07/2008 | 10/09/2008 | 47.5 | 6.45 | 6.54 | | | 6.50 | 0.98 | 10.07 | 9.75 | | | 9.91 | 2.28 |
| W40/19 | Deck-Span 6 (pier 5 - pier 6) in situ concrete | Sunny vibration table used | 24 h in forms, curing compound sprayed 1h after stripping | Top of deck | 2008/08/07 | 39753 | 39777 | 42.1 | 11.3 | 9.39 | 9.14 | 11.43 | 10.32 | 11.81 | 9.51 | cracked | 8.47 | 9.34 | 9.11 | 6.13 |
| W40/19 | Deck-Span 7 (pier 6 - pier 7) Beam **56** | Cloudy with sun | 24 h in forms, curing compound sprayed 1h after stripping | web | 2008/01/28 | 2008/05/07 | 2008/05/22 | 53.2 | 5.89 | | | | 5.89 | | 9.82 | | | | 9.82 | |
| W40/19 | Deck-Span 7 (pier 6 - pier 7) Beam **62** | Cloudy with sun + wind | 24 h in forms, curing compound sprayed 1h after stripping | web | 2008/02/11 | 2008/05/07 | 2008/05/22 | 53.2 | 5.68 | | | | 5.68 | | 10.1 | | | | 10.10 | |
| W40/19 | Deck-Span 8 (pier 7 - pier 8) Beam **45** | cloudy, few drops | 24 h in forms, curing compound sprayed 1h after stripping | web | 2008/12/04 | 2008/05/07 | 2008/05/22 | 62.5 | 5.16 | | | | 5.16 | | 9.88 | | | | 9.88 | |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| | average (individual) | 7.74 | | | average | 9.60 | |
| 57 | cov | 39.78 | 20 | 53 | cov | 3.92 | 20 |
| | average (overall) | 8.64 | | | average | 9.49 | |
| | cov | 39.98 | | | cov | 5.16 | |

# ANNEXURE 5 - RICHMOND ROAD INTERCHANGE BRIDGE DURABILITY TESTING RESULTS

**STRUCTURAL CONCRETE DURABILITY RESULTS**
**RICHMOND ROAD INTERCHANGE BRIDGE**

**1 DESIGN MIXES**

| Grade | Description | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity | | | | | | Oxygen Permeability | | | | | | Chloride Conductivity | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV |
| W35/19 | Substructures | 28..05.2008 | 25.06.2008 | 25.06.2008 | 47 | 4.12 | 6.72 | 7.36 | 9.68 | 6.97 | 32.80 | 9.68 | 9.23 | 9.23 | | 9.38 | 2.77 | 0.34 | 0.43 | 0.38 | 0.42 | 0.39 | 10.48 |
| | | | | | | | | | | | | | | | | | | | | | | | |
| W45/19 | Deck | 28.05.2008 | 25.06.2008 | 25.06.2008 | 52.5 | 3.24 | 6.72 | 8.2 | 4.95 | 5.78 | 37.23 | 10.25 | 10.27 | 10.55 | 9.96 | 10.26 | 2.35 | 0.16 | 0.23 | 0.12 | 0.14 | 0.16 | 29.46 |
| | | | | | | | 8 | | | | | | 7 | | | | | | 8 | | | | |

**2 TRIAL PANELS**

| Grade | Cast type | Weather conditions | Curing Regime | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity | | | | | | Oxygen Permeability | | | | | | Chloride Conductivity | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV |
| W35/19 | Substructures - Vertical | Fine and warm | 24 h in forms, curing compound sprayed 1h after stripping | 08.05.2008 | 17.06.2008 | 03.07.2008 | 43.7 | 4.33 | 5.15 | 4.33 | 4.77 | 4.65 | 8.51 | 9.98 | 10.05 | 10 | 9.93 | 9.99 | 0.50 | | | | | | |
| W45/19 | Decks - Horizontal | Fine and warm | 24 h in forms, curing compound sprayed 1h after stripping | 31.07.2008 | 29.08.2008 | 08.09.2008 | 50.6 | 5.65 | 5.44 | 5.43 | 5.61 | 5.53 | 2.06 | 9.78 | 10.01 | 9.47 | 9.52 | 9.70 | 2.58 | 0.20 | 0.20 | 0.22 | 0.22 | 0.21 | 5.50 |
| W45/19 | Decks - vertical | Fine and warm | 24 h in forms, curing compound sprayed 1h after stripping | 31.07.2008 | 29.08.2008 | 08.09.2008 | 50.6 | 3.89 | 4.72 | 6.40 | 4.34 | 4.84 | 22.65 | 10.46 | 10.40 | 10.96 | 10.30 | 10.53 | 2.79 | 0.17 | 0.14 | 0.17 | 0.19 | 0.17 | 12.31 |
| W45/19 | Balustrades - vertical | Fine and warm | 24 h in forms, curing compound sprayed 1h after stripping | 24.10.2008 | 21.11.2008 | 21.11.2008 | | 5.41 | 5.79 | 7.02 | 7.01 | 6.31 | 13.18 | 9.92 | 10.08 | 10.43 | 10.68 | 10.28 | 3.33 | | | | | | |
| W35/19 | Substructure columns - vertical | Fine and warm | 24 h in forms, curing compound sprayed 1h after stripping | 02.07.2008 | 29.08.2008 | 08.09.2008 | 26.0 | | | | | | | | | | | | | 0.20 | 0.17 | 0.15 | 0.21 | 0.18 | 15.09 |

| | Water Sorptivity | | | Oxygen Permeability | | | Chloride Conductivity |
|---|---|---|---|---|---|---|---|
| 16 | average (individual) | 5.33 | 4 | 16 | average (individual) | 10.12 | 4 |
| | cov | 14.16 | | | cov | 3.56 | 12 |
| | average (overall) | 5.33 | | | average (overall) | 10.12 | |
| | cov | 17.47 | | | cov | 3.93 | |

