THE DURABILITY OF PRECAST CONCRETE ELEMENTS

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A dissertation submitted to the Faculty of Engineering and the Built Environment, University of Cape Town, in partial fulfilment of the requirements for the degree of Master of Science in Engineering.

Cape Town, 2000

DECLARATION

I declare that this dissertation is my own, unaided work. It is being submitted for the degree of Master of Science in Engineering in the University of Cape Town. It has not been submitted before for any degree or examination in any other University.

Signed by candidate

Philip Dean Ronné

11 October 2000.

ABSTRACT

Modern fast track construction methods increasingly favour the use of precast concrete elements. Precast box culverts are structurally significant units, subject to an important combination of bridge loadings. Culverts occasionally in contact with water pose a high durability risk. Despite this, the current specifications allow a reduction in cover to reinforcing steel for precast culverts to only 20 mm from at least 40 mm for cast-in-place culverts.

Currently, little is known about the long-term performance of precast box culverts apart from limited anecdotal evidence. A visual method was developed in order to conduct a rational study of the in-service performance of both precast and cast-in-place box culverts. A large proportion, 38%, of precast culverts exhibited some sign of distress due to environmental exposure. Even in mild carbonating, moderate chloride- and sulphate-bearing environments, the durability performance of precast units was often inadequate. The occurrence of alkali-aggregate reaction was also observed. Softwater attack was however the most frequently observed environmental influence. The rates of carbonation are found to be mild, but increased with a corresponding decrease in concrete quality. The quality of concrete in precast culverts was shown to be superior to that of cast-in-place concrete, but the manufacturing quality of precast culverts in service was found to be inadequate. A large number of reinforcing bars have been determined at covers less than 20 mm and large variations in the covers achieved for individual units were also observed.

The current manufacturing practice was assessed at three precast manufacturers by monitoring steam and corresponding concrete temperatures, characterising concrete mixes and performing checks on the cover to steel. The steam and concrete temperatures were found to exceed the recommended limits, especially during summer. The concrete used in precast box culvert construction had good to excellent durability characteristics. Significant improvements in concrete durability were noted with the use of 30% binder replacement with fly ash. The required cover to steel was not currently being achieved, due to inadequate designs and poor control over the manufacturing process.

A review of the available literature suggests that an altered cement hydration mechanism occurs at elevated temperatures and results in microstructural changes. These changes are known to affect the mechanical properties of concrete, but may also affect the durability of concrete cured at elevated temperatures. Through the use of durability index testing, an increasingly permeable concrete was formed at increasing maximum steam curing temperatures. Measured increases in the water sorptivity, chloride conductivity and bulk porosity suggests that concretes developed larger volumes of macro pores at elevated temperatures. But micro porosity, measured using two electron microscope techniques, decreases for increasing curing temperature. Thus supporting an established suggestion that larger pores are formed at the expense of smaller pores for concretes cured at elevated temperatures.

Recommendations are made to include both steam and concrete temperature monitoring and cover to steel measurements in the SABS 986 specification. Improvements to the current manufacturing process, particularly with regard to control over the achievement of cover to steel and concrete quality are also suggested.

ACKNOWLEDGEMENTS

The National Research Foundation provided financial support for this research. In addition, the Concrete Pipe Association of Southern Africa and BKS (Pty) Ltd sponsored specific studies undertaken in this dissertation.

The Provincial Administration of the Western Cape and South African National Roads Agency granted permission to inspect and sample their structures.

Concrete Units, Fraser Fyfe, and Rocla (Western Cape factories) are thanked for the assistance received in obtaining material and concrete samples, for allowing samples to be taken from the finished products, and for facilitating the monitoring of concrete and steam temperatures. Their open approach and willingness to provide information is greatly appreciated.

Dr JR Mackechnie and Prof. MG Alexander are thanked for their guidance received throughout the dissertation period. Their support of practical research is gratefully acknowledged.

Charles May, departmental assistant at UCT, for his companionship during the fieldtrips and his assistance both on site and in the laboratory.

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Chapter 1:

Introduction

The durability performance of reinforced concrete structures, or lack of it, has been widely publicized in recent years. Up to 5% of some countries' GNP may be consumed by maintenance and repair costs to reinforced concrete structures. Major advancements have been made towards achieving durable concrete through intensive research into the causes and nature of concrete deterioration processes. The historical perception that a direct relationship exists between the strength of concrete and durability is slowly being dispelled (Mehta, 1997). The bulk of durability problems concern the corrosion of the reinforcing steel rather than deterioration of the concrete fabric itself. The modern durability approach is ensuring adequate protection of the steel by the concrete cover layer (Alexander *et al*, 1999).

The causes of concrete deterioration may be classified into two categories, namely physical and chemical causes. Physical deterioration may occur as surface wear or mass loss due to abrasion, erosion and cavitation; and cracking due to volume changes, structural loading and exposure to temperature extremes. Concrete is also susceptible to attack by a wide range chemicals and the mechanisms of deterioration are equally varied. The typical chemical degradation mechanisms are (Ballim, 1999):

(a) *Dissolution of the products of cement hydration:* The products of cement hydration are dissolved and leached out of the concrete causing destruction of the cement hydration products with an accompanying loss of strength. Dissolution of calcium hydroxide from concrete by pure water is typical of this form of deterioration.
(b) *Exchange reactions between acids and components of hardened cement paste*: Concretes made from Portland cement are, by nature, alkaline materials. The products of hydration as well as unhydrated cement react with acids causing corrosion of the concrete matrix. This form of deterioration starts from the surface and continues into the concrete until all the acid or the cement paste has been consumed.
(c) *Conversion of the products of hydration by external agents with associated expansive forces causing deterioration:* External aggressive agents enter the pore structure of the concrete and through crystallisation or reaction with the products of hydration, set up expansive forces which cause cracking and mechanical destruction of the concrete. The effect of sulphates on the concrete matrix is an example of this form of attack.
(d) *Internal processes of deterioration:* Incompatibilities between the constituents of the concrete or effects of the methods of concrete production, produces secondary products internally, which cause the concrete to deteriorate. This deterioration process usually requires only water to be supplied from outside the concrete. Alkali-silica reaction and delayed ettringite formation are typical of this form of deterioration.
(e) Steel *corrosion in reinforced concrete caused by depassivation of the steel:* Corrosive agents and actions (carbonation and ingress of chloride ions) may

> not, in themselves, be aggressive to concrete but cause the destruction of the gamma-ferric oxide layer on embedded reinforcement. The steel becomes depassivated, rendering it susceptible to corrosion if sufficient quantities of oxygen and moisture are present. The reduction in concrete alkalinity by carbonation and the presence of aggressive ions, such as chlorides and sulphates, are likely to cause this form of deterioration.

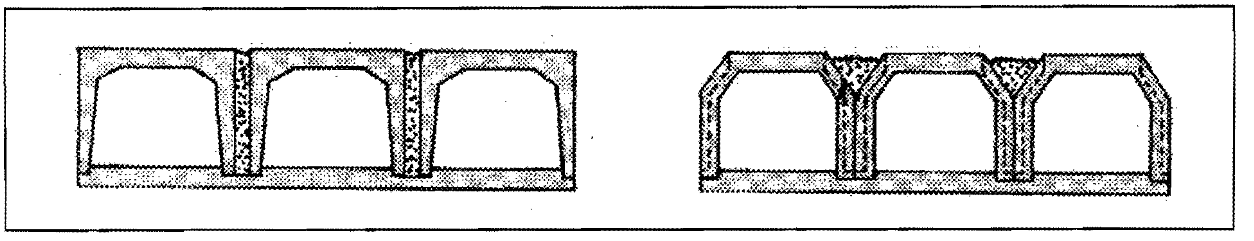
Currently most national building codes and specifications are of the "recipe" type, setting limits on water:cement ratios, cement contents, cover to steel, etc in their attempts to ensure durability. With the exception of cover to steel, these aspects are extremely difficult to measure on site and provide no quantification of the actual concrete quality. However simple index tests have been developed to characterise the quality of concrete at early ages. Correlations have been established relating these durability index tests to the long-term performance, thus facilitating service-life predictions. Engineers' specifications are generally conservative and remain apprehensive in adopting durability testing procedures. But in very aggressive environments, the codes provide no suitable recommendations. The particular requirements, such as increased cover, are left to the discretion of the designer.

Durability index tests can also be used in performance-based specifications. But Mass (1999) suggests that performance specifications are most often successful when well-detailed specifications and drawings, an experienced contractor and supervisory personnel in the specific type of concrete work, and upper level contractor management support for quality is present. Precast concrete manufacturers should achieve such well-controlled conditions. The "controlled" nature of factory type operations allows the mass-production of elements of superior quality. Compliance with performance-based specifications could allow higher prices to be paid for precast concrete elements exhibiting superior quality.

The current trend in the construction industry is fast-track scheduling. Mass suggests that this practice is contrary to the best interest of obtaining quality as there is often insufficient time in the construction schedule to resolve concrete quality issues. The emphasis of the construction becomes productivity, while quality is often overlooked. Precast elements are often used to ensure that a consistent level of quality is achieved, while the rapid construction pace is maintained.

The quality of precast concrete elements can be relied on, if the units are appropriately designed for the intended exposure environment. But the precast industry in South Africa has used the notion of superior quality to their benefit. Product marketing often draws similarities between ordinary wet-cast concrete used for conventional precast applications and high quality concrete used to construct sewer pipes by the roller suspension method. Precast box culverts provide a good example for evaluation of the precast construction industry in South Africa as these units are considered to be structurally significant, but are allowed generous reductions in cover based on the perception of vastly increased levels of quality. Although precast box culverts are widely used in the road construction industry, little is known about the performance of these units in their actual service environments. Discussions held with various industry role players revealed that the potential durability risk of the structures had not been adequately considered, as designers often relied on the provisions of the current specifications. To date, no major study has been undertaken to assess the durability performance of precast portal culverts in South Africa.

The **scope** of this dissertation involves the evaluation of the precast industry through the intensive study of a typical, yet structurally significant, precast concrete element. Precast box culverts are elemental portal-type structures, possessing the structural configuration of a two-pinned frame, and are produced in two forms as shown in figure 1.1.



a b

Figure 1.1: Types of box culverts: (a) Rectangular portal culvert units and (b) ribbed skew haunch culvert units.

The potential durability risk of box culverts will be identified and discussed in relation to other building standards. These units can be found in sufficient numbers in a wide range of exposure conditions, thereby facilitating a systematic investigation into the environment-related performance of these units. The proportion of units exhibiting deterioration will be determined and further classified into the appearance and causes of the deterioration. Since these units are mass-produced under controlled factory conditions, the manufacturing quality of these units can be assessed. Critical issues, particularly concrete quality and cover to steel, will be addressed. In addition, these units are cured using heat treatment methods. Thus the effect of heat treatment on concrete quality will be studied further.

The **objectives** of this dissertation are to quantify the performance of precast box culverts. Firstly, a rational method must be developed to categorise deterioration in various exposure environments in terms of individual units. The method should also allow the inference of the likely cause of the deterioration. The effect of the manufacturing quality on the in-service performance of these units should be determined using this method. A basis for the reduction in cover for precast culverts will be established by comparing the concrete quality of precast and cast-in-place box culverts.

The as-built manufacturing quality of precast culverts will be determined. The concrete quality and cover to steel will be assessed. Simple, yet reliable methods will be designed for this purpose. Further the heat curing procedures used in precast factories will be documented and evaluated in accordance with accepted practice.

A review of the effect of heat treatment on concrete mechanical properties will be presented. The lack of sufficient data relating to the effects of heat curing on concrete durability will be revealed. A laboratory study will be conducted with the purpose of quantifying the effect of heat treatment on the concrete quality.

This dissertation consists of three central chapters: In chapter 2 the in-service performance of box culvert units is presented. The proportion of units showing signs of distress is established. A breakdown of the physical forms of the deterioration and the likely causes of the deterioration is determined. The relationship of exposure environment and manufacturing quality is established in relation to in-service performance. Lastly, a comparison is presented between cast-in-place and precast box culverts.

In chapter 3 the current manufacturing quality of box culverts is established. Tests have been conducted at three precast manufacturers in the Western Cape: Concrete Units, Fraser Fyfe, and Rocla. In order to protect proprietary information, the manufacturers have been assigned arbitrary labels Manufacturer A, B and C (in no particular order). Steam and concrete temperatures are recorded and compared with accepted practice. The concrete quality is assessed. Inferences are made relating to the manufacturing process by determining the concrete quality of the actual unit. The achievement of cover to steel is determined. The control over the reinforcing steel placement is evaluated using a simple statistical approach. Finally changes to the current specifications are suggested where necessary.

In chapter 4 a review of current knowledge relating to accelerated curing is presented. The effect of heat curing on the cement hydration mechanism, concrete microstructural development, concrete mechanical properties and concrete durability is presented. The effect of the maximum steam temperature on concrete durability is assessed using the durability index approach.

Finally, the initial objectives will be considered and conclusions will be drawn and recommendations will be made where necessary. The relevant conclusions and recommendations can be found in chapter 5.

1.1 References

Alexander, M G, Ballim, Y and Mackechnie, J R, 1999, *Concrete durability index testing manual*, University of Cape Town: Department of Civil Engineering, pp 1-33.

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Chapter 2:

Evaluation of the In-service Performance of Precast Concrete Elements

Modern fast track construction methods increasingly favour the use of precast concrete elements. The benefits of using precast units include shortened overall construction times, decreased time to recuperation of project costs (build-operate-transfer schemes), homogeneity of units and a guaranteed performance criterion.

Precast box culverts can be considered structurally significant units for which the durability might be critical. Box culverts are hollow, rectangular units used to convey water from the upstream to the downstream side of a watercourse crossing the path of a road. Since culverts pass under roadways, they are subject to a significantly important combination of traffic, earth and water loadings. In environments like those of the Western Cape, culverts may be subject to repeated wetting and drying cycles. It is well known that wetting and drying cycles exacerbate reinforcing steel corrosion problems and increase the risk of other durability problems. Essentially the durability of these units is important in providing the continued ability to resist the applied loads. Despite this, engineers and owners of these structures appear to have complete faith in the current specification to avoid the potential durability problems.

Currently little is known about the long-term performance of precast box culverts apart from limited anecdotal evidence. In order to evaluate this evidence, a visual method of assessing the performance of box culverts is defined and observations relating to the long-term performance are quantified. Visual and analytical corroborating evidence is presented for selected sites. Furthermore, the manufacturing quality (closely related to the durability performance) is assessed. Finally, a comparison follows between the concrete quality of cast-in-place culverts and precast culverts.

2.1 CURRENT SPECIFICATIONS

Box culverts are included in the national bridge design code, TMH 7 – Parts 1 and 2 (Code of Practice for the Design of Highway Bridges and Culverts in South Africa). The inclusion of culverts in this code underlines the structural importance of culverts in providing resistance to the applicable set of bridge loadings. This implies that there is a reliance on a minimum level of resistance, which in turn needs to be guaranteed. Thus the durability of culverts is of the utmost importance, particularly with precast culverts, which are designed with a minimum cover to reinforcing steel.

2.1.1 The SABS 986-1994 specification

The SABS 986-1994 specification was revised in 1994 and was intended to form a cohesive, binding national standard for the construction of precast box culverts.

Previously, each local provincial administration and transportation department had their own requirements for precast box culverts. The main focus of the document was to standardise the proof load requirements for precast box culverts and to agree on suitable product testing arrangements necessary to guarantee the structural performance of culverts for SABS certification purposes. However, durability-related issues do not appear to have been adequately addressed in the specification.

Historically, precast box culverts have been afforded a significant reduction in the concrete cover to reinforcement compared with cast-in-place concrete. The minimum cover allowed (SABS 986-1994) is the greater of 20 mm or the diameter of the reinforcement. In comparison, if the same unit was to be cast-in-place, the minimum required cover would be 40 mm (or greater, depending on the exposure environment). The reduction in cover is supposedly a direct consequence of superior factory manufacturing quality. Precast construction has the perceived advantage that a greater degree of uniformity and control can be achieved. The reduction in cover is a measure of the increased quality control over the materials used, the mixing procedures, reinforcing steel arrangement and placement, and especially the control over the concrete casting, compacting and curing. In reality the precast concrete must have as much as four times better quality than cast-in-place concrete to facilitate this reduction in cover.

Clause 4.1 of SABS 986-1994 states that box culverts will be "adequately durable if the reinforcement is protected by a concrete cover as specified", except if very aggressive groundwater or conveyed water is encountered. However, the durability of culverts may be compromised if the presumed quality control of the concrete is not achieved. Although some controls are in place, such as a minimum 28 day compressive cube strength of 40 MPa, a minimum cementitious content of 350 kg/m^3, and the guaranteed proof load on the final product, neither of these controls can be used in a manner to achieve durability of the final product. The durability, or resistance to adverse effects from the environment, can be generally described as a function of the cover to the reinforcing steel and the quality of the concrete in the cover region.

Currently there are no recommendations in SABS 986-1994 to ensure a minimum level of durability. There is also no recommendation of the level of durability required for particular environments. It is clear that there is a substantial difference in the requirements for a culvert to be used in a severe marine environment and a culvert destined for a largely dry environment. This decision is currently based on the judgement of the design engineer. The measurement of manufacturing-related issues, specifically the achievement of the required cover, is not clearly defined in the current specification. The influence of accelerated curing procedures on concrete durability is also not adequately defined and guidelines representing good accelerated (or steam) curing practices should be recommended.

The in-service performance of precast box culverts will be affected by the influences as discussed, as well as the environment and period of time to which the culvert has been exposed.

2.1.2 Other national building codes

The reduction in cover for precast concrete is also apparent in a number of other national and international codes. For example, the South African Transport Services Bridge Code (SATS, 1983) recommends a minimum cover of 30 mm to be used for the inside faces of factory made culvert units in severe environments (defined in SABS 0100–Part II). In comparison, the American building code ACI 318-95R recommends a reduction in cover from 50 mm for cast-in-place walls and slabs to 38 mm for precast concrete. The Australian construction code AS 3600-1994 recommends a reduction in cover from 30 mm to 25 mm for precast members with the interior surfaces subject to repeated wetting and drying.

A reduction in cover to only 20 mm is not however recommended in any other code of practice. Since the depth of penetration of most aggressive agents to concrete can be modelled as being approximately proportional to the square root of time, the effect of increasing the cover from 20 mm to 25 mm can increase the time taken to reach the reinforcing steel by more than 50 %. An increase in cover from 20 mm to 30 mm would more than double the time taken for an aggressive agent to reach the reinforcing steel. Thus highlighting the potential risk to the durability of precast box culverts with design covers of 20 mm.

2.2 INVESTIGATIVE METHODS

A visual inspection technique, based on the method currently being used to inspect bridges, was developed to categorise the main environmental influences on culverts in the Western Cape. Since very little is known about the in-service performance of box culverts, an important part of the method consisted of gathering sufficient physical information to accurately identify a particular structure. The use of the road stake value system was adopted to accurately identify the position of the culvert. The manufacturing details, particularly the precast manufacturer, construction date and size were necessary for comparisons to be made. This information facilitated follow-up studies, which were able to relate degradation with exposure time. Culverts with height and span dimensions exceeding one metre were considered for this investigation as inspection of smaller units proved awkward.

The characterisation of the exposure (macro) environment was based on general grouping of typical environments in accordance with recommendations of CEB (1992). However marine exposure environments were further classified in accordance with Mackechnie (1997), thereby accounting for the potential influence of strong local winds on the distribution of airborne chlorides from the sea.

The in-service performance was assessed with the use of a rational checklist of critical factors, which included recording the quality of the concrete surface, the presence of telltale stains and physical evidence of distress. Signs of surface damage that can be easily identified include pop outs (indicates reactive aggregates), honeycombing (indicates poor construction techniques), spalling (indicates corrosion of reinforcing steel). Stains often present telltale signs of rebar corrosion (rust stains) or alkali-aggregate reaction (white gel). Abrasion damage due to sediment transport (including small boulders), as well as the erosive effects of softwater attack, is another visually identifiable signs of deterioration.

Cracks may appear in reinforced concrete structures due to a variety of reasons. Cracks were recorded by their surface crack width. The interpretation of cracks with regard to the potential adverse effect on the durability of the structure remains a contentious issue. However, structurally significant cracks were defined as having a surface crack width of greater than 0.5 mm.

Since manufacturing quality issues are critical to the performance of precast culverts, a cover survey of two selected units from each culvert was performed, and a spot *in situ* carbonation depth measurement was also recorded. Carbonation testing was conducted by removing a small section of the concrete surface and then spraying with a 1% phenolphthalein solution. The average distance from the structure's outer surface to the colour change was recorded.

The cover survey was carried out using an electronic cover meter along longitudinal lines as shown in figure 2.1. A simple statistical analysis of all the covers achieved was later calculated. The cover measurements were analysed by calculating the mean and standard deviation for the entire inner culvert surface. A coefficient of variation was calculated by expressing the standard deviation as a percentage of the mean, thereby indicating the degree of control over the placement of reinforcing steel. Most importantly, from the durability perspective, the number of reinforcing bars with less than 20 mm cover was noted. The analysis was conducted with collated data as the entire inner culvert surface is presumed to have the same durability risk. Occasionally it was impossible to reach the soffit of the structure without the use of a stepladder. In such cases, data was only calculated for the wall sections. The outer surfaces of the culvert are inaccessible as the road pavement layers enclose these surfaces.

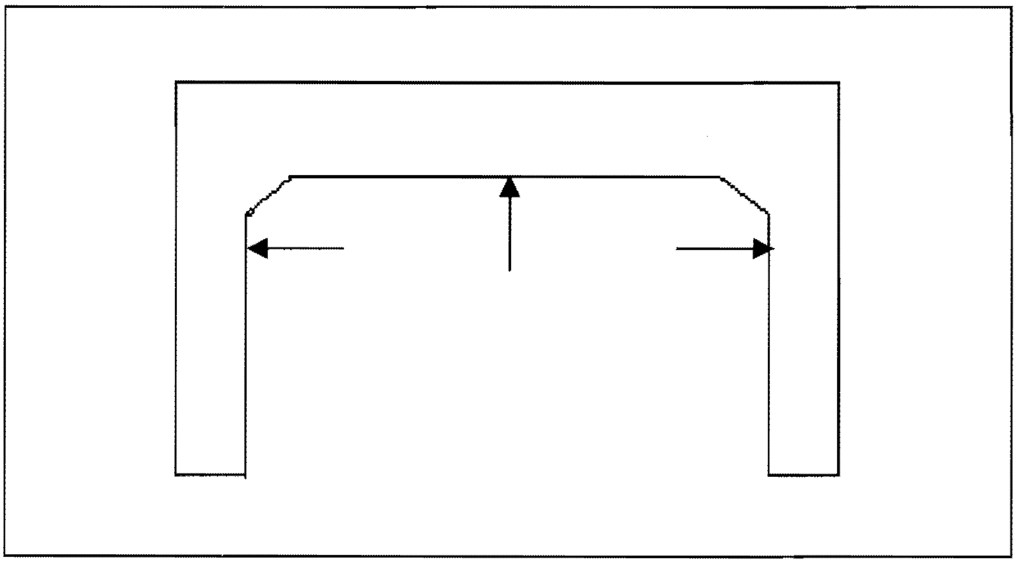


Figure 2.1: Positions along the culvert width, indicated by arrows, at which the cover to steel was checked.

A typical example of the visual inspection form is included as figure 2.2. The form is intended to gather information relating specifically to box culverts in service and makes extensive use of terms relating to the concrete repair and rehabilitation industry, as defined by Rilem (1994). The form is particularly suited as a primary information-gathering tool for the concrete repair and rehabilitation process. Emphasis is placed on three areas of information required: the physical location and construction-related information, classification of the macro environment and signs of deterioration, and information relating to the manufacturing quality. In environments containing highly mineralised soil, the outer surface is likely to be the critical surface to ensure the durability of the structure.

Route:	Stake Value:
Survey Date:	Total units:____ rows of ____ units
Manufacturer:	Type of culvert: Precast / In-situ
Construction Date:	SABS: yes / no
Size:	Class:

Environment

1) Marine environment: Very severe (Severe chloride attack: sea water splash zone, frequent wetting & drying)

Severe (Structure near shore, 50-1000 m in an exposed marine location; spray zone)

Moderate (Structure in a sheltered location within 1000 m; maximum 30 km of coast)

2) Aggressive (without chlorides): Frequent major variations in RH: frequent wetting & drying cycles

3) Moderately aggressive: Environments with infrequent variations in RH: occasional risk of condensation

Signs of Deterioration

Surface damage: Popouts: Y / N. Honeycombing: Y / N.

Spalling: Y / N Number of units affected:

Erosion Abrasion: Softwater attack: Y / N. depth = mm.

Other:

Stains: Rebar corrosion: Y / N Number of units affected:

AAR (white gel): Y/ N Number of units affected:

Other (leaking joints, efflorescence, humic deposits, etc):

Cracks: max crack width = mm

Pattern: map, longitudinal, transverse to rebar, other:

Likely cause: AAR, rebar corrosion, structural loading, other:

Average In-situ Carbonation Depth = mm

Other comments (e.g. possible sulphate bearing groundwater):

Cover Survey

Unit description:

Left:	Middle (optional):	Right :

Analysis:
Mean = mm
Standard Deviation = mm
Coefficient of Variation = %

Unit description:

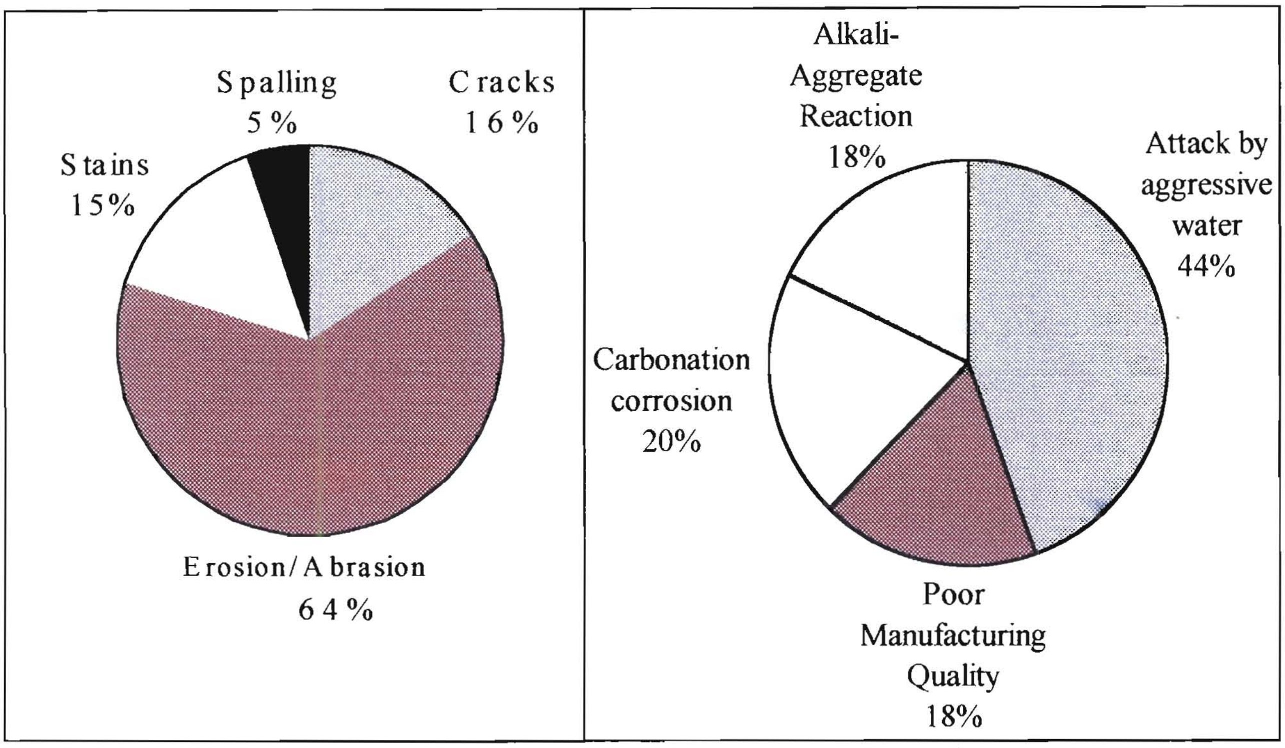
Left:	Middle (optional):	Right:

Analysis:
Mean = mm
Standard Deviation = mm
Coefficient of Variation = %

Figure 2.2: The visual inspection form.

2.3 INTERPRETATION OF DATA

The data collected for each culvert structure using the visual inspection method is presented in the appendices. Each structure was assigned a reference number and the physical information, environmental deterioration and manufacturing quality is presented in terms of that same reference number in appendices A1, A2 and A3. However, the deterioration of culverts has been summarized in terms of the individual units. In total 1313 units were surveyed, of which 498 units (38%) exhibited some sign of deterioration. The physical manifestations of deterioration were classified in terms of erosion/abrasion, staining, and cracks as shown in figure 2.3 a. Likely causes for this deterioration were interpreted as either alkali-aggregate reaction, carbonation corrosion, attack by aggressive waters or poor manufacturing quality as shown graphically in figure 2.3 b.



(a) (b)

Figure 2.3: Analysis of the visual inspection data showing (a) physical manifestation of deterioration and (b) the likely causes of the observed damage.

Surface attack due to presence of 'ion hungry' softwater, prevalent in the Western Cape, formed an overwhelming portion (44%) of this deterioration. In instances where corrosion staining was noted and carbonation depths exceeded the cover achieved, carbonation corrosion was attributed as the likely cause. A significant amount of culverts showed signs of distress due to alkali-aggregate reaction. This is due to the fact that rich concrete mixes are used in an effort to gain early concrete strengths. The maximum alkali content may have been exceeded, for use with reactive greywacke aggregate, due to normal batching variations. However, the presence of an external source of alkalis cannot be excluded. Alkali-aggregate reaction will only develop fully if sufficient moisture is present in the concrete.

A point of concern was that a significant portion of deterioration could be directly linked with poor manufacturing quality. Poor manufacturing quality was characterized by either corroding reinforcing bars with covers less than 10 mm or blowholes at least 20 mm in diameter and 5 mm deep. In both these instances, the effective cover has been reduced, such that the reinforcing steel is not fully protected by the covercrete.

A discussion of particular cases of deterioration backed by visual documentation and supporting analytical evidence, is presented in the following subsections.

2.3.1 Evidence of softwater attack

Softwater attack is a frontal phenomenon in which calcium hydroxide is leached from the concrete matrix. This leads to a layer of softened concrete being formed at the surface, which may be easily removed through scour action by passing water. An example of softwater attack with exposed aggregates to a height of approximately 1 m is shown in figure 2.4. The depth of the softened layer was found to be 3 mm as determined from cores taken from an unscoured section of the culvert. The softened layer had a dark-brown colour typical of attack by softwater containing humic compounds (see figure 2.5). Thus softwater attack is found to be significant if the softened portions are periodically eroded.

Figure 2.4: Softwater attack with constant removal of softened layer at Reference Site 21.

Figure 2.5: Softwater attack with brown humic deposit (Ref. No. 21).

2.3.2 Evidence of sulphate attack

Sulphate ions are known to react with the constituents of hardened cement paste, causing expansion within the concrete and resultant softening and spalling. The ingress of mild concentrations of sulphate ions was found to be sufficient to cause widespread distress as shown in figure 2.6. The large portions of cracking and spalling, together with the presence of localised areas in which the cement paste has been removed and the simultaneous presence of white efflorescent markings, are typical indications of sulphate attack. However, in areas of localised disintegration of the cement paste, the surface concentration was found to be as high as 2,5 % (total sulphate content by mass of cement). Figure 2.7 shows a core sample with signs of erosion of the cement paste, the severe corrosion of a reinforcing bar at low cover, and a large crack extending into the core, most likely as a result of the expansion due to corrosion of the reinforcement. The presence of sulphate ions is known to lead to depassivation, and thereby corrosion, of the reinforcing steel. This process occurs in a similar fashion as with chloride ions, however the threshold limits necessary for corrosion have yet to be accurately defined.

The ingress of sulphate ions into the concrete is represented by the concentration profile shown in figure 2.8. The sulphate levels were determined from sample cores that were sliced in 10 mm depth increments in the laboratory. The slices were crushed using a mortar and pestle. The powdered concrete samples were analysed by gravimetric precipitation of $BaSO_4$. The sulphate content, expressed as SO_3, was reported as a percentage of the concrete sample. For the purpose of this investigation, a correction based on the assumption that the concrete contained 15% cement, was applied in order to express the sulphate content as a percentage by mass of cement. The background sulphate level due to the presence of gypsum in cement has been established at 1,3%. However, the sulphate profile data has not been corrected for this information.

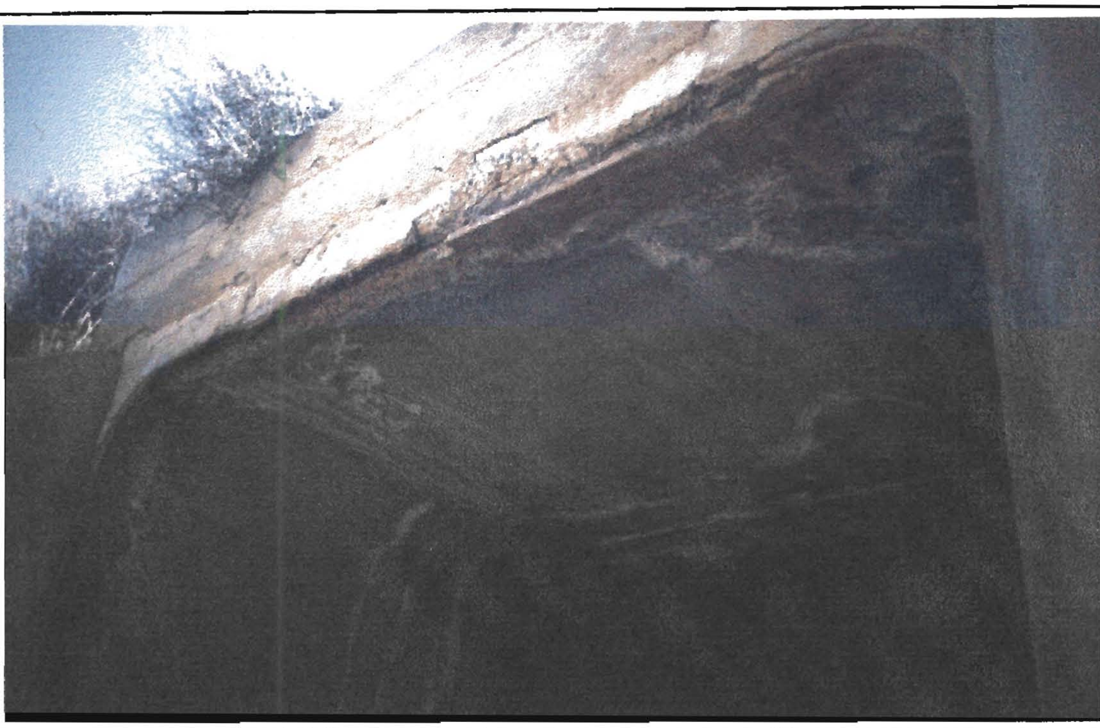


Figure 2.6: A severely distressed culvert exhibiting sulphate attack (Ref. No. 12).

Figure 2.7: Core sample taken through a severely distressed unit exhibiting sulphate attack (Ref. No. 12).