**3 TEST CUBES (wet cured)**

| Grade | Element | Weather conditions | Curing Regime | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity | | | | | | Oxygen Permeability | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov |
| W35/19 | A1 & A2 Bases | Fine and warm | wet cured | 16.05.2008 | 13.06.2008 | 13.06.2008 | 46.8 | 3.46 | 3.43 | 3.97 | 3.64 | 3.63 | 6.84 | 10.94 | 10.85 | 10.04 | 10.83 | 10.67 | 3.93 |
| W35/19 | D1 Base | Fine and warm | wet cured | 20.05.2008 | 17.06.2008 | 17.06.2008 | 43.7 | 3.54 | 3.63 | 5.02 | 4.95 | 4.29 | 18.89 | 10.04 | 10.08 | 10.82 | 10.22 | 10.29 | 3.51 |
| W35/19 | Test Panel | Fine and warm | wet cured | 20.05.2008 | 17.06.2008 | 17.06.2008 | | 4.33 | 5.15 | 4.33 | 4.77 | 4.65 | 8.51 | 9.98 | 10.05 | 10.00 | 9.93 | 9.99 | 0.50 |
| W35/19 | D2 Base | Fine and warm | wet cured | 21.05.2008 | 18.06.2008 | 18.06.2008 | 45.3 | 3.93 | 3.89 | 3.61 | 4.14 | 3.89 | 5.60 | 10.52 | 9.82 | 9.86 | 10.54 | 10.19 | 3.92 |
| W35/19 | B2 Base | Fine and warm | wet cured | 22.05.2008 | 19.06.2008 | 19.06.2008 | 46.8 | 3.94 | 3.71 | 3.84 | 3.70 | 3.80 | 3.01 | 10.57 | 9.35 | 9.93 | 10.63 | 10.12 | 5.96 |
| W35/19 | BI Base | Fine and warm | wet cured | 23.05.2008 | 20.06.2008 | 20.06.2008 | 43.1 | 4.41 | 4.70 | 4.59 | 5.30 | 4.75 | 8.12 | 9.97 | 10.09 | 10.62 | 10.22 | 10.23 | 2.76 |
| W35/19 | A1 & D1 Columns (1st Lift) | Fine and warm | wet cured | 26.05.2008 | 23.06.2008 | 23.06.2008 | 45.9 | 3.84 | 4.29 | 3.56 | 4.92 | 4.15 | 14.29 | 9.46 | 10.17 | 10.49 | 10.45 | 10.14 | 4.70 |
| W35/19 | C2 Base | Fine and warm | wet cured | 27.05.2008 | 24.06.2008 | 24.06.2008 | 45.9 | 5.23 | 4.81 | 4.96 | 5.44 | 5.11 | 5.49 | 9.78 | 9.73 | 9.95 | 10.03 | 9.87 | 1.43 |
| W35/19 | C1 Base & A2 Column (1st Lift) | Fine and warm | wet cured | 28.05.2008 | 25.06.2008 | 25.06.2008 | 44.1 | 4.77 | 4.06 | 5.16 | 4.37 | 4.59 | 10.42 | 10.43 | 10.52 | 10.25 | 10.51 | 10.43 | 1.20 |
| W35/19 | D2 Column (1stLift) | Fine and warm | wet cured | 29.05.2008 | 26.06.2008 | 26.06.2008 | 42.2 | 3.77 | 4.31 | 4.03 | 4.32 | 4.11 | 6.38 | 9.81 | 10.31 | 10.09 | 10.30 | 10.13 | 2.32 |
| W35/19 | B1 Column (1stLift) | Fine and warm | wet cured | 02.06.2008 | 30.06.2008 | 30.06.2008 | 40.3 | 4.21 | 3.35 | 3.96 | 4.89 | 4.10 | 15.53 | 10.18 | 10.11 | 10.09 | 10.23 | 10.15 | 0.64 |
| W35/19 | C1 Column (1stLift) | Fine and warm | wet cured | 04.06.2008 | 02.07.2008 | 02.07.2008 | 31.7 | 5.52 | 5.91 | 5.21 | 5.47 | 5.53 | 5.23 | 9.88 | 9.70 | 9.69 | 9.76 | 9.76 | 0.89 |
| W35/19 | Pier D - Wall | Fine and warm | wet cured | 05.06.2008 | 03.07.2008 | 03.07.2008 | 43.2 | 4.37 | 5.59 | 5.42 | 5.43 | 5.20 | 10.77 | 10.57 | 10.33 | 10.48 | 10.53 | 10.48 | 1.00 |
| W35/19 | B2 Column (1stLift) | Fine and warm | wet cured | 06.06.2008 | 04.07.2008 | 04.07.2008 | 29.7 | * | 5.06 | 6.55 | 5.35 | 5.65 | 13.97 | * | 10.19 | 9.86 | 10.06 | 10.04 | 1.66 |
| W35/19 | C2 Column (1stLift) | Fine and warm | wet cured | 10.06.2008 | 08.07.2008 | 08.07.2008 | 39.5 | 6.45 | 4.29 | 5.90 | 5.89 | 5.63 | 16.56 | 10.30 | 10.39 | 10.63 | 10.41 | 10.43 | 1.34 |