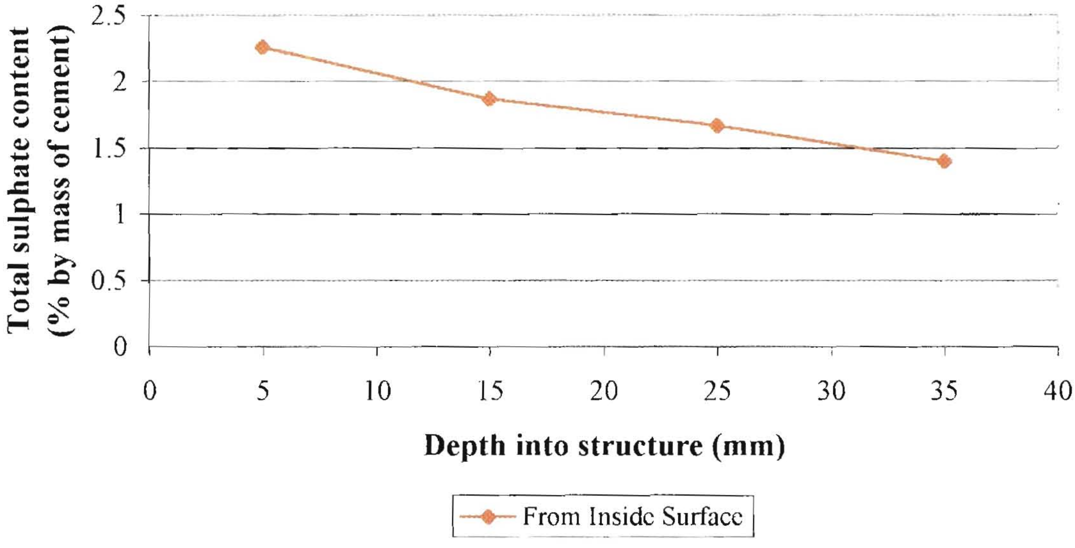


Figure 2.8: Sulphate ion profile determined from an uncracked core (Ref. No. 12).

2.3.3 Effect of chloride ion penetration

Although no culverts were inspected in very severe marine environments during the course of this study, culverts containing a 50% Corex slag blend with 20 mm cover have recently been installed in a housing development at the V&A Waterfront in Cape Town. In moderate marine environments chloride ions were found to readily diffuse through conventional ordinary Portland cement concrete culverts, and reached sufficient concentrations (0,4 % by mass of cement) at the level of the reinforcing steel to activate corrosion. A typical profile taken from a relatively dry and unaffected end unit in a moderate marine environment is shown in figure 2.9. Samples taken from one of many spalled areas near the water level of the same culvert exhibited (inner) surface chloride ion concentrations of 1,6 % by mass of cement. Judging by the large number of units exhibiting spalling and corrosion, sufficient chloride ion concentrations exist at the reinforcing steel to sustain high levels of corrosion (chloride ion concentrations above 1,0 % by mass of cement). It is also clear that the chloride ions are diffusing from the outside surface of the culvert. The material used to backfill the units is the most likely source of the chlorides.

The total chloride ion contents were determined from powdered concrete samples. The chloride ions were subjected to digestion by nitric acid for one hour. The solution was then stabilized using sodium acetate. The chloride content of the concrete sample was determined using automated potentiometric titration techniques and a silver nitrate solution. A Mettler autotitrator equipped with a platinum ring selective ion electrode was used to perform the potentiometric titration. The chloride ion concentrations were then expressed as a percentage of the cementitious content, based on an assumption that the concrete contained 15% cement.

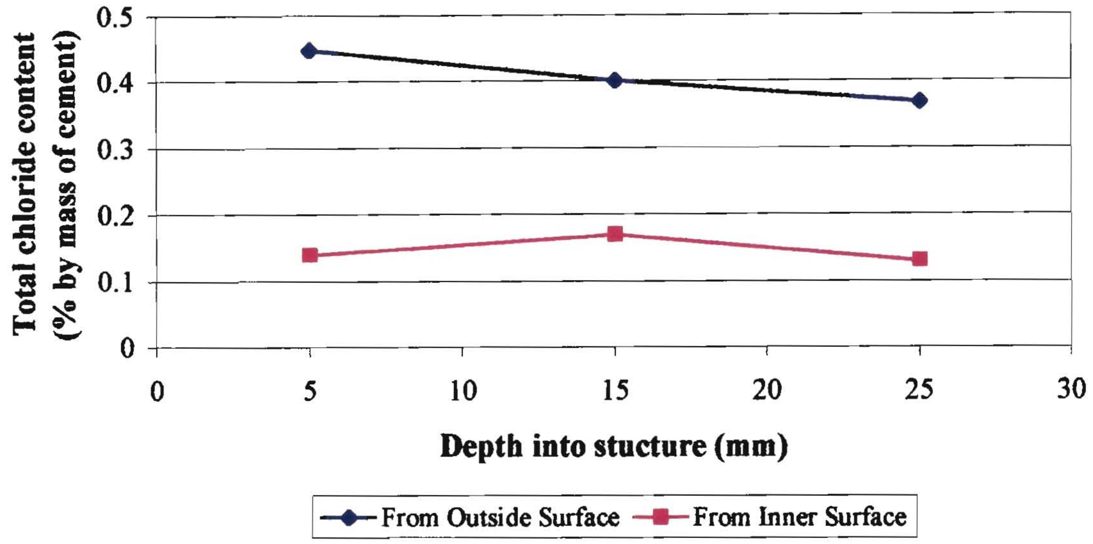


Figure 2.9: Chloride ion profile of a largely unaffected culvert in a moderate marine environment (Ref. No. 52).

2.3.4 Alkali-aggregate reaction

Alkali-aggregate reaction should be completely preventable by either avoiding the use of reactive aggregates, using appropriate replacement levels of cement extenders or limiting the cement content to acceptable limits with the use of reactive aggregates. However limiting the cement content may not be sufficient if an external source of alkalis exist. If sufficient alkalis and a reactive aggregate are present, the severity of the reaction is aggravated by the presence of moisture. The typical map-type cracking pattern, the presence of white gel products and the continually wet appearance of cracks characteristic of alkali-aggregate reaction are shown in figure 2.10.

Exploratory cores taken from the structure (site reference number 22) revealed the presence of typical white reaction products rimming greywacke aggregate. In severe cases, the alkali-aggregate reaction may lead to the formation of extensive areas of delamination. When core samples are removed from concrete affected by alkali-aggregate reaction, the cores often break off along areas of localised weakness, which often exhibit more significant concentrations of reaction products. The fracture surface of a 45 mm diameter core sample, exhibiting increased volume of alkali-silica gel, is shown in figure 2.11.

Figure 2.10: Typical appearance of map cracking patterns and staining due to alkali-aggregate reaction (Ref. No. 22).

Figure 2.11: White alkali-aggregate reaction products rimming reactive Greywacke aggregate on a core sample (Ref. No. 22).

2.3.5 Carbonation testing

Data obtained from in-situ carbonation tests was entered into a typical carbonation model, from which a carbonation coefficient, normalised for exposure time, was calculated. This allowed the evaluation of carbonation rates of typical precast concrete. The rate of carbonation was modelled using the equation:

$$X = K_c * t^{0.4}$$

where X is the depth of carbonation front in mm
K_c is the carbonation coefficient (mm/year$^{0.4}$)
t is the length of exposure time in years.

Through the use of this empirical equation to solve for the carbonation coefficient, an indication of the quality of the concrete material may be attained (Mackechnie, 1999). The computed coefficient can also be used to predict future carbonation depths with time. The mean carbonation coefficient for precast box culverts was found to be 3,0 mm/year$^{0.4}$, and a highest carbonation coefficient value of 7,5 mm/year$^{0.4}$ was determined. Figure 2.12 represents the corresponding carbonation profiles plotted for a period of 30 years. Mackechnie (1999) determined the carbonation coefficient of laboratory grade 40 OPC concrete exposed to a mild (outdoor) carbonating environment to be 3,2 mm/year$^{0.4}$.

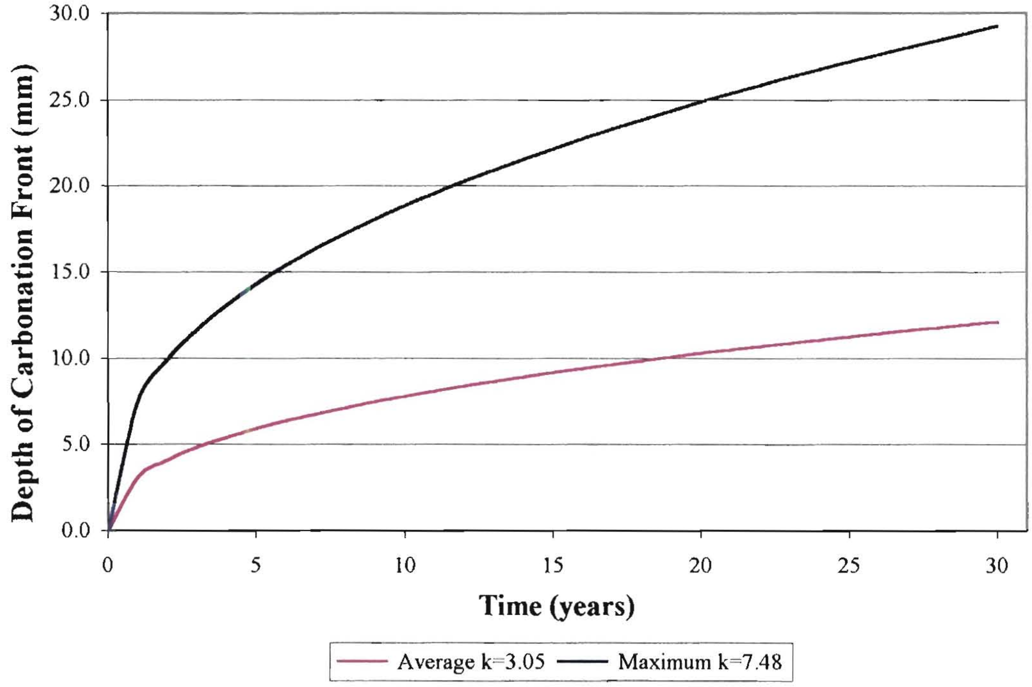


Figure 2.12: Carbonation profiles generated using carbonation coefficients calculated from spot carbonation measurements.

Where high carbonation coefficients were calculated for specific structures, exploratory cores were taken and durability index testing was performed in order to determine whether the concrete was of sub-standard quality or if the environment was particularly aggressive. A number of researchers have suggested correlations between oxygen permeability index (OPI) measurements and carbonation depths (Alexander *et al*, 1999a and Mackechnie, 1999). However such comparisons are specific to the instant at which carbonation depths were measured. Expressing the relationship between carbonation coefficients and OPI measurements is a more general way to quantify the relationship.

A good correlation (R^2=0.908 and correlation coefficient=0.953) was found to exist between the carbonation coefficient and OPI measurements as determined for 5 precast units sampled from a range of typical Western Cape environments. A linear relationship has been established, showing that carbonation coefficient is inversely proportional to the OPI measurements (see figure 2.13). Thus concretes of higher quality, measured using the OPI, are likely to have lower carbonation coefficients. In addition, the use of the carbonation coefficient as a material parameter to compare a large population of structures in similar environments is justified.

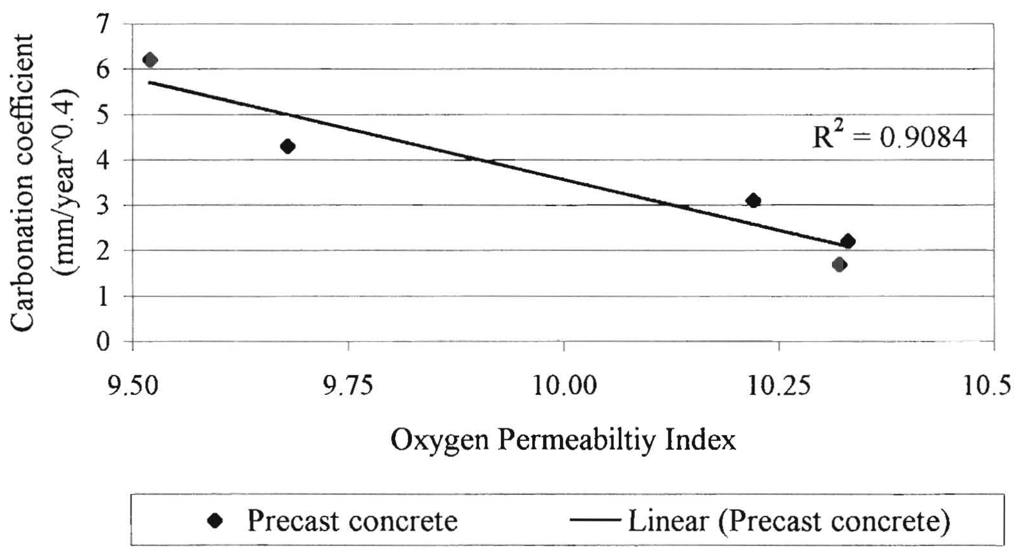


Figure 2.13: Relationship between carbonation coefficients and OPI measurements for precast concrete in the Western Cape.

2.3.6 Achievement of cover over reinforcing steel

Despite the preceding findings, the lack of achievement of the required 20 mm cover is the most frequently observed problem with precast culverts. The durability performance of a structure in a particular environment is a function of the thickness of cover and the quality of the concrete. Thus structures possessing low covers will most likely have a shorter service life. Units constructed by three precast culvert manufacturers, denoted by P, R and T, showed a consistent trend in the lack of achievement of covers:

- P: 9 out of 20 units tested were found to have at least one reinforcing bar less than the required 20 mm cover with a maximum of 9 bars of low cover found for one particular unit (figure 2.14a).
- R: 21 out of 41 units had low covers, with a maximum of 11 bars of low cover found for one particular unit (figure 2.14b).
- T: 20 out of 36 units had low covers, with a maximum of 7 bars of low cover found in one particular unit (figure 2.14c).

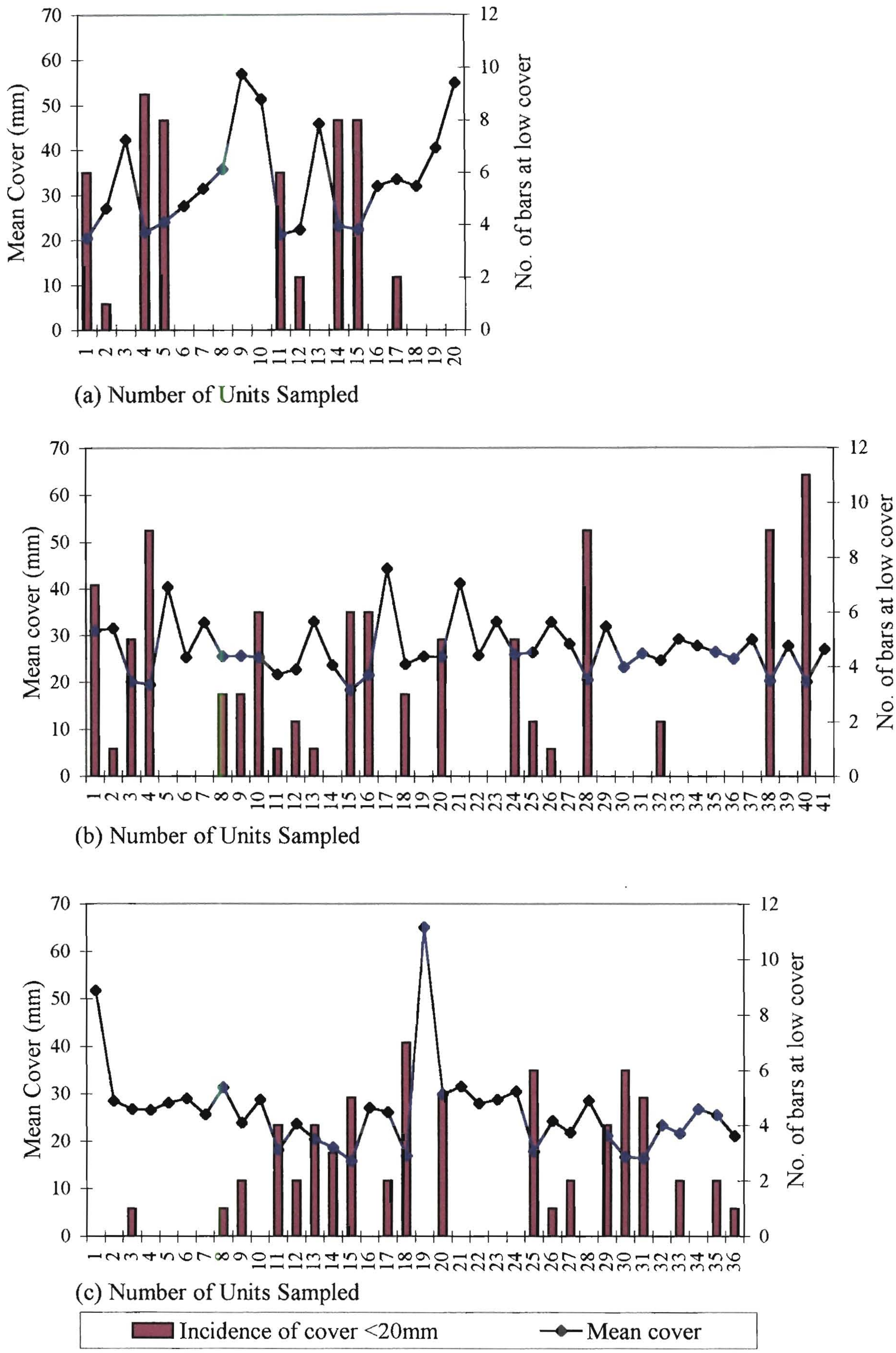


Figure 2.14: Analysis of covers achieved for (a) Manufacturer P, (b) Manufacturer R, and Manufacturer T.

The consistent lack of achieving the minimum required cover points to a lack of control of manufacturing quality, particularly in placing the reinforcement. Figure 2.14 shows that the incidence of reinforcing bars at low covers increases when lower mean covers are determined. However, little information relating to the consistency of cover achieved can be inferred from this information. The coefficient of variation was used as an analytical tool to gauge the degree of control the manufacturer achieved in placing the reinforcing steel. The coefficient of variation was determined by expressing the standard deviation as a percentage of the mean cover determined for each unit.

Figure 2.15 shows the inherent variation of the covers achieved in relation to the mean cover. Excessive variability of covers, as measured by the coefficient of variation, was also found to exist. The mean coefficient of variation for all the units sampled was 23% indicating fair control. From the recommendations made by Sharp (1997), the expected coefficient of variation for the precast industry should be less than 15%. Coefficients of variation up to 70% were found indicating poor control in the placement of reinforcing steel, however coefficients between 5% and 15% indicating good control were also found in some cases. Figure 2.15 highlights that a general increase in the coefficient of variation is observed when reduced mean cover is obtained. This suggests that the degree of control over the placement of steel is compromised at lower covers.

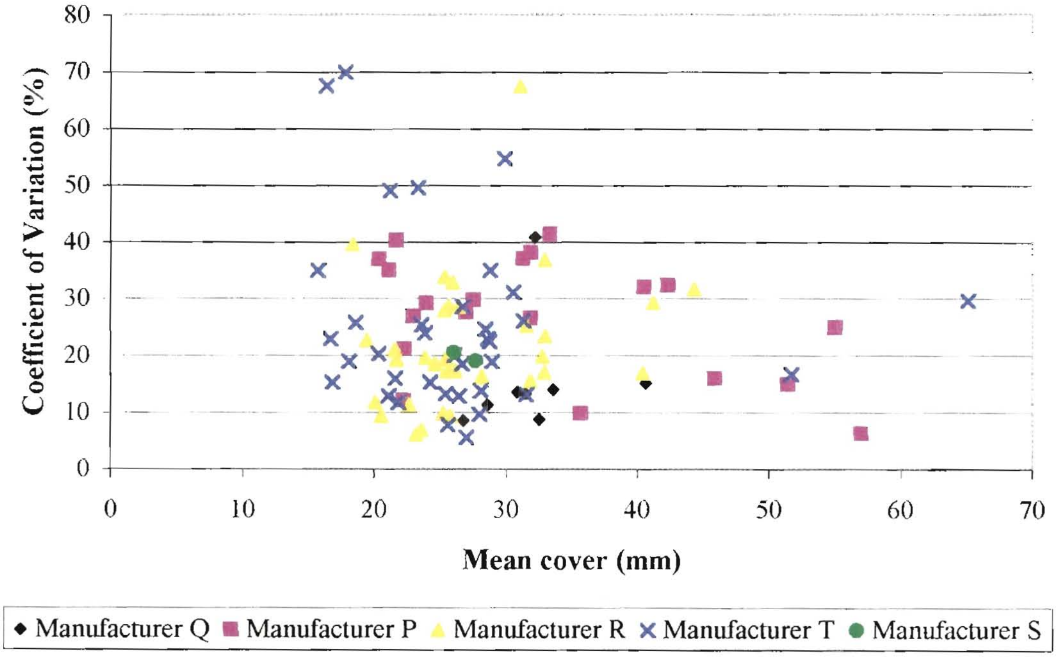


Figure 2.15: Variability of covers achieved.

Further investigations into the manufacturing quality currently being achieved at precast yards need to be conducted. Firstly to assess the achievement of adequate cover, and relate current manufacturing practice to the cover achieved. Once a holistic view of the current manufacturing quality was established, including concrete quality and cover, recommendations could be made in order to avoid the lack of adequate cover and lack of manufacturing quality as determined from this "historical" data.

2.4 COMPARISON OF THE PERFORMANCE OF CAST-IN-PLACE WITH PRECAST CULVERTS

It has not been possible to obtain the construction date of cast-in-place culverts, thus making age or rate of attack comparisons very difficult. The durability index approach was adopted, particularly the oxygen permeability index (OPI), as a means to compare the general microstructure and macrostructure of the relevant concretes used in box culvert construction. The OPI is particularly sensitive to the degree of compaction achieved, the presence of bleed voids and channels, and the interconnectedness of the pore structure. OPI has also been shown to correlate well with carbonation depth measurements (subsection 2.3.6 and Alexander *et al,* 1999a). OPI was determined by measuring the gas permeability of oven-dried concrete. Core samples were obtained from actual structures from which slices of the uncarbonated concrete were prepared and tested in accordance with methods and equipment detailed in Alexander *et al* (1999b). The results are shown graphically in figure 2.16.

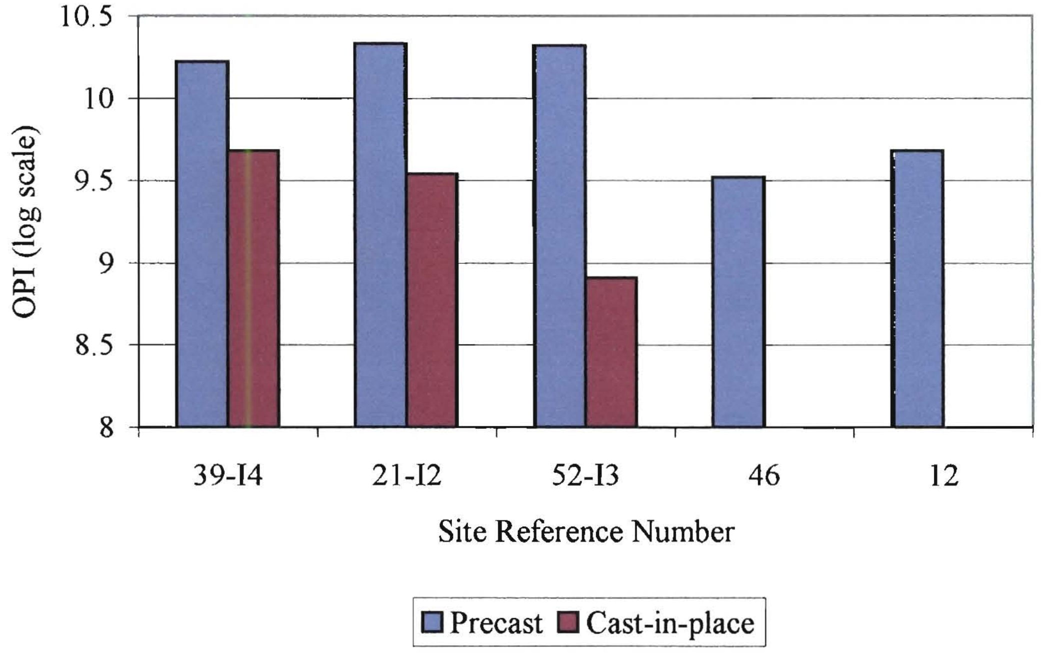


Figure 2.16: Oxygen permeability index testing to determine concrete quality.

Significant differences in overall concrete quality, as measured by the OPI, were found to exist between precast and cast-in-place concretes. Generally, precast concrete achieved significantly better OPI values in excess of 10.20, while cast-in-place concrete achieved OPI values between 8.91 and 9.68 (OPI values measured on a logarithmic scale) indicating that cast-in-place concretes are more permeable than precast concrete. The precast and cast-in-place units compared at each location are situated near each other and are therefore free of environmental bias. Figure 2.16 shows that from the locations tested that precast concrete is of better quality and compaction than cast-in-place concrete. Concretes with OPI greater than 10 represent excellent durability class, while OPI between 9,5 and 10 represent good durability class. Sites Reference numbers 12 and 46 exhibited large carbonation coefficients, as measured using the in-situ carbonation test; a decrease in the quality of those precast units was found as measured by a decrease in OPI values (as discussed in subsection 2.3.6). The lowest OPI values achieved for precast concrete was approximately equal to the highest OPI value achieved for cast-in-place concrete.

The cover achieved for the cast-in-place culverts ranged from 32.0 mm to 61.7 mm and the coefficient of variance ranged between 5.7% and 25.9%. One observation of poor manufacturing quality was made for Site Reference Number I2, where honeycombing was noted. However these observations apply to a limited number of structures representing cast-in-place culverts constructed using modern construction techniques. Cast-in-place culverts constructed using slatted timber formwork was specifically excluded, as steel shuttering and timber sheet material are most often used in modern construction techniques.

Precast concrete has been shown to be of superior quality to cast-in-place concrete. The consistency of covers achieved for cast-in-place is similar, if not better than the cover achieved for precast concrete.

2.5 CONCLUSIONS

It is clear that the current precast culvert requirements are not conservative when compared to similar national and international codes. A reduction in the cover requirements, based on the expected improved quality is acceptable. Yet covers as low as 20 mm seem extreme based on the potential risk of failure of the structure.

A large proportion, 38%, of all precast culvert units surveyed exhibited some degree of deterioration. The occurrence of alkali-aggregate reaction should be completely preventable by applying sound concrete mix design principles. Softwater attack is unlikely to cause serviceability failures, even where turbulent waters are encountered, but allowance should be made for this type of attack. Carbonation-induced corrosion was a frequent occurrence, due to the low covers and exposure to frequent wetting and drying cycles. Precast culverts with 20 mm cover are very susceptible to mild concentrations of aggressive ion species, particularly in chloride and sulphate rich environments.

Poor manufacturing quality was responsible for a significant portion of the deterioration observed. Generally, rates of carbonation were low, except where aggressive microenvironments and lower manufacturing quality were encountered. Measurement of the covers achieved historically, show a significant lack in control of reinforcing steel placement. The current manufacturing quality of precast culverts needs to be investigated in order to fully assess the need for the inclusion of manufacturing-related issues in the SABS 986 specification.

A reduction in cover with precast concrete is justified by showing that the quality of precast concrete is superior to cast-in-place concrete as determined by OPI measurements. However, the concrete quality of concrete currently being achieved in precast factories needs to be determined in order to evaluate the actual reduction of cover that is permissible for culverts.

2.6 SUMMARY

A review of the SABS 986-1994 specification shows that the allowed reduction in cover to reinforcement is extreme in relation to other national and international building codes. The specification also does not adequately address durability and manufacturing quality issues.

A visual inspection method has been developed for box culverts. Inspections of both cast-in-place and precast box culverts have been conducted using this visual method in the Western Cape. Analysis of the data suggests that a large proportion (38%) of precast culverts are exhibiting signs of distress due to environmental exposure. Despite the reduction of cover in precast concrete, due to a perceived increase in quality, a number of manufacturing-related issues are shown to be inadequate. In particular a large number of reinforcing bars have been determined at covers less than 20 mm and a large variation in the covers achieved for individual units have been observed. Even in mild carbonating, moderate chloride- and sulphate-bearing environments, the durability performance of precast units is often lacking. The occurrence of alkali-aggregate reaction was also observed. Softwater attack was however the most frequently observed environmental influence.

Rates of carbonation were found to be mild, but increase with a corresponding decrease in concrete quality. The quality of concrete in precast culverts is shown to be superior to that of cast-in-place concrete. Justification of the reduction in cover needs to be considered through a holistic assessment of the concrete quality and the actual cover being achieved currently at precast factories.

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Chapter 3:

Establishing the Current Manufacturing Quality of Precast Box Culvert Units

The preceding investigation (Chapter 2) into the in-service performance of precast box culverts established that a lack of adequate manufacturing quality was often responsible for a reduction in the performance of precast units. Current manufacturing specifications allow the reduction of cover due to a perceived improvement in the manufacturing quality. This assumption needs to be verified by quantifying the actual manufacturing quality being achieved at precasting yards. The durability performance of a structure in a particular environment is a function of the thickness of cover and the quality of the concrete in the cover region. The manufacturing quality of box culverts is defined in terms of the quality of the actual concrete mix, the concrete quality of the final product (as influenced by the manufacturing process), and the consistency in the achievement of the required cover. The current specification does not provide adequate guidance relating to manufacturing quality issues, particularly with regard to monitoring of steam curing regimes, identifying suitable concrete quality limits, and the achievement of adequate cover. These factors are known to directly affect durability, and in so doing the quality, of the final product.

This chapter presents recorded steam and concrete temperatures in order to assess compliance with generally accepted steam curing practice. The quality of the concrete both at mixing and in the final product (after completion of the full manufacturing process) is quantified through the use of the durability index approach. Checks on the achievement of adequate cover over reinforcing steel were performed to verify compliance with the SABS 986-1994 specification. Further analysis of the data was used to assess the manufacturing quality relating to the consistent achievement of cover.

Currently box culverts are constructed by placing a preformed reinforcing cage into steel moulds. The designed cover is achieved with the use of plastic cover blocks, which are easily clipped onto the reinforcing cage at appropriate positions. A concrete mix complying with a minimum specified 28-day cube strength of 40 MPa is placed into the steel mould using a crane and skip arrangement. Compaction takes place using high-frequency shutter vibrators. The units are then cured, most often through heat treatment.

3.1 STEAM CURING PRACTICE IN THE WESTERN CAPE

Precast manufacturers use heat treatment to accelerate the rate at which concrete hardens. Electrical heating methods are used, but steam remains the most efficient and economical method of supplying the heat required. A large amount of additional energy (approximately five times) is required to produce steam from boiling water. This energy, or latent heat, is released and effectively heats the concrete when the steam condenses.

3.1.1 Recommended steam curing practice

Steam curing procedures have been subject to a number of empirically derived limits, based on restricting severe long-term strength losses. The decisive factors limited by most recommendations are the age of concrete at the commencement of the heat treatment, the rate of the temperature rise, the maximum temperature, the duration of the heat treatment and the rate of cooling (CEB, 1990). Practical curing cycles are chosen as a compromise between the early and late strength requirements but are more importantly governed by the time available (e.g. length of work shifts) (Neville, 1981). The optimum steam curing cycle of precast members is influenced by the cement type, water:cement ratio, size of members, the desired strength at a given time and other factors (ACI 517, 1969). Economic considerations usually determine whether the curing cycle should be matched to a given concrete mix or alternatively whether the mix ought to be chosen to fit a convenient steam curing cycle (Pfeifer, 1982).

The generally recommended curing regime is represented graphically in figure 3.1. A delay of three to five hours before commencing heat treatment or limiting the concrete temperature during the first three hours to below 30°C and below 40°C during the first four hours are recommended (CEB, 1990). The rate of heating should not exceed 25°C/hour, and storage at a maximum temperature limited between 60°C and 70°C (ACI 517 and Neville allow 82°C). The rate of cooling should be limited to 25°C/hour and elements should only be stripped once the concrete temperature is within 20°C of ambient temperature (Addis, 1994). This regime represents a unit that is likely to be heat treated for approximately 18 hours (excluding delay period). Therefore the expected mould turnover period is 24 hours. But significant economic advantages can be afforded to the precast manufacturer if the production rate can be doubled, thereby demanding a shorter curing cycle.

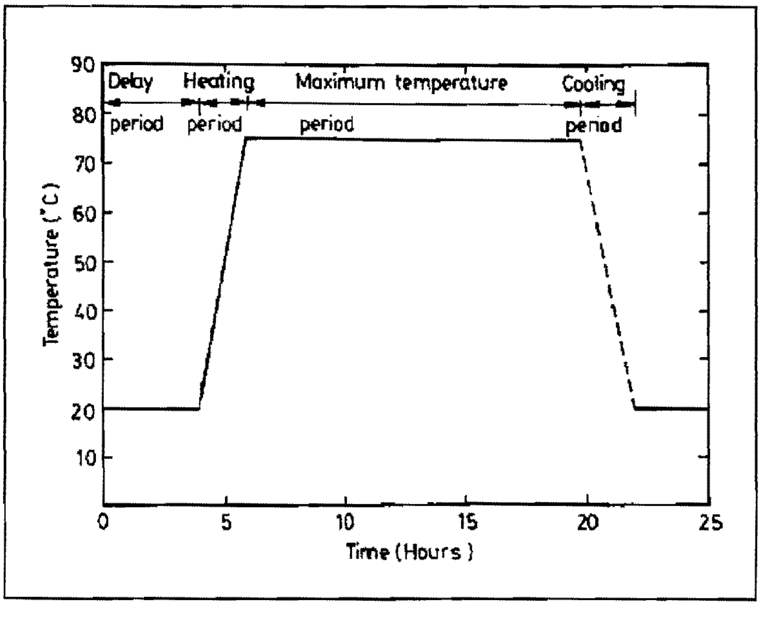

Figure 3.1: Typical recommended steam curing cycle (Addis, 1994).

One method currently used to shorten the steam curing cycle involves the application of heat immediately after applying finishing techniques to the product in order to reduce the setting time of the concrete. Some authors suggest that this practice leads to excessive expansion due to high pressures set up in the pores by the rising temperature while the concrete is still too weak to resist this pressure. However ACI 517 allows the early heating of concrete if the rate of heating is less than 11°C/hour. Alternatively, the concrete mix can be heated in the mixer and heated from that elevated temperature.

3.1.2 The prevention of delayed ettringite formation

Delayed ettringite formation (DEF) is a concrete deterioration mechanism commonly associated with the precast concrete industry. The occurrence of DEF is manifested as extensive cracking, and finally disintegration, resulting from internal disruptions in the concrete. Normal ettringite is a hydrated calcium sulfoaluminate that is conventionally formed during cement hydration under ambient conditions. When temperatures inside the concrete in excess of 65°C are encountered, the calcium sulfoaluminate hydration products become unstable and readily decompose. The higher temperature levels increase the rate of sulphate adsorption by the Portland cement hydrates (Fu *et al*, 1996). Such high concrete temperatures are possible if the concrete is subjected to severe steam curing regimes. Fu *et al* suggest that the slower release of sulphate from an internal sulphate source is the critical condition facilitating delayed ettringite formation. The ettringite develops again at later ages in concretes exposed to water (either intermittently or permanently) and is often deposited at the tips of microcracks in concrete. Expansion and cracking of the affected concrete results. However researchers are still actively debating the fundamental DEF reaction mechanism.

Collepardi's (1999) holistic DEF model suggests that a common set of factors; particularly late release of sulphates, subsequent exposure to water and inherent degree of microcracking must simultaneously exist to cause DEF-related damage. Removal of just one factor would inhibit the DEF mechanism. Hime (1996) suggests the limitation of the sulphate content in the clinker to below 1.5% and further limitations of the SO_3 to Al_2O_3 ratio below 0.55 to restrict the amount of sulphates present for reaction. The use of extenders, especially fly ash and silica fume, show promise in preventing DEF due to the ability of these concretes to limit ionic movement. Skalny *et al* (1997) propose a solution that can easily be instituted and controlled by precast manufacturers. They suggest that a delay period of at least 3 hours should be strictly enforced, the rate of heating and cooling should be limited to below 20°C/hour, and that the maximum concrete temperature must remain below 65°C (and the maximum steam temperature below 60°C). Adhering to these more gentle steam curing regimes should lower the inherent degree of microcracking, as well as prevent the thermal decomposition of the normal ettringite.

The modern concrete research approach sets durability considerations above strength considerations. Therefore the steam curing cycle limits presented in the light of preventing the occurrence of DEF should be used as a standard to control the manufacturing process used by precast box culvert manufacturers. Recent building codes (CEB, 1990) have responded to these recommendations.