| W35/19 | Abutment A-Crossbeam (1stLift) | Fine and warm | wet cured | 11.06.2008 | 09.07.2008 | 09.07.2008 | 44.8 | 6.87 | 7.65 | 7.80 | 4.40 | 6.68 | 23.56 | 10.10 | 9.98 | 9.39 | 10.39 | 9.97 | 4.22 |
| W35/19 | B1 Column (2ndLift) | Fine and warm | wet cured | 12.06.2008 | 10.07.2008 | 10.07.2008 | 37.4 | 6.43 | 7.62 | 7.40 | 4.37 | 6.46 | 22.97 | 10.11 | 9.91 | 9.20 | 10.20 | 9.86 | 4.60 |
| W35/19 | C1 Column (2ndLift) | Fine and warm | wet cured | 18.06.2008 | 16.07.2008 | 16.07.2008 | 33.8 | 4.61 | 4.82 | 4.76 | 5.25 | 4.86 | 5.65 | 10.01 | 10.07 | 10.03 | 10.39 | 10.13 | 1.76 |
| W35/19 | Pier B - Wall | Fine and warm | wet cured | 19.06.2008 | 17.07.2008 | 17.07.2008 | 35.5 | 4.60 | 4.73 | 4.48 | 5.31 | 4.78 | 7.69 | 10.50 | 10.40 | 9.82 | 10.39 | 10.28 | 3.01 |
| W35/19 | Abutment A - Curtain Wall | Fine and warm | wet cured | 23.06.2008 | 21.07.2008 | 21.07.2008 | 43.5 | 2.87 | 3.90 | 4.86 | 4.16 | 3.95 | 20.90 | 10.72 | 10.68 | 11.32 | 11.62 | 11.09 | 4.16 |
| W35/19 | Pier C2 Column (2ndLift) | Fine and warm | wet cured | 25.06.2008 | 23.07.2008 | 23.07.2008 | 36.0 | 4.96 | 5.52 | 3.97 | 5.02 | 4.87 | 13.33 | 10.37 | 10.13 | 10.03 | 9.95 | 10.12 | 1.80 |
| W35/19 | Base E2 | Fine and warm | wet cured | 26.06.2008 | 24.07.2008 | 24.07.2008 | 32.3 | 4.78 | 4.96 | 4.42 | 3.39 | 4.39 | 16.00 | 10.12 | 9.98 | 10.06 | 10.47 | 10.16 | 2.13 |
| W35/19 | B2 Column (2nd Lift) | Fine and warm | wet cured | 27.06.2008 | 25.07.2008 | 25.07.2008 | 35.0 | 4.36 | 5.07 | 6.34 | 6.47 | 5.56 | 18.33 | 10.34 | 10.35 | 10.06 | 9.90 | 10.16 | 2.17 |
| W35/19 | Pier E2 - Column (1stLift) | Fine and warm | wet cured | 02.07.2008 | 30.07.2008 | 30.07.2008 | 26.0 | 3.71 | 3.72 | 4.03 | 4.10 | 3.89 | 5.25 | 10.89 | 10.69 | 10.65 | 10.87 | 10.78 | 1.14 |
| W35/19 | Pier B2 Column (3rdLift) | Fine and warm | wet cured | 02.07.2008 | 30.07.2008 | 30.07.2008 | 36.8 | 4.35 | 4.64 | 4.41 | 4.57 | 4.49 | 3.01 | 10.27 | 10.09 | 10.67 | 10.82 | 10.46 | 3.25 |
| W35/19 | Pier C Wall | Fine and warm | wet cured | 03.07.2008 | 31.07.2008 | 31.07.2008 | 38.5 | 3.89 | 4.24 | 4.42 | 3.70 | 4.06 | 8.05 | 10.00 | 10.33 | 10.35 | 10.32 | 10.25 | 1.63 |
| W45/19 | Deck-Bottom Slab/ webs | Fine and warm | wet cured | 25.09.2008 | 23.10.2008 | 23.10.2008 | 52.4 | 3.48 | 3.68 | 4.50 | 4.84 | 4.13 | 15.75 | 10.50 | 10.52 | 10.90 | 10.82 | 10.69 | 1.92 |
| | | | | | | | | | 3.98 | 4.29 | 4.45 | 4.24 | 5.64 | 10.57 | 9.78 | 10.86 | 10.83 | 10.51 | 4.79 |
| W45/19 | Deck - Top Slab | Fine and warm | wet cured | 04.10.2008 | 01.11.2008 | 01.11.2008 | 51.8 | 5.02 | 6.33 | 4.82 | 5.78 | 5.49 | 12.71 | 11.13 | 10.75 | 10.74 | 10.99 | 10.90 | 1.75 |

| | Water Sorptivity | | | Oxygen Permeability | | | |
|---|---|---|---|---|---|---|---|
| 114 | average (individual) | 4.72 | 29 | 115 | average (individual) | 10.28 | 29 |
| | cov | 16.71 | | | cov | 3.08 | |
| | average (overall) | 4.72 | | | average (overall) | 10.29 | |
| | cov | 20.55 | | | cov | 3.97 | |