3.1.3 Monitoring steam temperatures in precast factories

Steam temperatures and corresponding concrete temperatures were monitored to verify compliance with the latest recommended steam curing practice. Prefabricated insulated thermocouple wires were inserted into the inner steaming chamber to monitor steam temperatures. Additional thermocouple wires were wrapped along the shaft with insulating cloth, leaving the temperature sensitive tip exposed. These were then securely fastened to the reinforcing cage at selected positions and connected to a three-channel chart recorder in order to monitor the concrete temperature. A check on the range and accuracy of all thermocouples was performed; by placing the tips in hot water (95°C) and then in ice water (3°C) and confirming the readings with those obtained using a conventional thermometer. All thermocouples used were accurate to within 2°C of the conventional thermometer readings.

Initial temperature monitoring was conducted in winter, July 1999, to identify the maximum concrete temperature attained, and its position in the unit. All the thermocouple wires were placed along the centre of the width of the unit. Three positions along the span and height were selected: centre of height along the wall, centre of the span (slab), and centre of the haunch thickening at the junction of the slab and wall section. The ambient temperature was 17°C at casting (late afternoon) and dropped to about 6°C five hours after mixing. The temperatures recorded in the concrete are represented graphically in figure 3.2.

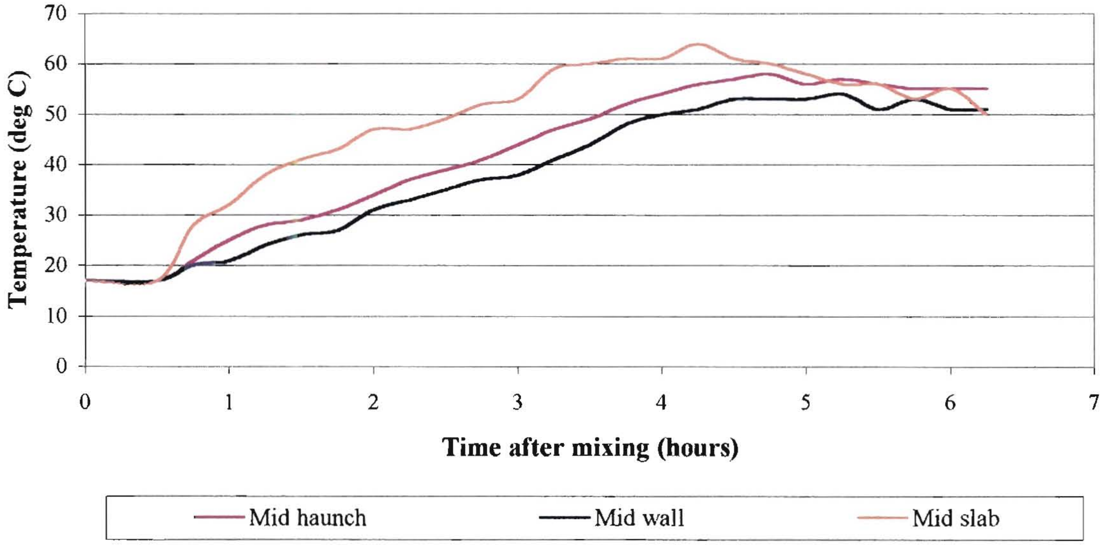

Figure 3.2: Concrete temperature evolution measured on 1 July 1999 (typical cold, wet winter conditions) at Manufacturer B in an 1800x900, 50S rectangular box culvert.

The maximum concrete temperature recorded was 65°C in the centre of the slab after 4,25 hours after mixing. A maximum temperature of 58°C was attained in the mid haunch position after 4,75 hours, while 54°C was attained in the mid wall position after 5,25 hours. Fairly high temperature differentials were developed in the slab during the application of the heat treatment. For example, a temperature differential of 13°C is noted between the mid slab and mid haunch positions after 2 hours. However this remains within the 20°C heating limit suggested by Addis (1994).

A very short delay period of 0,5 hour was used. The rate of heating was initially high at 24°C/hour but the hourly average heating rate was generally less than the 20°C/hour specified values. Although the actual steam temperature was not recorded in this experiment, the steam temperature can be inferred from temperatures in the mid wall position and is assumed to be 53°C. The maximum concrete temperature of 65°C is the same as the upper temperature limit suggested to prevent DEF. The product was heat cured without covering the exposed surface to prevent moisture and heat loss. Temperature measurements during warmer ambient conditions would provide further information relating to control over the maximum steam and concrete temperatures.

Temperature evolution was monitored at two precast manufacturers in summer, April 2000. Temperatures were monitored at the mid haunch position in 1200x1200, 75S (Manufacturer A) and 1200x900, 100S (Manufacturer B) ribbed skew haunch culverts. The mid haunch position was chosen as both manufacturers did not cover the products during heat treatment and the moulds were steel-lined on both surfaces of the haunch. Consequently, the highest concrete temperature was likely to be found in this region. The ambient temperatures were 21°C at Manufacturer A and 26°C at Manufacturer B at the time of casting.

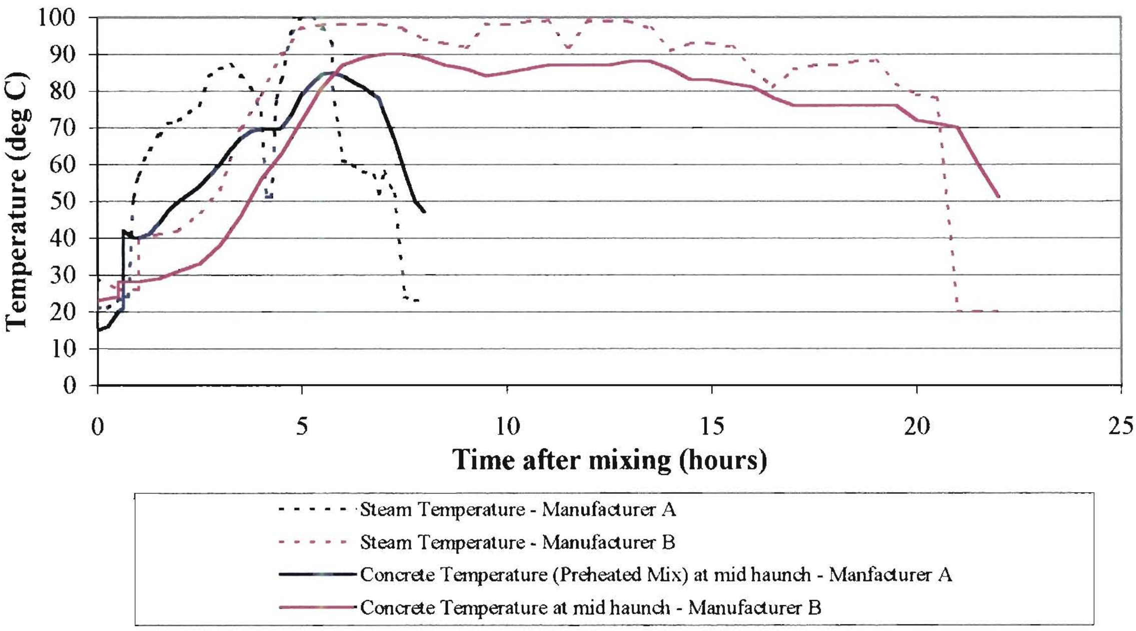

Figure 3.3: Comparison of temperatures monitored at precast manufacturers in April 2000 (typical hot, dry summer conditions).

Manufacturer B managed a delay period of 0.75 hours and a mild initial heating rate. The concrete heating rate increased to 17°C/hour between 3 and 5 hours after mixing. A maximum steam temperature of 99°C and a maximum concrete temperature of 90°C were attained. The product was actively steam cured at these severe temperatures for approximately 20 hours. The steam curing practice employed by Manufacturer B is excessive and dangerous in light of the DEF recommendations.

Manufacturer B then removed the product (70°C) from the mould without allowing the product to cool. Lateral cracking along the haunch on both inside and outside surfaces was observed immediately after removal from the mould. These cracks widened on further cooling to 0.3 mm. Removing the product from the moulds at high temperatures almost certainly caused the cracks. The product was removed with great difficulty from the hot steel inner mould. The hot steel mould had expanded, resulting in confinement of the rigid concrete product. The walls of the product were hence forced to bend slightly outwards on removal from the mould, causing the observed lateral cracks.

Manufacturer B was using a modified concrete mix containing 40% Corex Slag (GGCS). Conventionally wet-cured concrete cube samples, obtained from the same mix used to form the product, exhibited a dark green colour due to the release of sulphate-bearing compounds from the hydrated slag. The walls of the product exhibited a light green colour, indicating partial but adequate hydration levels. However the green colour was not noted halfway up the haunch and in the slab. Exploratory coring of the slab section also did not show any evidence of the green colour. This evidence suggests that the slag did not achieve adequate hydration. The lack of adequate covering to prevent moisture loss, coupled with extremely high concrete temperatures, are responsible for the premature drying of the exposed, upper regions of the product. Thus highlighting problems related to the current steam curing procedures used by Manufacturer B.

Manufacturer A achieved the ideal double rate of production and used a 40°C preheated mix introduced into a mould at ambient conditions. The apparent delay period (approximately 0,5 hours) represents the time taken to fill and compact the unit adjacent to the second, monitored unit in the mould. The steam supply was immediately activated on completion of compaction of the second unit. The maximum rate of heating of the concrete was 12°C/hour between 3 and 4 hours after mixing.

A severe decrease in the steam temperature and a corresponding constant concrete temperature is noted approximately 4 hours after mixing. The steam supply pipes were removed just prior to and for the duration of the workers' lunchtime. The steam is apparently a convenient way to warm food. The concrete temperature remained constant over this period. If cooler ambient conditions existed, a reduction in the concrete temperature may have been observed. The steam supply was replaced at the end of lunchtime and heating of the concrete resumed. A maximum steam temperature of 100°C and a maximum concrete temperature of 85°C were recorded. At 5,5 hours after mixing the steam supply was removed and the product was allowed to cool for 1,5 hours. The product was removed from the mould at 58°C. Lateral cracking along the haunch on the inside surface was again noted. Although the product and mould was allowed to cool, evidence suggests that the minor reduction in temperature was insufficient to prevent cracking. The cracks were however less severe, approximately 0.15 mm wide, and were only noted on the inside face.

Analytical, as well as practical, observations suggest that the steam curing procedures used by box culvert manufacturers in the Western Cape are extreme in nature. Maximum concrete temperatures are far in excess of the recommended durability limits.

3.2 CONCRETE MIX CHARACTERISATION

The durability index approach was used to characterise, or more precisely to quantify the level of concrete quality produced by precast manufacturers. The durability index approach makes use of bulk engineering measurements of the relevant transport mechanisms important to the long-term durability performance of concrete structures. In particular, the oxygen permeability index (OPI) is determined by measuring the gas permeability of an oven-dried concrete. OPI measurements are useful in assessing the overall microstructure and macrostructure of a particular concrete with particular reference to the degree of compaction, the presence of bleed voids and channels, and the degree of interconnectedness of the pore structure (Alexander *et al*, 1999a). The water sorptivity test is used as a measure of the rate at which calcium hydroxide-saturated water is absorbed into concrete under capillary suction. Capillary suction has been found to be dependant on the pore geometry (Alexander *et al*, 1999b), and is useful in assessing the nature and extent of early curing on the cover concrete. The chloride conductivity test is used to assess the resistance of concrete to ingress by chloride ions and is useful as a check on the quality of the mix materials, particularly the binder (Alexander *et al*, 1999a).

The durability index values have been shown to be sensitive to the important material, environmental and constructional factors known to influence concrete durability. Thus durability index testing can be used to assess the following (Alexander *et al*, 1999b):

- Quality control of site concrete
- Concrete mix optimisation for durability
- Performance-based specifications
- Predictions of long-term performance

The durability indexes can be used as a quality control mechanism to evaluate the effect of construction procedures on concrete durability. A degree of flexibility is incorporated through the use of a variety of binder systems and water:binder ratios to optimise a mix for durability. Fully wet cured samples may be used to identify the level of durability and thereby produce a set of acceptance and rejection criteria for performance-based specifications. Established correlations with actual long-term measurements allow the prediction of long-term performance. Alexander *et al* (1999b) have suggested a system of facilitating the classification of durability index values into a representative level (or class) of durability.

Table 3.1: The suggested ranges for durability classification system using index values (Alexander *et al*, 1999b).

Durability Class	**OPI** (log scale)	**Sorptivity** (mm/√hr)	**Chloride Conductivity** (mS/cm)
Excellent	> 10	< 6	< 0,75
Good	9,5 – 10	6 – 10	0,75 – 1,50
Poor	9,0 – 9,5	10 – 15	1,50 – 2,50
Very poor	< 9,0	> 15	> 2,50

Ten 100 mm cube samples were obtained from actual factory mixes during industrial visits. Each cube was cast in two layers and each layer was compacted with 35 blows of a hand-tamping rod. The cubes were transported to the concrete laboratory and fully wet cured for 28 days at 22°C. At 28 days, three cubes were tested for compressive strength (fcu) in accordance with SABS Standard Method 863. The remaining cubes were cored to a diameter of 68 mm and two nominally 25 mm slices cut from either side of the centre of the core. These slices were placed in a 50°C drying oven for at least 7 days. The durability indexes were then determined in accordance with the apparatus and test methods detailed in *Concrete durability index testing manual* (Alexander *et al*, 1999a).

The OPI values represent the mean value of at least 7 samples (maximum 9). The individual sample OPI values were accepted if a minimum correlation coefficient of 0,999 was obtained from a linear regression analysis of the recorded pressure readings (expressed as ln(Po/Pt)) with time. Water sorptivity values represent the mean of at least 5 samples (maximum 6). The individual sample sorptivity values were accepted if a minimum correlation coefficient of 0,99 was obtained from a linear regression analysis of the recorded mass of absorbed water with the square root of time. The chloride conductivity test results represent the average of at least 5 samples (maximum 6). Statistical outliers were removed if the coefficient of variation, determined from variability analysis, was larger than 15%. The results obtained from mix characterisation of actual factory mixes are presented in table 3.2.

Table 3.2: Mix characterisation of actual factory concrete mixes (fully wet cured).

Concrete mix type	**Date Cast**	**Curing Regime**	**OPI** Log scale	**Sorptivity** mm/hr^0.5	**Cl conductivity** mS/cm	**fcu** MPa
Manufacturer A:						
OPC Factory A	14-07-99	28 days@22 deg C	10,12	6,1	1,64	45,0
Rapo Factory A	06-04-00	28 days@22 deg C	10,46	5,6	1,52	49,9
Manufacturer B:						
OPC Factory B	01-07-99	28 days@22 deg C	10,32	5,4	0,70	61,3
40% GGCS Fact B	12-04-00	28 days@22 deg C	9,65	10,0	0,73	64,5

The cube strengths for manufacturer A exceeded the minimum stipulated value of 40 MPa (SABS 986-1994). The OPI values for both mixes from Manufacturer A were in the excellent durability class. The water sorptivity value for the OPC mix was in the good durability class while the value for the Rapo mix (CEM I 42,5R – rapid hardening cement) was in the excellent durability class. The difference in the sorptivity values is however marginal. The chloride conductivity values for both mixes fall in the poor durability class. Thus emphasizing the restricted ability of plain Portland cement concretes to resist the ingress of aggressive ions.

Manufacturer B's OPC mix also surpassed the minimum cube strength limit and both the OPI and water sorptivity values were in the excellent durability class. The chloride conductivity value was also in the excellent durability class. Manufacturer B used a 1% superplasticized concrete mix with a low water:cement ratio as indicated by the increased relative cube strength.

Manufacturer B's 40% GGCS mix exceeded the minimum specified cube strength. The OPI values were in the good durability class. Slag concretes are known to be more permeable and have higher carbonation rates in practice (Mackechnie, 1999). The water sorptivity values fell on the border between the good and poor durability classes. However the chloride conductivity values were in the excellent durability class. The mix was intended for use in culverts subjected to a severe marine environment, and seems adequate for this purpose. Perhaps a larger portion (50%) of the binder should have been replaced with GGCS.

It was evident that conventional factory concrete mixes, subject to good compaction and full wet curing, are capable of providing excellent durability, except when subject to attack by aggressive ions. The levels of durability determined show that the concrete produced at precast manufacturers is comparable with normal structural concrete of good quality. However, the box culvert concrete mixes are in no way similar to the very dense concretes used to produce roller compacted pipes.

3.3 CONCRETE QUALITY OF FINAL PRODUCT

The concrete quality of the final product is affected by the quality of actual concrete mix, as well as the compaction and curing procedures, used in the manufacture of the product. The durability index approach was used to gauge the concrete quality of the final product. The index values were determined as detailed in section 3.2 on concrete samples of 45 mm diameter. The samples cores were taken from the outer surface of the deck of a particular unit. The samples were obtained from a central area of the slab, defined by the centre length portion comprising of half the unit's span and width dimensions. The cores had to exceed a length of 65 mm in order to each provide two samples nominally 25 mm thick for indexing purposes and account for the wastage of the outer 5 mm laitance layer. Precast manufacturers often despatch the units to site when the stock volume reaches the required transport loads. Due to the limited selection of units at factories, units at the approximate age of 28 days were sampled. The durability index values below represent the average of at least 4 samples (maximum 5 samples for OPI and conductivity tests).

Table 3.3: Characterisation of concrete samples cored from actual units.

Concrete mix type and sampling age	**Date Cast**	**OPI** Log scale	**Sorptivity** mm/hr^0.5	**Cl conductivity** mS/cm
Manufacturer A:				
Rapo-Unit 1: 28 days	08-03-00	10,03	6,2	1,57
Rapo-Unit 2: 21 days	16-03-00	9,78	4,1	2,07
Manufacturer B:				
OPC-Unit 1: 27 days	19-08-99	9,53	4,5	1,74
OPC-Unit 2: 36 days	08-03-00	9,77	7,7	1,84
OPC-Unit 3: 26 days	18-03-00	10,09	5,0	1,07
OPC-Unit 4: 24 days	20.03-00	9,87	6,6	1,74
OPC-Unit 5: 32 days	12-03-00	10,27	5,7	1,11

<u>Note:</u> Steam curing regimes not recorded for the units sampled in table 3.3.