**4 IN-SITU (CORES)**

| Grade | Element | Weather conditions | Curing Regime | Core location | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity | | | | | | Oxygen Permeability | | | | | | Chloride Conductivity | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov |
| W35/19 | Pier A1 1st lift | Fine and warm | curing compound | top of lift 1 | 26.05.2008 | 6.08.2008 | 6.08.2008 | 44.7 | 7.09 | 6.84 | 6.25 | 5.48 | 6.42 | 11.16 | 9 | 9.71 | 10.54 | 10.48 | 9.93 | 7.32 | | | | | | |
| W35/19 | Pier D1 1st lift | Fine and warm | curing compound | top of lift 1 | 26.05.2008 | 6.08.2008 | 6.08.2008 | 44.7 | 6.42 | 9.22 | 7.75 | 7.33 | 7.68 | 15.20 | 10.05 | 9.48 | 9.77 | 9.96 | 9.815 | 2.57 | | | | | | |
| W35/19 | Pier B1 1st lift | Fine and warm | curing compound | top of lift 1 | 2.06.2008 | 6.08.2008 | 6.08.2008 | 42.9 | 10.81 | 8.5 | 7.28 | 6.18 | 8.19 | 24.24 | 9.19 | 10.07 | 9.65 | 9.68 | 9.65 | 3.73 | | | | | | |
| W35/19 | Pier C1 1st lift | Fine and warm | curing compound | top of lift 1 | 4.06.2008 | 6.08.2008 | 6.08.2008 | 30.1 | 10.18 | 11.17 | 8.63 | 14.28 | 11.07 | 21.55 | 9.4 | 9.25 | 9.21 | 9.07 | 9.23 | 1.47 | | | | | | |
| W35/19 | Pier E2 1st lift | Fine and warm | curing compound | top of lift 1 | 4.06.2008 | 6.08.2008 | 6.08.2008 | 26 | 10.27 | 9.65 | 9.12 | 10.23 | 9.82 | 5.55 | 9.78 | 9.58 | 9.27 | 9.53 | 9.54 | 2.20 | | | | | | |
| W45/19 | Deck-Bottom Slab/ webs | Fine and warm | curing compound | side of web-west | 25.09.2008 | 01.11.2008 | 01.11.2008 | 52.4 | 2.91 | 3.32 | 4.57 | 3.55 | 3.59 | 19.69 | 10.76 | 10.75 | 10.96 | 10.84 | 10.83 | 0.90 | 0.15 | 0.15 | 0.16 | 0.14 | 0.15 | 5.44 |
| | | | | | | | | | 2.82 | 2.96 | 3.10 | 4.10 | 3.25 | 17.92 | 11.08 | 10.76 | 10.86 | 10.97 | 10.92 | 1.27 | 0.17 | 0.11 | 0.15 | 0.13 | 0.14 | 18.44 |
| W45/19 | Deck - Top Slab | Fine and warm | curing compound | top of deck-east | 04.10.2008 | 01.11.2008 | 01.11.2008 | 51.8 | 2.41 | 3.27 | 2.61 | 3.00 | 2.82 | 13.68 | 10.92 | 10.49 | 10.57 | 10.85 | 10.71 | 1.96 | 0.22 | 0.19 | 0.14 | 0.13 | 0.17 | 24.96 |

| | Water Sorptivity | | | Oxygen Permeability | | | Chloride Conductivity |
|---|---|---|---|---|---|---|---|
| | average (individual) | 6.60 | | | average (individual) | 10.08 | |
| 32 | cov | 47.42 | 8 | 32 | cov | 6.43 | 8 |
| | | | | | | | 12 |
| | average (overall) | 6.60 | | | average (overall) | 10.08 | |
| | cov | 48.11 | | | cov | 6.75 | |

**5 TEST CUBES (curing compound/ air cured)**

24 hours in moulds. Curing Compound sprayed 1 hour after stripping. Cured at bridge site.

| Grade | Element | Weather conditions | Curing Regime | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity | | | | | | Oxygen Permeability | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov |
| W35/19 | Pier D wall | Fine and warm | Curing compound | 5.06.2008 | 7.08.2008 | 7.08.2008 | 43.2 | 4.37 | 5.59 | 5.42 | 5.43 | 5.20 | 10.77 | 10.57 | 10.33 | 10.48 | 10.53 | 10.48 | 1.00 |
| W35/19 | Pier B2 1st lift | Fine and warm | Curing compound | 6.06.2008 | 7.08.2008 | 7.08.2008 | 29.7 | 5.06 | 6.55 | 5.35 | | 5.65 | 13.97 | 10.19 | 9.86 | 10.06 | | 10.04 | 1.66 |
| W35/19 | C2 Column (1stLift) | Fine and warm | Curing compound | 10.06.2008 | 08.07.2008 | 08.07.2008 | 39.5 | 7.02 | 7.07 | 6.93 | 6.45 | 6.87 | 4.14 | 10.02 | 9.81 | 9.85 | 9.91 | 9.90 | 0.92 |
| W35/19 | Abutment A-Crossbeam (1stLift) | Fine and warm | Curing compound | 11.06.2008 | 09.07.2008 | 09.07.2008 | 44.8 | 5.11 | 5.92 | 5.03 | 6.34 | 5.60 | 11.36 | 10.07 | 10.16 | 10.00 | 10.11 | 10.09 | 0.67 |
| W35/19 | B1 Column (2ndLift) | Fine and warm | Curing compound | 12.06.2008 | 10.07.2008 | 10.07.2008 | 37.4 | 9.61 | 9.64 | 9.86 | 9.71 | 9.71 | 1.15 | 9.47 | 10.97 | 9.49 | 10.48 | 10.10 | 7.39 |
| W35/19 | C1 Column (2ndLift) | Fine and warm | Curing compound | 18.06.2008 | 16.07.2008 | 16.07.2008 | 33.8 | 5.36 | 5.42 | 5.69 | 5.51 | 5.50 | 2.62 | 9.92 | 10.06 | 9.90 | 9.99 | 9.97 | 0.73 |
| W35/19 | Pier B - Wall | Fine and warm | Curing compound | 19.06.2008 | 17.07.2008 | 17.07.2008 | 35.5 | 6.73 | 6.22 | 9.00 | 6.08 | 7.01 | 19.37 | 9.98 | 9.95 | 9.87 | 9.93 | 9.93 | 0.47 |
| W35/19 | Abutment A - Curtain Wall | Fine and warm | Curing compound | 23.06.2008 | 21.07.2008 | 21.07.2008 | 43.5 | 3.98 | 4.22 | 4.12 | 4.36 | 4.17 | 3.85 | 10.39 | 10.33 | 10.29 | 10.31 | 10.33 | 0.42 |
| W35/19 | Pier C2 Column (2ndLift) | Fine and warm | Curing compound | 25.06.2008 | 23.07.2008 | 23.07.2008 | 36.0 | 4.86 | 4.82 | 4.65 | 5.82 | 5.04 | 10.51 | 10.05 | 10.37 | 10.15 | 10.27 | 10.21 | 1.37 |
| W35/19 | Base E2 | Fine and warm | Curing compound | 26.06.2008 | 24.07.2008 | 24.07.2008 | 32.3 | 3.96 | 6.66 | 3.79 | 4.58 | 4.75 | 27.79 | 10.33 | 8.27 | 10.42 | 10.49 | 9.88 | 10.87 |
| W35/19 | B2 Column (2ndLift) | Fine and warm | Curing compound | 27.06.2008 | 25.07.2008 | 25.07.2008 | 35.0 | 7.92 | 4.77 | 4.59 | 4.81 | 5.52 | 28.99 | 10.13 | 10.07 | 9.97 | 10.19 | 10.09 | 0.93 |
| W35/19 | Pier E2 - Column (1stLift) | Fine and warm | Curing compound | 02.07.2008 | 30.07.2008 | 30.07.2008 | 26.0 | 9.25 | 8.77 | 14.12 | 9.91 | 10.51 | 23.31 | 9.68 | 9.76 | 9.49 | 9.67 | 9.65 | 1.18 |
| W35/19 | Pier B2 Column (3rdLift) | Fine and warm | Curing compound | 02.07.2008 | 30.07.2008 | 30.07.2008 | 36.8 | 10.05 | 10.57 | 7.26 | 7.69 | 8.89 | 18.67 | 9.56 | 9.42 | 9.64 | 9.81 | 9.61 | 1.69 |
| W35/19 | Pier C Wall | Fine and warm | Curing compound | 03.07.2008 | 31.07.2008 | 31.07.2008 | 38.5 | 9.10 | 8.40 | 5.64 | 8.93 | 8.02 | 20.12 | 9.69 | 10.05 | 9.77 | 10.05 | 9.89 | 1.90 |
| W45/19 | Deck-Bottom Slab/ webs | Fine and warm | Curing compound | 25.09.2008 | 23.10.2008 | 23.10.2008 | 52.4 | 3.56 | 4.03 | | | 3.80 | 8.76 | 10.81 | 10.87 | | | 10.84 | 0.39 |
| | | | | | | | | 3.57 | 2.82 | | | 3.20 | 16.60 | 10.73 | 10.31 | | | 10.52 | 2.82 |
| | | | | | | | | 4.02 | 2.61 | | | 3.32 | 30.08 | 10.45 | 10.74 | | | 10.60 | 1.94 |
| | | | | | | | | 4.22 | 5.13 | | | 4.68 | 13.76 | 10.48 | 10.25 | | | 10.37 | 1.57 |
| | | | | | | | | 7.90 | 5.10 | | | 6.50 | 30.46 | 10.21 | 10.16 | | | 10.19 | 0.35 |
| W45/19 | Deck - Top Slab | Fine and warm | Curing compound | 04.10.2008 | 01.11.2008 | 01.11.2008 | 51.8 | 5.77 | 5.59 | | | 5.68 | 2.24 | 10.07 | 10.97 | | | 10.52 | 6.05 |
| | | | | | | | | 5.35 | 6.00 | | | 5.68 | 8.10 | 10.93 | 9.78 | | | 10.36 | 7.85 |
| | | | | | | | | 6.92 | 6.17 | | | 6.55 | 8.10 | 10.78 | 11.60 | | | 11.19 | 5.18 |
| | | | | | | | | 5.08 | 3.62 | | | 4.35 | 23.73 | 10.82 | 10.52 | | | 10.67 | 1.99 |
| | | | | | | | | 4.63 | 5.85 | | | 5.24 | 16.46 | 10.37 | 11.11 | | | 10.74 | 4.87 |
| | | | | | | | | 8.19 | 6.88 | | | 7.54 | 12.29 | 11.64 | 11.43 | | | 11.54 | 1.29 |