Manufacturer A achieved OPI results in both the excellent and good durability classes. Sorptivity values in both the excellent and good durability classes and chloride conductivity values in the poor durability class were also determined. These levels of durability seem to be in line with values determined in the mix characterisation (section 3.2). However a small, yet consistent deterioration in the index values was noted.

Manufacturer B achieved a spread of OPI results across both the good and excellent durability classes. Sorptivity values in both the excellent and good durability classes were observed. Chloride conductivity values in the good and the poor durability classes were achieved. Similar trends were established for the mix characterisation process.

A general deterioration of the durability index values is observed, suggesting that the manufacturing process, particularly the curing and compacting procedures, adversely affect the final quality of the product. However, the effect of the manufacturing process cannot be fully quantified since the quality of the particular concrete mix is not known.

3.4 ACHIEVEMENT OF COVER TO STEEL

The preceding investigation into in-service performance of precast box culverts (subsection 2.3.7) showed a consistent non-achievement of the required cover to reinforcement. Many authors have noted that the failure to achieve adequate cover is the single most significant factor in the premature deterioration of reinforced and prestressed concrete structures. The non-achievement of cover is a critical durability issue for precast box culverts, especially considering that the minimum cover allowed is reduced to 20 mm (SABS 986-1994). This reduction is allowed due to the perceived increase in manufacturing quality, especially in the bending and placement of reinforcing steel.

Sharp (1997) provides a statistical basis for evaluating the compliance of covers to specified values. Sharp stressed that the distribution of cover in normal practice is Gaussian, and that tolerances and acceptance criteria follow similarly to those for mix design and quality control. The standard of control may be evaluated in terms of a coefficient of variation. The coefficient of variation represents the ratio of the standard deviation to the statistical mean and is calculated by expressing the standard deviation as a percentage of the mean.

Neville (1999) and Sharp (1997) both provide comment on the confusion relating to use of the terms 'nominal' and 'minimum' cover. For example, SABS 986-1994 requires a minimum cover of 20 mm. This should be the absolute lowest cover measured in the precast box culvert. Thus precast manufacturers should design the reinforcement with covers larger than 20 mm to account for anticipated statistical variations. The larger cover that the manufacturer aims to achieve is defined as the nominal cover. The incremental increase in cover is related the standard of control, and hence the coefficient of variation, achieved by the precast manufacturer. Characteristic (typically 5%) minimum cover values should not be used unless the population to be tested can be accurately defined.

The cover to the reinforcing steel was determined using an electronic cover meter along the width of 10 randomly selected units at two manufacturers. The lines along which the covers were checked are shown below in figure 3.4. The smallest cover obtained for each individual bar was recorded. Following this, bars less than the required 20 mm cover were noted. The mean of all bars was obtained for bars on the inner surface of the unit, along with the standard deviation. Following this, a coefficient of variation (c.v.) was calculated. The c.v. was used to provide an indication of the covers achieved, with a low c.v. indicating good control and vice versa for a high c.v. The same procedure was repeated for covers obtained along the outside surface of the unit. The cover achieved for the outside surface is of particular interest, as these surfaces are most likely to be subjected to moist ground conditions for extended periods of time. Observations made in chapter 2, indicate that the inner surface will only convey water for short periods during, and immediately following, periods of active precipitation. The full analysis of the covers achieved is shown in table 3.4.

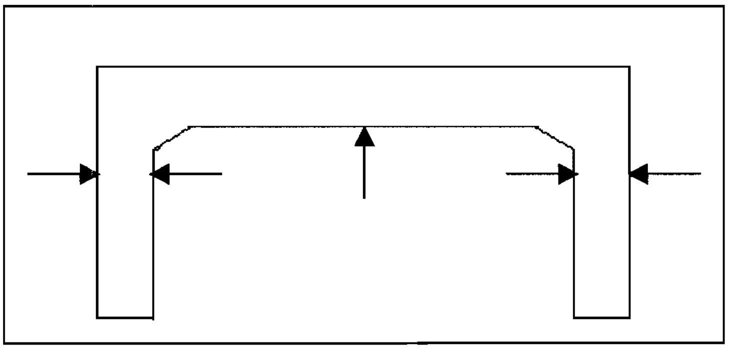

Figure 3.4: Lines, indicated by arrows in cross-section, along which cover to steel was checked.

Table 3.4: Analysis of cover achieved in units complying with SABS 986-1994.

Unit Size & Class	Construction Date	Incidence of cover <20mm	Cover: Inside Mean mm	Cover: Inside Standard deviation	Cover: Inside c.v. %	Cover: Outside Mean mm	Cover: Outside Standard deviation	Cover: Outside c.v. %
Manufacturer A:								
1800x1200, 75S	06-04-98	4	22	5	20	46	14	30
1800x1200, 75S	25-08-98	5	20	6	29	37	10	28
1800x1200, 75S	21-08-98	3	23	8	36	53	14	26
1800x1200, 75S	21-08-98	2	23	4	19	47	6	13
1500x1500, 75S	06-05-99	-	29	13	43	25	4	16
1500x1500, 75S	06-05-99	-	24	5	22	26	4	15
1500x1500, 75S	04-05-99	11	42	30	71	17	3	16
1200x1200, 100S	20-03-98	5	32	23	70	23	2	8
1200x1200, 100S	07-03-98	5	35	28	79	28	3	12
1200x1200, 100S	10-03-98	2	35	18	53	27	3	11
Manufacturer B:								
2100x1800, 75S	14-01-00	10	22	2	10	32	9	29
2100x1800, 75S	21-01-00	24	20	2	11	37	5	15
2100x1800, 75S	17-01-00	15	20	3	14	31	5	17
2100x1800, 75S	13-01-00	15	22	4	18	31	13	40
2400x1200, 50S	25-08-99	28	20	6	28	24	3	11
2400x1200, 50S	23-08-99	8	26	5	18	22	3	12
2400x1200, 50S	30-03-99	12	23	6	26	30	4	15
1200x900, 100S	04-03-00	8	22	3	13	29	10	34
1200x900, 100S	06-03-00	10	21	4	17	31	7	21
1200x900, 100S	02-03-00	6	22	2	10	36	7	20

The analysis confirms that the required cover is not being satisfactorily achieved. Specifically the majority of units tested at both manufacturers had significant numbers of bars at covers less than 20 mm. Manufacturer A had average low covers of 14 mm and Manufacturer B had average low covers of 18 mm. The reasons for the lack of achievement of cover, based on observations made during industrial visits to box culvert manufacturers, are:

- **Lack of adequate design**. Especially designs using minimum, instead of nominal, cover specifications.
- **Actual cover achieved not as per design requirements**. This may occur due to a loose reinforcing bar (isolated incident), the inadequate placement of cover blocks, standing on the reinforcement cage by workers, the release of concrete from skips in large masses and from significant heights, and rotation of the reinforcement cage during compaction (related to inadequate placement of cover blocks).

8 out of 10 units tested from Manufacturer A had at least one bar with less than 20 mm cover, with the maximum of 11 bars in one particular unit being less than the minimum cover. The mean cover determined for the inside surface of the units ranged from 20 mm to 42 mm and the c.v. calculated for the inside surface ranged between 19% and 79%. Detailed analysis of the data showed that the c.v. of the slab surface varied between 7% and 25% while the c.v. for the inside surface of walls varied from 25% to 63%. This implies that Manufacturer A achieved more control over reinforcing steel placement in the slab than in the walls. The mean cover achieved for the individual portions differed slightly from the average values calculated for the entire inner surface (difference ranged between 2% and 13%).

The mean cover achieved for the outside walls ranged between 17 mm and 53 mm and the c.v. varied from 8% to 30%. This confirms that Manufacturer A did not have adequate control over the placement of reinforcing steel in the walls.

The mean c.v. determined for the ten units tested at Manufacturer A were:

Entire inside surface = 44%
Inside surface of walls = 46%
Slab only = 11%
Outside surface of walls = 17%

The entire inner surface of a box culvert has the same durability risk. Since the protection of the reinforcement from corrosion is a function of the concrete quality and the thickness of the cover, large variations of the cover relate to variable durability performance. The method of calculating statistics on covers for the entire inner surface is sufficiently sensitive to these variations. Since the slab is heavily reinforced and the inner walls are reinforced to lesser degree, the method proved sufficiently sensitive to large variations experienced by small populations. Further, the variation of cover achieved for the inner wall sections exceeded the range recommended by Sharp (see table 3.5). Thus indicating that an exceptionally poor standard of control was achieved for the inner surfaces of walls. An excellent degree of control was achieved for the slab and a fair degree of control was achieved for the outside surfaces of walls.

All the units tested from Manufacturer B had at least one bar less than 20 mm cover, with the maximum of 28 bars in one particular unit being less than the specified minimum cover. The mean cover determined for the inside surface of the units ranged from 20 mm to 26 mm and the c.v. calculated for the inside surface ranged between 10% and 28%. Detailed analysis of the data showed that the c.v. of the slab surface varied between 4% and 15% and the c.v. for the inside surfaces of walls varied from 9% to 35%. Despite the large number of bars with cover less than 20 mm, the statistics show that Manufacturer B did have control over the steel bending and placing operations. Once again, the mean cover achieved for the individual portions differed marginally from the average values calculated for the entire inner surface (difference ranged between 1% and 8%).

The mean cover achieved for the outside surfaces of walls at Manufacturer B ranged between 22 mm and 37 mm and the c.v. varied from 11% to 40%.

The mean c.v. determined for the ten units tested at Manufacturer B were:

Entire inside surface = 17%
Inside surface of walls = 17%
Slab only = 9%
Outside surface of walls = 21%

The method of calculating statistics on covers for the entire inner surface is once again shown to be sensitive to variations in a small population of the total data. A fair standard of control was achieved for the entire inner surface. Further, the variation of cover achieved for the inner surfaces of walls was indicative of a fair degree of control. Near-laboratory precision was achieved for the slab and a fair-minus degree of control was achieved for the outside walls. It is evident, from the statistical analysis, that Manufacturer B attained sufficient control relating to the bending and placement of reinforcing steel. However the large number of bars less than 20 mm indicates that the reinforcement design incorrectly used 20 mm as the nominal cover.

Both manufacturers have shown a consistent lack of achievement of cover, with large numbers of bars observed at covers less than 20 mm. A suggested table of nominal covers statistically required to achieve a range of minimum covers for varying degrees of control is presented in table 3.5. Individual precast manufacturers should determine the coefficient of variation that is being achieved for their individual processes and apply the relevant adjustment to the reinforcement design in order to achieve the stipulated minimum cover.

Table 3.5: Nominal design covers used to achieve stipulated minimum covers with varying standards of control. (Calculations based on method presented by Sharp (1997)).

Standard of Control	**Coefficient of Variation** (%)	**Nominal Cover Required (mm)** Minimum Stipulated Cover of:		
		20 mm	30 mm	40 mm
Near-laboratory Precision	10	28	42	56
Excellent	12	29	44	59
Good	15	31	47	63
Fair	20	35	53	70
Fair Minus	25	40	60	80
Poor	30	47	70	93

It is the author's opinion that reinforcement designed to the above nominal covers will significantly reduce the number of reinforcing bars having covers less than the stipulated minimum value, especially if good construction practice and sufficient number of correctly placed cover blocks are used.

3.5 CASE STUDY: Use of Fly Ash to Achieve Quality in Precast Box Culvert Construction

An investigation into the use of fly ash to achieve durability in precast box culvert construction was conducted between August and December 1999 on behalf of a consulting engineering firm. The presence of chloride-containing groundwater, the strategic importance of the site, and an increase in the design life of the structures to forty years, all prompted the need to ensure the durability of the culvert structures.

A concrete mix containing 30% fly ash, a maximum water:binder ratio of 0.5 and a minimum cover to steel of 40 mm were specified. The concrete mix also had to provide adequate durability, as determined by exceeding the prescribed set of durability indexes shown in table 3.6. The prescribed index values apply to concrete mixes cured for 28 days in water at 22°C and hence represent an indication of the concrete microstructure development and an additional check on the chemical mix constituents.

Table 3.6: The prescribed set of durability indexes.

Oxygen permeability index	≥ 9.5
Sorptivity	< 10 mm/√hr
Chloride Conductivity	$< 2,0$ mS/cm

After the quality of the concrete mix was verified using fully cured samples, the quality achieved in the actual product was checked. The actual units sampled were cast with the exact concrete mix sampled for characterisation purposes. This allowed the direct assessment of individual manufacturers' construction process. Finally, it was necessary to verify the achievement of the required cover to steel and to obtain an indication of the consistency of the actual covers achieved.

3.5.1 Concrete mix characterisation

Ten 100 mm cube samples for each curing regime were obtained from actual factory mixes, cast in two layers, each compacted with 35 blows of a hand-tamping rod. In order to provide the earliest possible results so that corrective actions could be taken if required, it was decided to make use of an accelerated curing regime. The chosen accelerated curing regime involved a period of 14 days in 35°C water, which is approximately equivalent to the regular curing regime of 28 days in 22°C water. It was also necessary to fully quantify the long-term benefits corresponding to the use of fly ash. Cubes fully cured for 96 days in 22°C water were used for this purpose.

After completion of the required curing regime, three cubes were tested for compressive strength, while the remaining cubes were cored to a diameter of 68 mm. Two nominally 25 mm slices were removed from either side of the centre of the core. The durability indexes were performed as detailed in section 3.2. The results obtained for the mix characterisation process are presented in table 3.7.

Table 3.7: Mix characterisation results.

Concrete mix	Date Cast	Curing Regime	OPI Log scale	Sorptivity mm/hr^0.5	Cl conductivity mS/cm	fcu MPa
Manufacturer A:						
Fly Ash 1	05-11-99	14 days@35 deg C	10,54	5,8	1,46	45,8
Fly Ash 1	05-11-99	28 days@22 deg C	10,44	5,7	1,07	47,1
Fly Ash 2	25-11-99	14 days@35 deg C	10,54	4,8	0,90	47,6
Manufacturer B:						
Fly Ash 1	01-09-99	14 days@35 deg C	10,69	3,8	0,23	67,0
Fly Ash 1	01-09-99	28 days@22 deg C	10,58	4,1	0,67	64,8
Fly Ash 1	01-09-99	96 days@22 deg C	10,83	6,2	0,26	76,0
Manufacturer C:						
Fly Ash 1	27-08-99	14 days@35 deg C	10,20	5,4	0,43	52,5
Fly Ash 1	27-08-99	28 days@22 deg C	10,41	5,5	0,73	66,1
Fly Ash 1	27-08-99	96 days@22 deg C	10,94	5,6	0,51	75,8
Fly Ash 2	01-10-99	14 days@35 deg C	10,10	5,0	1,01	53,8
Fly Ash 3	29-10-99	14 days@35 deg C	10,23	6,2	0,77	51,4

All the index values were better than the prescribed durability index values as stated in table 3.6. The compressive strengths of all the mixes were found to be greater than the stipulated lower limit of 40 MPa (SABS 986-1994). Large differences in the compressive strengths were noted. This was attributed to the use of largely different water: binder ratios as follows:

Manufacturer A used a water:binder ratio of 0,47
Manufacturer B used a water:binder ratio of 0,40
and Manufacturer C used a water:binder ratio of 0,40.

The OPI values, for all manufacturers, were in the excellent durability class, indicating that all the mixes could be easily compacted. A significant improvement over the values determined for conventional mixes (for Manufacturers A and B) in section 3.2 is noted. This improvement can be attributed to the refinements of the microstructure of concretes containing fly ash.

The sorptivity results, with the exception of Manufacturer C's fly ash mix 3, were also in the excellent durability class. The sorptivity value for Manufacturer C's fly ash mix 3 was in the good durability class. It is interesting to note that for mix 1 of both Manufacturer B and C, the sorptivity values increased with longer duration of curing when in fact they should have decreased (although the differences were small). This anomaly prompted a check on the bulk water-filled porosity of the concrete. The porosity was found to decrease significantly with curing time, from 8,4% at 28 days to 4,0% at 96 days for Manufacturer B and from 8,6% at 28 days to 5,1% at 96 days for Manufacturer C. This suggests that most small voids were effectively "blocked" leaving larger voids. This is possible through a process known as pore refinement, in which pozzolanic reaction products (from the reacted fly ash) are deposited along the walls of the pores, increasing the discontinuity of the capillary pores. The remaining larger pores are easily and rapidly filled with water during the sorptivity test, giving slightly distorted results. In general, the sorptivity values are largely similar to those obtained in section 3.2.

Generally, the chloride conductivity values varied with differences in the individual manufacturers mix parameters. Manufacturer B achieved the best results with values falling well within the excellent durability class. Manufacturer C produced conductivity values either side of the good-excellent durability border, with some of the results lying well within the excellent durability class. Interestingly, the accelerated curing regime was consistently able to produce chloride conductivity values equal to the same concrete cured for 96 days at 22°C. This is probably due to the influence of maturity on both the accelerated curing and 96 day curing regimes. The chloride conductivity values are significantly lower than the values obtained for CEM I (OPC) concrete mixes in section 3.2. Thus highlighting the improved chemical resistance afforded to concrete with the use of cement extenders, particularly fly ash.

3.5.2 Achievement of quality and characterisation of actual culvert units

The durability of the actual culvert units, as affected by the individual manufacturers' construction processes was assessed. The units, cast with concrete sampled for the purpose of mix characterisation, i.e. fly ash mix 1, were used for coring to allow the direct evaluation of the applicable manufacturing process. The three manufacturers had different manufacturing processes as shown below:

- Manufacturer A used shutter vibrators to achieve compaction and indirect steam curing as an accelerated curing method.
- Manufacturer B used shutter vibrators to achieve compaction and indirect steam curing, until a maturity of 360°C-hours was achieved, as an accelerated curing method.
- Manufacturer C used hand-held poker vibrators to achieve compaction and employed a one-day in-mould curing period together with a resin based curing compound to provide adequate levels of curing.

Tests were conducted on 45 mm diameter samples obtained and tested using the methods detailed in section 3.3. Initially, a suitable test age had to be determined. Testing periods at 7, 14 and 28 days were used, but due to the poor early conductivity results, 28 days was adopted as the future testing age.