| | Water Sorptivity | | | Oxygen Permeability | | | |
|---|---|---|---|---|---|---|---|
| | average (individual) | 5.96 | | | average (individual) | 10.31 | |
| 77 | cov | 31.33 | 25 | 77 | cov | 4.45 | 25 |
| | average (overall) | 6.20 | | | average (overall) | 10.20 | |
| | cov | 34.21 | | | cov | 5.10 | |

**6 ADDITIONAL CORING AT VARIOUS DEPTHS TO CHECK VARIANCE IN DURABILITY RESULTS AT AGE OF 111 DAYS**

| GRADE | ELEMENT | DATE CAST | DATE CORED | REF | No | DEPTH (MIDPOINT (mm) | 28 day Strength (Mpa) | Water Sorptivity | | | | | | Oxygen Permeability | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov |
| W45/19 | Deck Top Slab | 04.10.2008 | 23-01-09 | 1A-D | 1A | 17.5 | | 3.51 | 4.95 | 3.55 | 4.54 | 4.14 | 17.43 | 10.18 | 11.24 | 10.73 | 10.80 | 10.74 | 4.05 |
| | | 04.10.2008 | 23-01-09 | 2A-D | 1B | 25.0 | | 2.85 | 2.80 | 2.22 | 3.14 | 2.75 | 14.00 | 11.35 | 11.18 | 10.68 | 10.82 | 11.01 | 2.82 |
| | | 04.10.2008 | 23-01-09 | 3A-D | 1C | 30.0 | | 2.56 | 3.09 | 2.83 | 2.82 | 2.83 | 7.66 | 11.12 | 10.92 | 10.59 | 10.77 | 10.85 | 2.07 |
| | | 04.10.2008 | 23-01-09 | 4A-D | 1D | 35.0 | | 2.01 | 2.20 | 2.93 | 3.03 | 2.54 | 20.17 | 10.71 | 10.69 | 10.54 | 9.86 | 10.45 | 3.83 |
| | | 04.10.2008 | 23-01-09 | 1A/2-D2 | 1A/2 | 55.0 | | 3.93 | 3.61 | 2.67 | 2.22 | 3.11 | 25.66 | 11.05 | 10.92 | 10.75 | 10.78 | 10.88 | 1.27 |
| | | 04.10.2008 | 23-01-09 | 2A/2-D2 | 1B/2 | 60.0 | | 2.75 | 1.58 | 2.29 | 3.11 | 2.43 | 27.13 | 10.29 | 11.00 | 10.61 | 10.65 | 10.64 | 2.73 |
| | | 04.10.2008 | 23-01-09 | 3A/2-D2 | 1C/2 | 65.0 | | 2.43 | 3.07 | 4.77 | 3.95 | 3.56 | 28.75 | 11.35 | 10.95 | 10.56 | 10.77 | 10.91 | 3.07 |
| | | 04.10.2008 | 23-01-09 | 4A/2-D2 | 1D/2 | 70.0 | | 2.76 | 2.06 | 2.35 | 4.20 | 2.84 | 33.40 | 11.37 | 10.47 | 10.53 | 10.82 | 10.80 | 3.81 |

| | Water Sorptivity | | | Oxygen Permeability | | | |
|---|---|---|---|---|---|---|---|
| 32 | average (individual) | 3.02 | 8 | 32 | average (individual) | 10.78 | 8 |
| | cov | 18.77 | | | cov | 1.62 | |
| | average (overall) | 3.02 | | | average (overall) | 10.78 | |
| | cov | 27.22 | | | cov | 3.12 | |

# ANNEXURE 6 - KING SHAKA AIRPORT BRIDGES DURABILITY TESTING RESULTS

**STRUCTURAL CONCRETE DURABILITY RESULTS**
**KING SHAKA AIRPORT INTERCHANGE**
**OVERPASS BRIDGE, RAMP E BRIDGE**

**1 DESIGN MIXES**

| Grade | Description | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity | | | | | | Oxygen Permeability | | | | | | Chloride Conductivity | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV |
| W40/19 | Substructures | 05.06.2008 | | 11.07.2008 | 47.3 | 4.3 | 4.18 | 4.83 | 3.75 | 4.27 | 10.42 | 10.47 | 10.42 | 10.44 | 10.75 | 10.52 | 1.47 | 0.21 | 0.22 | 0.23 | 0.23 | 0.22 | 4.30 |
| W40/19 | Overpass Bridge Deck | 05.06.2008 | | 11.07.2008 | 47.3 | 3.98 | 3.51 | 3.65 | 3.94 | 3.77 | 6.03 | 10.59 | 10.59 | 10.6 | 10.6 | 10.60 | 0.05 | 0.24 | 0.3 | 0.24 | 0.25 | 0.25 | 11.49 |
| W60/19 | Ramp E Bridge Deck (Cubes) | 02.03.2009 | | 14.04.2009 | 80.6 | 6.58 | 6.58 | 5.48 | 5.87 | 6.13 | 8.91 | 11.12 | 10.76 | 10.47 | 10.78 | 10.78 | 2.47 | 0.17 | 0.13 | 0.16 | 0.14 | 0.15 | 12.17 |

12 12 12

**2 TRIAL PANELS**

| Grade | Cast type | Weather conditions | Curing Regime | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity | | | | | | Oxygen Permeability | | | | | | Chloride Conductivity | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV |
| W30/19 | Substructures - Vertical | Fine and partly cloudy | 72h in forms, curing compound painted on 1h after stripping | 26.09.2008 | | 20.11.2008 | 50.6 | 7.1 | 4.75 | 5.5 | 6.09 | 5.86 | 16.93 | 9.8 | 9.78 | 10.1 | 10.39 | 10.02 | 2.88 | 0.18 | 0.19 | 0.17 | 0.15 | 0.17 | 9.90 |
| | | | | | | | | | | | | | | | | | | | | | | | | | |

| | Water Sorptivity | | | | Oxygen Permeability | | | | Chloride Conductivity | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 4 | average (individual) | 5.86 | 1 | 4 | average (individual) | 10.02 | 1 | 4 | average (individual) | 0.17 | |
| | | cov | #DIV/0! | | | cov | #DIV/0! | | | cov | #DIV/0! | |
| | | average (overall) | 5.86 | | | average (overall) | 10.02 | | | average (overall) | 0.17 | |
| | | cov | 16.93 | | | cov | 2.88 | | | cov | 9.90 | |

**3 TEST CUBES (wet cured)**

| Grade | Element | Weather conditions | Curing Regime | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity | | | | | | Oxygen Permeability | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov |
| W30/19 | Overpass Bridge - Pier 3 Pilecap | Fine and partly cloudy | Wet cured | 26.09.2008 | | 20.11.2008 | 50.6 | 4.85 | 3.75 | 3.71 | 5.56 | 4.47 | 20.14 | 10.62 | 9.80 | 10.14 | 10.33 | 10.22 | 3.36 |
| W30/19 | Ramp E Bridge - North Abutment Pilecap | Overcast, light rain | Wet cured | 28.10.2008 | | 03.12.2008 | 48.8 | 4.33 | 4.39 | 9.02 | | 5.91 | 45.50 | 10.34 | 10.37 | 9.67 | | 10.13 | 3.91 |
| W30/19 | Overpass Bridge - Pier 1 , first lift | Overcast | Wet cured | 21.01.2009 | | 13.03.2009 | 32.9 | 6.22 | 5.73 | 4.78 | 5.05 | 5.45 | 12.00 | 10.31 | 10.29 | 10.22 | 10.19 | 10.25 | 0.55 |
| W30/19 | Ramp C Box Culvert - Panel No. 5 Base | Overcast, hot and humid | Wet cured | 02.02.2009 | | 27.03.2009 | 35.4 | 6.51 | 6.97 | 4.90 | 6.05 | 6.11 | 14.54 | 9.87 | 9.68 | 10.03 | 10.03 | 9.88 | 1.68 |
| W30/19 | Ramp C Box Culvert - Panel No. 2 Base | Sunny and warm | Wet cured | 10.02.2009 | | 27.03.2009 | 45 | 6.54 | 6.96 | 5.37 | 7.77 | 6.66 | 15.02 | 9.52 | 9.28 | 9.79 | 9.76 | 9.54 | 2.49 |
| W30/19 | Ramp C Box Culvert - Panel No. 1 Base | Fine and partly cloudy | Wet cured | 24.02.2009 | | 27.03.2009 | | 6.64 | 6.59 | 5.70 | 5.10 | 6.01 | 12.37 | 9.05* | 9.35 | 9.37 | 9.74 | 9.45 | 2.32 |
| W30/19 | Ramp C Box Culvert - Panel No. 3 Walls and Deck | Sunny and warm | Wet cured | 18.02.2009 | | 27.03.2009 | 43.7 | 5.17 | 8.85* | 6.36 | 5.98 | 5.84 | 10.41 | 9.42 | 8.97 | 7.82* | 9.26 | 9.17 | 2.49 |

| | Water Sorptivity | | | | Oxygen Permeability | | | |
|---|---|---|---|---|---|---|---|---|
| | 26 | average (individual) | 5.78 | 7 | 25 | average (individual) | 9.81 | 7 |
| | | cov | 11.81 | | | cov | 4.33 | |
| | | average (overall) | 5.77 | | | average (overall) | 9.85 | |
| | | cov | 21.11 | | | cov | 4.37 | |