The results for the testing of the actual products are included in table 3.8 for the relevant manufacturers at the various ages.

Table 3.8: Characterisation of actual units as a check on manufacturing process.

Concrete mix type and sampling age	**Date Cast**	**OPI** Log scale	**Sorptivity** mm/hr^0.5	**Cl conductivity** mS/cm
Manufacturer A:				
Fly Ash-Unit 1	05-11-99			
28 days		9,65	5,1	1,21
Manufacturer B:				
Fly Ash-Unit 1	01-09-99			
7 days		9,77	-	0,87
14 days		10,15	5,2	1,10
28 days		10,08	4,6	0,72
Manufacturer C:				
Fly Ash-Unit 1a	27-08-99			
7 days		10,39	7,4	-
14 days		10,18	-	1,96
28 days		10,21	5,1	1,20
Fly Ash-Unit 1b	27-08-99			
7 days		10,09	6,3	1,46
14 days		10,05	-	1,31
28 days		10,08	6,0	1,21

All the durability index results were better than the limits stipulated in table 3.6. However increasing variability was observed, compared to fully wet cured cube samples. OPI values were all less than those achieved in hand-tamped concrete cube samples, indicating that compaction was more difficult to achieve in actual units. OPI reductions of approximately 0,5 were determined for actual units at 28 days, representing half an order of magnitude increase in gas permeability compared with fully wet cured 28 day samples. Generally, all the manufacturers achieved OPI values on actual units in either the good or excellent durability classes, probably due to improvements in the workability of concrete mixes containing fly ash. The sorptivity values reduced with increasing age (although differences are small), with steam-cured units (from both Manufacturer A and Manufacturer B) achieving sorptivity values between 4,6 and 5,1 mm/√hour. Manufacturer C was able to achieve similar, yet marginally worse, sorptivity values between 5,1 and 6,0 mm/√hour. Generally, the sorptivity values of the units were in the excellent durability class. The units were found to have similar sorptivity values to those achieved in the mix characterisation process, indicating that a real effort was made by the manufacturers to effectively cure the units.

The conductivity values were also found to decrease with increasing age. At 7 and 14 days, the conductivity values were high, sometimes near the stipulated limit of 2,0 mS/cm. However, at 28 days this value had decreased significantly to more acceptable levels. For this reason, it is recommended that, in future, testing on units be carried out not earlier than 28 days. Manufacturer A and Manufacturer C were able to achieve conductivities in the good durability class, while Manufacturer B was able to achieve results in the excellent durability class at 28 days. This was probably the result of the differences in the individual mix ratios used by the different manufacturers.

Completed units were conveniently stored on the grounds of the factory and left exposed to the environment. Weather records were obtained from the weather office at Cape Town International Airport. Analysis of the data shows that the ambient conditions, for the period corresponding to product testing at Manufacturers B and C, could be summarised by a mean temperature of 13,5°C, an average relative humidity of 77% and the total precipitation for the period of 114,1 mm. Analysis of the data for the period corresponding to product testing at Manufacturer A could be summarised by a mean temperature of 18,9°C, an average relative humidity of 66% and total precipitation for the period of 14,2 mm. The higher precipitation levels could be accountable for the minor improvements of index values for Manufacturers B and C. Comments regarding Manufacturer A were withheld due to insufficient data.

3.5.3 Control of concrete cover to steel

Cover to reinforcing steel was determined along the width of 10 randomly selected units at two manufacturers (Manufacturer B did not produce a sufficient number of units). Compliance with the required minimum cover of 40 mm was tested and analysed according to the method documented in section 3.4. With the exception that bars with less than 40 mm cover were noted; within this set of bars, covers less than 20 mm were then noted. The results are presented in table 3.9.

Table 3.9: Analysis of cover to steel measurements.

Unit Size & Class	Construction Date	Incidence of cover < 40mm	Incidence of cover < 20mm	Cover: Inside Mean mm	Cover: Inside Standard deviation	Cover: Inside c.v. %	Cover: Outside Mean mm	Cover: Outside Standard deviation	Cover: Outside c.v. %
Manufacturer A:									
3000X900, 75S	24-11-99	17	-	33.8	19.2	56.9	45.1	5.8	12.8
3000X900, 75S	24-11-99	24	-	35.2	17.5	49.6	29.8	9.7	32.7
3000X900, 75S	19-11-99	24	-	30.7	11.0	35.9	40.1	14.3	35.6
3000X900, 75S	10-11-99	24	-	32.6	14.5	44.6	37.4	17.3	46.3
3000X900, 75S	03-11-99	19	-	34.3	17.6	51.4	49.8	12.6	25.4
3000X900, 75S	19-11-99	13	2	29.6	14.6	49.1	67.3	4.6	6.8
3000X900, 75S	05-11-99	22	4	30.2	19.9	65.8	39.1	12.7	32.4
3000X900, 75S	23-11-99	21	-	31.7	13.7	43.1	43.9	8.3	18.9
3000X900, 75S	02-11-99	22	2	33.8	22.9	67.6	48.2	27.3	56.6
3000X900, 75S	01-11-99	20	1	32.7	12.3	37.7	33.1	12.5	37.6
Manufacturer C:									
3600X1200, 75S	05-11-99	1	-	51.8	5.2	9.9	52.8	3.5	6.7
3600X1200, 75S	05-11-99	-	-	58.2	6.4	10.9	56.8	5.7	10.0
3600X1200, 75S	05-11-99	-	-	51.7	6.0	11.7	55.1	7.9	14.3
3600X1200, 75S	10-11-99	-	-	50.2	6.9	13.8	49.0	2.8	5.7
3600X1200, 75S	25-11-99	1	-	48.3	6.0	12.5	45.9	3.0	6.5
3600X1200, 75S	26-11-99	-	-	50.0	5.0	9.9	52.6	3.3	6.3
3600X1200, 75S	23-11-99	2	-	49.3	8.9	18.1	52.1	2.3	4.4
3600X1200, 75S	23-11-99	-	-	53.2	8.6	16.2	52.1	3.5	6.7
3600X1200, 75S	03-11-99	1	-	49.6	4.6	9.3	48.6	4.7	9.7
3600X1200, 75S	19-11-99	-	-	51.2	4.8	9.4	51.3	1.6	3.1

An anomaly occurred with the cover to steel obtained from Manufacturer A. It was evident that Manufacturer A had designed the reinforcement to achieve minimum cover of 20 mm as required by SABS 986-1994 instead of the required 40 mm. The control of covers achieved, as measured by the coefficient of variation, also fluctuates substantially. Since coefficients of variation in the order of 50% are being achieved, this gives an indication that the required control was not exercised. Some units were found to have up to 4 bars with a cover below the 20 mm "design" limit. The consulting engineers recalled the units from service, which were subsequently replaced by units adequately achieving 40 mm cover.

Manufacturer C managed to obtain considerably better results. Some units were found to have up to 2 bars that had covers less than the stipulated 40 mm. Generally the coefficient of variation was found to be in the order of 10%. This indicates that Manufacturer C achieved excellent control in bending and placing of reinforcing steel. Discussions held with Manufacturer C revealed that the reinforcement layout had been designed using a nominal cover of 50 mm. The isolated incidents of low cover can therefore be attributed to isolated loose reinforcing bars.

3.6 CURRENT MANUFACTURING TRENDS AND SUGGESTED MODIFICATIONS TO CURRENT PRACTICE

Discussions held early in 1999 with precast manufacturers revealed a general reluctance to the use of cement extenders to achieve durability in box culvert construction. Manufacturers did recognise that consulting engineers were increasingly requesting covers of 40 mm. The manufacturers expressed concern at this trend as material costs constituted a significant portion of the total unit cost. Following the completion of the case study requiring the mandatory use of fly ash, the negative perceptions relating to the use of extenders had been overcome. During 1999, PPC Cement launched Ground Granulated Corex Slag (GGCS), the first locally sourced cement extender in the Western Cape. The slag is currently cheaper than cement in the Western Cape. Due to the inherent cost savings, two manufacturers have opted for standard culvert mixes containing 50% GGCS. Although these mixes are likely to have improved resistance to aggressive ions (Mackechnie *et al*, 1999), particularly chlorides and sulphates, slag concretes may have increased carbonation rates (Mackechnie, 1999).

The construction of box culverts appears well suited to utilize performance-based specifications. Most precast manufacturers have well equipped concrete laboratories and testing facilities and are increasingly becoming ISO 9001/2/3 compliant. The durability index tests are inherently simple, practical tests and require only small capital investments to acquire the necessary equipment. The minimum cover requirements and appropriate levels of concrete quality can be easily integrated in a performance-based specification.

It is important that a degree of flexibility be incorporated in any specification relating to the precast industry. This allows the manufacturers to develop proprietary technologies and systems, thereby keeping the industry competitive. A performance-based specification is tentatively suggested in table 3.10. This specification is applicable to culverts destined for an aggressive environment (defined in section 2.2) with a high susceptibility to attack by carbonation. A design life of 25 years is assumed, corresponding to the expected road pavement life. The suggested values have been inferred from data collected in chapters 2 and 3, Mackechnie (1999) and Mackechnie *et al* (1999) and relates primarily to Western Cape construction materials.

Table 3.10: Suggested performance levels for various cover depths.

Required Durability Indexes	Minimum Cover		
	20 mm	30 mm	40 mm
OPI (log scale)	>10,40	> 10,00	> 9,50
Water Sorptivity (mm/hr^0.5)	< 6,0	< 10,0	< 15,0
Chloride Conductivity (mS/cm)	< 0,75	< 1,50	< 2,5

The values suggested in table 3.10 apply to index tests conducted on fully wet cured samples. It must be stressed that the suggested index values are not sufficient in extreme, very severe or severe marine environments or in sulphate-bearing environments. Specialist advice, particularly recommendations of Mackechnie *et al* (1999), should be consulted in such instances. Further testing at manufacturers is required to produce a similar table relating to the minimum levels of quality required in actual units. However from the limited results presented in subsection 3.5.2, a reduction of 0,25 could be allowed for OPI measurements while water sorptivity and chloride conductivity values should be upheld for testing of the actual units.

Furthermore, the monitoring of steam temperatures needs to be conducted on a regular basis, particularly in summer, at precasting yards. The use of less severe steam curing practices should be made mandatory. Modern limits relating to the recommended steam practice should also be documented in the SABS specification.

The improvements in the quality of the concrete will be of little use if the minimum cover specified is not adequately attained. A blanket increase of the minimum cover will do little to improve the current situation. However, the use of nominal covers suggested in table 3.5 will considerably improve achievement of the required cover. The SABS 986 specification also needs to provide a suitable method to check the cover achieved and suggest a suitable sampling rate to allow regular testing of units.

3.7 SUMMARY

Steam curing practice as used by the Western Cape precast manufacturers was found to be severe, and often exceeds the recommended and durability limits. Premature drying and cracking of units was observed due to the use of harsh steam curing practice.

Problems in achieving adequate cover, first identified in chapter 2, are still prevalent in the industry. A table of nominal covers, based on the standard of control of reinforcing steel bending and placement, have been recommended in an attempt to overcome the present trend.

The concrete used in box culvert construction is essentially normal structural concrete having good to excellent durability characteristics. The use of fly ash (30% replacement) considerably improves the durability of the concrete. The concrete quality is shown to decrease in the actual units, probably as a result of less effective compaction. The influence of accelerated curing procedures cannot be excluded. The effect of accelerated curing on concrete durability still remains to be quantified.

Reductions in cover for precast box culverts can be maintained, however suitable performance-based specifications need to be instituted to provide a realistic reason for the reductions in cover. A matrix relating required concrete quality with minimum covers is suggested for typically aggressive Western Cape environments. The matrix allows freedom to optimise the mix using any suitable binder type and water:binder ratio, while existing limits on concrete strength and proof load requirements are maintained.

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Chapter 4:

Quantifying the Effects of Elevated Temperature Curing on Concrete Durability

Precast concrete manufacturers use accelerated curing methods primarily for economic reasons. Accelerated curing of concrete, particularly heat curing, is used as a simple method to provide rapid gains in early-age strength. The provision of early strength allows the product to be removed from the formwork earlier; as a result the mould turnover period is shortened, thereby increasing productivity. Increases in productivity lead to additional benefits derived from the more effective use of both plant and labour, but the rate of production must at least be doubled to realise these additional benefits. The increases in productivity lead directly to a reduced unit cost of the product and therefore higher profit margins.

Further benefits of accelerated curing include the earlier handling and transporting of precast elements. Storage periods are suitably reduced as the elements have sufficient strength to allow earlier transportation. A consequential reduction in stock volumes and storage space otherwise required for normal curing, results and the precast manufacturer's cash-flow position is improved (Kirkbride, 1971). The overall product construction time is also reduced, thus providing the precast manufacturers the ability to satisfy rush orders at short notice. The increasing use of fast track construction techniques strongly favours the use of precast concrete elements, as effective cost savings are made through reductions in on-site construction time.

It is a well-researched fact that the use of accelerated curing provides early gains in concrete strength but compromises long-term strength. Research evidence suggests that this phenomenon is due to the altered cement hydration mechanism and microstructural changes that occur as a result of the use of accelerated curing methods. The various types of accelerated curing available for use are discussed. In addition to strength effects, the changes brought about with accelerated curing techniques may affect the final quality, particularly durability, of the precast unit. To date little research, with regard to the effects of accelerated curing techniques on the durability of concrete, has been undertaken. Experimental evidence is presented which quantifies the effects of steam curing regimes on the durability of wet-cast concrete, as measured using the durability index approach.

4.1 TYPES OF ACCELERATED CURING METHODS

The hydration of cement is essentially an exothermic chemical reaction, and is affected by ambient temperature and specific characteristics of the cement such as chemical composition and fineness. Accelerated curing is most commonly achieved through the use of thermal means, but the use of high early strength cement or accelerating admixtures may also produce the required effect.

Thermal methods involve the supply of heat in order to speed up the hydration reaction. The heat can be provided in a variety of ways, particularly through the use of fluid mediums such as water, steam, air or oil. The heat may be delivered either directly to the concrete or indirectly via conduction.

Hot water may be introduced directly into the mix, but the additional heating of aggregates and moulds is required to avoid initial heat losses. Introducing steam directly into the mix is a more efficient way of directly heating the concrete mix. Alternatively, the products may be placed in a curing shed and steam introduced directly to cure the products. Similarly hot, humidified air can also be blown into enclosures or hot oil may also be passed through chambers containing water, thereby creating steam.

In the Western Cape, box culverts are constructed by casting into steel moulds. A natural steel-lined chamber is formed along the interior face of the culvert. Currently, saturated steam is introduced into this chamber. The steam condenses on the steel surface, releasing its latent energy, which is then conducted through the concrete. Since the steam is never in contact with the concrete, drying of the concrete needs to be prevented, especially at high temperatures. Blowing heated air into the chamber, passing hot water through pipes embedded in the moulds or the use of bonded electrical heating elements, would produce similar results. Therefore the terms heat curing and elevated temperature curing are used to describe this indirect thermal curing method. Saturated steam is currently the most efficient method of providing the required heat. Since steam will preferentially condense on the cooler portions, an even supply of heat is obtained.

The heat produced by the exothermic reaction of cement hydration can be utilised by making use of insulated moulds and insulating exposed concrete surfaces. There is a need for a high cement content (in the region of 500 kg/m^3) to generate sufficient heat, but the peak temperature (approximately 30°C) may only be reached 8-12 hours after mixing.

Heat may be generated electrically by making use of the resistance of the reinforcement, but a low-voltage high-current supply and an expensive heavy duty transformer is required (Levitt, 1982). Alternatively, wire coils or heating panels (heating panels bonded to the steel plate) can be placed in contact with steel moulds. Even distribution of heat must be assured to avoid developing excessive thermal gradients and sufficient moisture must be maintained in the concrete at all times.

Experiments using infra-red and microwave methods have been tested. It was found that infra-red methods only heat the concrete surface and that the microwave method requires expensive plant. Microwave curing methods are also sensitive to the moisture content. Neither method is considered viable for use with large precast units (Levitt, 1982).

Ultra high early strength Portland cements have a very high fineness, between 700 and 900 m^2/kg (Addis, 1994), and are manufactured by separating fines from rapid hardening cement. It is possible to obtain 24-hour strengths four to seven times higher than with the use of ordinary Portland cement. By applying additional heat treatment, compressive strengths of 35 MPa can be obtained in four hours (Kirkbride, 1971).

The chemical admixture most widely used for accelerating the hardening of concrete is calcium chloride. Calcium chloride has the effect of significantly increasing the rate of hardening. It is possible to double the 1-day compressive strength with a dosage of 1,5%

by mass of cement (Kirkbride, 1971). The use of this chloride-containing admixture is now banned by most building codes due to likely reinforcement corrosion problems. Other non-chloride admixtures are available, but are costly relative to calcium chloride.

4.2 THE EFFECTS OF ELEVATED TEMPERATURE CURING ON CEMENT REACTION KINETICS

When water is initially added to Portland cement, a number of ions are released from the calcium silicate minerals. These ions react to form calcium silicate hydrate (CSH). In a through-solution mechanism, the hydrated ions form at the surface, pass into solution where the CSH is formed. In a topochemical mechanism, the CSH forms directly on the surfaces of the silicate grains. The rate and mechanisms of hydration of cement are controlled by the concentration and constitution of the liquid phase at various sites, nucleation and crystal growth in the liquid or in hydrates, the composition and morphology of hydrates, the pore structure and the ease of mass transfer in the product (Kondo *et al*, 1968). The hydration temperature and the degree of hydration of the system directly influence these factors.

Physical observations of the hydration process show that at early hydration stages, a chemical process other than diffusion controls the reaction rate. At more advanced stages, the hydration products are formed only at the surface of the reactant. At later stages, diffusion of the dissolved cement compounds into the bulk solution through a product layer is the rate-controlling step. Kjellson (1990) modelled the apparent activation energy with varying degrees of hydration. He found that up to a degree of hydration of 30%, the activation energy is typical of a chemically controlled process. After a degree of hydration of 65% was reached, the activation energy indicated that the reaction had become a diffusion-controlled process.