**4 IN-SITU (CORES)**

| Grade | Element | Weather conditions | Curing Regime | Core location | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity | | | | | | Oxygen Permeability | | | | | | Chloride Conductivity | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | CoV |
| W30/19 | Overpass Bridge - Pier 3 Pilecap South | Fine and partly cloudy | Curing compound | 800mm from bottom | 26.09.2008 | | 20.11.2008 | 50.6 | 6.41 | 5.99 | 5.82 | 5.8 | 6.01 | 4.72 | 9.82 | 9.4 | 10.06 | 10.24 | 9.88 | 3.68 | | | | | | |
| W30/19 | Overpass Bridge - Pier 3 Pilecap North | Fine and partly cloudy | Curing compound | 800mm from bottom | 26.09.2008 | | 20.11.2008 | 50.6 | 5.6 | 5.48 | 7.66 | 5.38 | 6.03 | 18.08 | 10 | 8.34 | 9.65 | 8.33 | 9.08 | 9.60 | | | | | | |
| W30/19 | Overpass Bridge - West Abutment Pilecap | overcast and windy | Curing compound | 600mm from bottom | 25.09.2008 | | 20.11.2008 | 50.5 | 6.93 | 7.16 | 5.16 | 7.93 | 6.80 | 17.23 | 10.27 | 9.52 | 10.57 | 10.11 | 10.12 | 4.36 | | | | | | |
| W30/19 | Overpass Bridge - Pier 3 South | Overcast | Curing compound | 500mm from top of P/C | 15.10.2008 | | 17.02.2009 | 50.3 | 6.72 | 7.98 | 4.56 | 5.49 | 6.19 | 24.03 | 10.48 | 9.6 | 10.29 | 10.08 | 10.11 | 3.75 | | | | | | |
| W30/19 | Overpass Bridge - Pier 3 North | Partly cloudy | Curing compound | 500mm from top of P/C | 10.10.2008 | | 17.02.2009 | 42.4 | 7.16 | 8.3 | 8.12 | 8.57 | 8.04 | 7.63 | 9.89 | 9.58 | 9.72 | 9.36 | 9.64 | 2.33 | | | | | | |
| W30/19 | Overpass Bridge - Pier 1 South | Overcast | Curing compound | 1800mm from top of P/C | 21.01.2009 | 01.10.2009 | 01.10.2009 | 32.9 | 9.07 | 10.06 | 8.47 | 8.83 | 9.11 | 7.48 | 8.92 | outlier | 9.31 | 9.65 | 9.29 | 3.93 | 0.38 | 0.43 | 0.33 | 0.33 | 0.37 | 13.60 |
| W30/19 | Overpass Bridge - Pier 1 North | Overcast | Curing compound | 1800mm from top of P/C | 21.01.2009 | 01.10.2009 | 01.10.2009 | 32.9 | 10.88 | 7.62 | 8.67 | 8.26 | 8.86 | 15.98 | 8.81 | 8.93 | 8.85 | 8.67 | 8.82 | 1.23 | 0.45 | 0.36 | 0.42 | 0.41 | 0.41 | 9.30 |

| | Water Sorptivity | | | | Oxygen Permeability | | | |
|---|---|---|---|---|---|---|---|---|
| | | average (individual) | 7.29 | | | average (individual) | 9.56 | |
| | 28 | cov | 18.59 | 7 | 27 | cov | 5.37 | 7 |
| | | average (overall) | 7.29 | | | average (overall) | 9.77 | |
| | | cov | 21.64 | | | cov | 6.47 | |

**5 TEST CUBES (curing compound/ air cured)**

24 hours in moulds. Curing Compound sprayed 1 hour after stripping. Cured at bridge site.

| Grade | Element | Weather conditions | Curing Regime | Date Cast | Date Cored | Date tested | 28 day Strength | Water Sorptivity | | | | | | Oxygen Permeability | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov | Test result 1 | Test result 2 | Test result 3 | Test result 4 | Average | cov |
| W30/19 | Overpass Bridge - Pier 3 Pilecap South | Fine and partly cloudy | Curing compound | 26.09.2008 | | 20.11.2008 | 50.6 | 6.86 | 8.67 | 10.03 | 7.54 | 8.28 | 16.77 | 10.1 | 9.69 | 10.09 | 10.15 | 10.01 | 2.13 |
| W30/19 | Ramp E Bridge - North Abutment Pilecap | Overcast, light rain | Air cured | 28.10.2008 | | 03.12.2008 | 48.8 | 5.2 | 4.71 | 3.37 | | 4.43 | 21.40 | 9.89 | 9.69 | 9.87 | | 9.82 | 1.12 |
| W30/19 | Overpass Bridge - West Abutment | Sunny and warm | Air cured | 09.12.2008 | | 03.02.2009 | 45.95 | 6.43 | 3.78 | 5.02 | 4.79 | 5.01 | 21.82 | 10.05 | 10.19 | 10.19 | 10.38 | 10.20 | 1.33 |
| W30/19 | Overpass Bridge - Pier 1 , first lift | Overcast | Air cured | 21.01.2009 | | 13.03.2009 | 32.9 | 6.00 | 7.09 | 4.85 | 4.32 | 5.57 | 22.19 | 10.18 | 10.16 | 10.05 | 10.04 | 10.11 | 0.72 |
| | | | | | | | | | | | | | | | | | | | |

| | Water Sorptivity | | | | Oxygen Permeability | | | |
|---|---|---|---|---|---|---|---|---|
| | | average (individual) | 5.82 | | | average (individual) | 10.03 | |
| | 15 | cov | 29.27 | 4 | 15 | cov | 1.65 | 4 |
| | | average (overall) | 5.91 | | | average (overall) | 10.05 | |
| | | cov | 31.47 | | | cov | 1.90 | |