Through scanning electron microscopy (SEM) Kjellson concluded that a shift from through-solution to topochemical hydration mechanism occurred from degree of hydration of approximately 30%. This effect is favoured at lower hydration levels for higher curing temperatures. Verbeck and Helmuth (1968) first proposed a theory dealing with the influence of temperature on the hydration characteristics of cement. They stated that at low temperatures, and therefore slow hydration rates, sufficient time allows the hydration products to diffuse and precipitate uniformly throughout the interstitial space among the cement grains. However at higher temperatures, the rate of hydration increases to such an extent that there is insufficient time to allow the diffusion of the hydration products. Therefore a high concentration of hydration products is built up encapsulating the hydrating cement grain. They postulated that the presence of dense rims around hydrating cement grains could lead to retardation of subsequent hydration and non-uniform distribution of the concrete microstructure (and therefore reduced ultimate strength).

Recently Escalante-García *et al* (1998) conclusively showed that the anhydrous phases in cement had different rates of reaction at elevated temperatures. Through the use of quantitative X-ray diffraction, they found the early hydration of the anhydrous phases was accelerated by increases in the hydration temperature. A temperature inversion was established for the hydration of alite (C_3S) and the ferrite (C_4AF) phases. Thus lower ultimate degree of hydration corresponded to the highest curing temperature for these phases. Since C_3S is the most abundant phase in modern cements, a lower ultimate degree of hydration can be expected with the use of elevated curing temperatures.

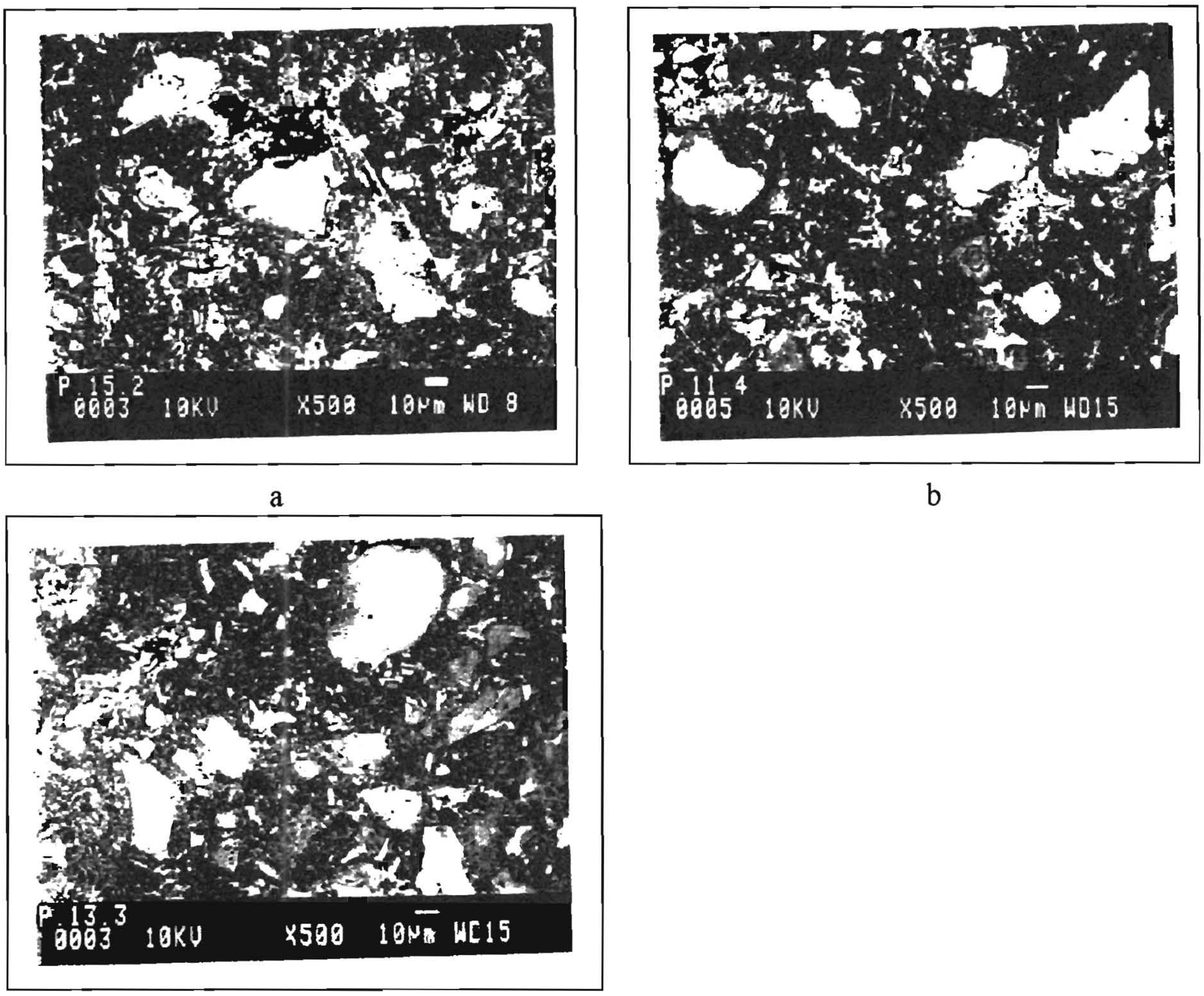
Odler *et al* (1986) concluded that elevated temperature curing altered the nature of the hydration products. They observed a decrease in the ratios of bound water to both hydrated C_3S and free lime. They suggested that a CSH phase with a lower water content and higher calcium-silicate ratio was formed at elevated temperatures.

The combined effect of the reaction kinetics results in physical changes in the microstructure of concretes hydrated at elevated temperatures.

4.3 THE EFFECTS OF ELEVATED TEMPERATURE CURING ON THE DEVELOPMENT OF CONCRETE MICROSTRUCTURE

Through the use of SEM techniques applied to plain cement pastes, Kjellson (1990) confirmed the hydration shell theory proposed by Verbeck and Helmuth. Backscattered electron imaging techniques were used to identify between the different relative densities of different phases of the microstructure. Up to 30% hydration, no significant difference was noted for both occurrence of hydration shells or the distribution of the CSH matrix between curing temperatures of 5°C and 50°C. At 70% hydration, the hydration products of specimens cured at 5°C were generally well distributed throughout the matrix, and the interstitial pores were found to be small. Specimens cured at 20°C did develop shells around most of the hydrating grains while the interstitial space was more porous. However specimens cured at 50°C had very distinct hydration shells and exhibited an inhomogeneous distribution of CSH matrix. A significant increase in porosity was observed for specimens cured at 50°C. Using variable curing temperature regimes Kjellson observed that the curing temperature after 30% hydration largely influences the distribution of hydration products at later stages (70% hydration). A visual account of Kjellson's findings is shown on the following page in figure 4.1.

Escalante-García *et al* (1998) confirmed Kjellson's observations for cement samples hydrated at 10°C and 60°C for 360 days. They found that samples hydrated at 10°C showed a more compact and more homogeneously distributed structure with less apparent porosity. A gap around some of the hydrating cement grains was noted for samples cured at 60°C.



a b

c

Figure 4.1: Backscattered electron images of isothermally cured cement pastes hydrated at (a) 5°C, (b) 20°C and (c) 50°C (Kjellson *et al*, 1990)

Besides physical observations of porosity, Kjellson performed greyscale image analysis of a number backscattered electron images (BEIA) in order to quantify the effect of elevated temperature curing on the relative portions of hydrated cement phases. A notable increase in porosity was determined by BEIA for samples cured at elevated temperatures. Mercury intrusion porosimetry (MIP) studies confirmed the relative increase in porosity, however these values cannot be used for direct comparison purposes. MIP is able to measure much smaller pore sizes than BEIA, but tends to exaggerate the volume of the smaller pores due to the "ink bottle effect" (Kjellson, 1990). MIP measurements tend measure the size of the entrance of the pore. Consequently, large pores with relatively small entrances are reflected as a large number of pores having a nominal size equal to the pore entrance. This inaccuracy is termed the "ink bottle effect".

Table 4.1: Total porosity of samples isothermally cured at different temperatures to 70% hydration (Kjellson, 1990).

Curing Temperature °C	Porosity (MIP)	Porosity (BEIA)
5°C	33.2%	4.3%
20°C	34.2%	10.9%
50°C	35.7%	15.1%

Kjellson performed MIP measurements and found similar pore size distributions, except that the volume of pores of radius 200-1000 Å (10000 Å = 1 μm) increased from 5°C to 50°C. From BEIA, the difference in pore size distribution occurred for pores of radius 2500 to 12500 Å. Radjy *et al* (1973) suggested that particle growth occurs by a redistribution of pore volume. In their model small pores are annihilated while larger pores are developed at elevated temperatures. They measured a corresponding coarsening of CSH particles through decreases in the BET (water) surface area at elevated temperatures. Idorn (1968) reviewed work showing that measurements of the specific surface area of hydration products decreases with increasing temperature at later ages of hydration, confirming that coarser particles are formed at elevated temperatures. In Kjellson's review of curing at different temperatures, he presents the work of other researchers confirming the decrease in surface area of hydrated C_3S pastes. The cited author notably calculated an increase of the average hydraulic radius of pores at elevated temperatures.

The effects of elevated temperature curing on concrete microstructure can be summarised as a coarsening of CSH particles with a corresponding increase in the formation of large pores. Since properties such as strength, durability, shrinkage, creep, and ionic diffusion are directly influenced or controlled by the relative amounts of the different types and sizes of pores (Hearn *et al*, 1994), the observed increase in porosity may have potentially adverse effects on concrete cured at elevated temperatures.

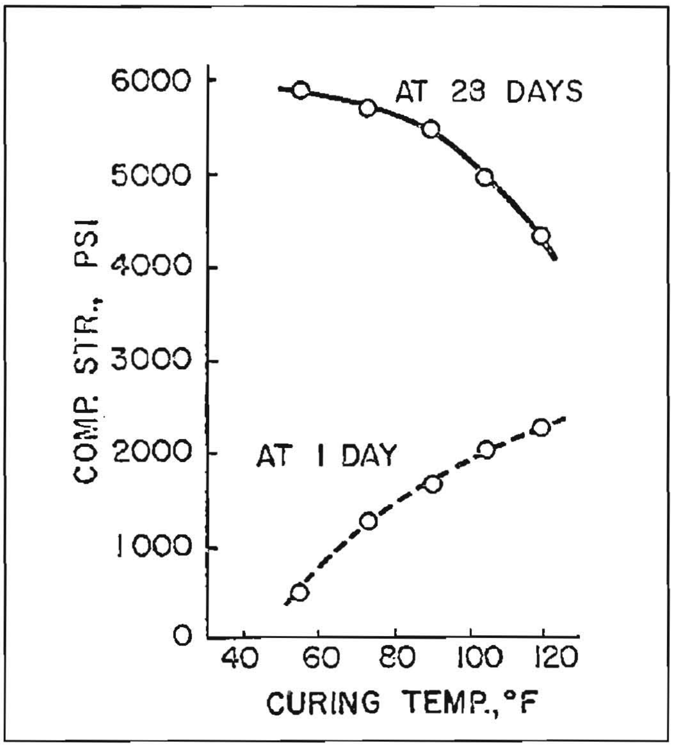
4.4 EFFECT OF ELEVATED TEMPERATURE CURING ON MECHANICAL PROPERTIES

The adverse effects of elevated temperature curing on concrete mechanical properties, particularly compressive strength, have long been established. Some authors even accept the reduction in long-term properties as a compromise for high early strength (Neville, 1981). The increase in porosity and the reduced ultimate degree of hydration have been used to explain the reduction of later age mechanical properties (Verbeck *et al*, 1968).

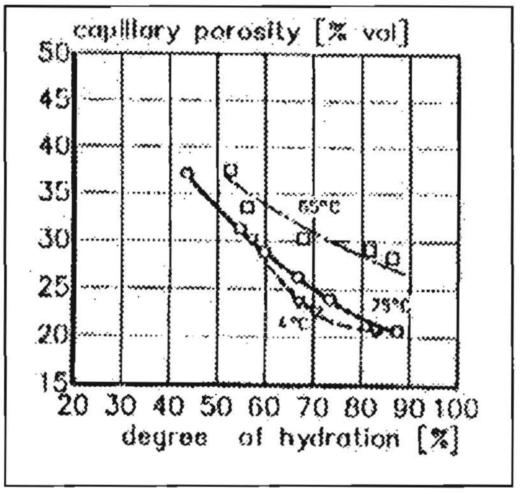
Maturity models have been extensively used to predict early age concrete strengths and thereby product stripping times. But the maturity, expressed as the product of curing temperature and time, is subject to the type of cement, water-cement ratio, range of optimum temperatures, duration of treatment, and the delay period used to cure the product (Saul, 1951). The maturity model fails to predict concrete strength at later ages and is not able to accurately account for microstructural changes brought about by elevated temperature curing. Kjellson (1990) proposed the use of a model based on the calculation of apparent activation energy as a function of the relative strength. He reported a distinct improvement in the prediction of strength at later ages.

4.4.1 Compressive strength

Steam curing parameters, particularly delay period, curing temperature and rates of heating and cooling, have been investigated in order to optimise the compressive strength of concrete products. However durability requirements, as presented in chapter 3, dictate modern steam curing procedures. Generally, steam curing improves the early compressive strength of steam-cured concretes up to about 7 days. Thereafter, the compressive strength of low temperature wet cured concrete produce improvements that become increasingly important at later ages (Al-Rawi, 1976, Ravina *et al*, 1971 and Verbeck *et al*, 1968). Typical compressive strength trends at 1 day and 28 days in relation to curing temperature are shown in figure 4.2(a).



a



b

Figure 4.2: (a) One day strength increases with increasing curing temperature but 28-day strength decreases with increasing curing temperature (Verbeck *et al*, 1968) and (b) capillary porosity of C_3S pastes hydrated at various temperatures (Bentur, 1979).

Shideler *et* al (1949) found strength reductions up to 45% in the 28 day compressive strength of concrete initially steam cured at 85°C. Soroka *et* al (1978) briefly reviewed work by other researchers and reported strength reductions of 15% for curing temperatures between 50°C and 60°C, and 20% for temperatures of 80°C and 90°C. Seroka *et al* determined strength reductions of 22-35% at 28 days and 26-40% at 90 days for concretes steam cured at 70°C. Sherman *et al* (1996b) observed strength reductions of 8-14% at 28 days and 17-21% at 180 days for concretes heat-cured at 63°C. Recently Hewlett (1998) showed average strength reductions of 18%, 12% and 7% for concrete cubes initially cured at 95°C, 85°C and 75°C respectively and then stored in 20°C water until 28 days. These strength reductions were generally attributed to increased porosity, microstructural changes and reduced ultimate degree of hydration with the use of elevated curing temperatures.

Bentur *et al* (1979) showed that the capillary porosity reduces with increasing degree of hydration. However more capillary porosity is observed for higher curing temperatures than for lower temperatures at equal degrees of hydration (figure 4.2(b)). Long established correlations exist between reductions in concrete compressive strength and increases in capillary porosity. Similarly Van Breugel (1995) argued that the volume of cement involved in the formation of interparticle contacts could be considered as a universal strength parameter, as opposed to the degree of hydration. The effect of embedded cement volume on concrete strength is found to be independent of both the water-cement ratio and curing temperature. Comparison of the amounts of embedded cement calculated at equal degrees of hydration but for different curing temperatures allow the prediction of strength relative to a reference curing temperature. Predicted strength reduction factors for isothermally cured concretes, validated by experimental data, are presented in figure 4.3.

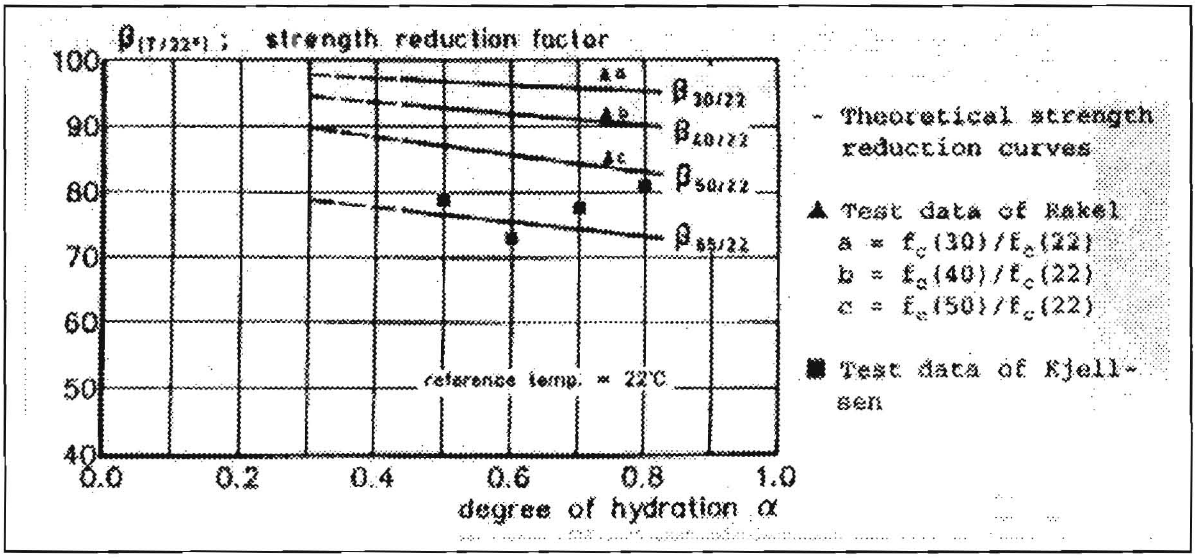


Figure 4.3: Reduction factors for the influence of temperature on strength development. Kjellson's data plotted for relative strengths of pastes cured at 5°C and 50°C, i.e. a net difference of 45°C (Van Breugel, 1995).

Soroka *et al* (1978) suggested that the long-term strength reduction, for samples cured at moderate temperatures for short periods (2-5 hours), occurred due to the lack of adequate curing of the specimens. They found that a large proportion of the 28-day strength could be recovered with 7 days supplementary wet curing.

French *et al* (1993) suggested that heat curing improved the early age performance of concretes containing microsilica. Heat-cured microsilica concretes were able to rapidly reach maturity i.e. 85% of 28-day strength was reached after 24 hours.

4.4.2 Flexural, tensile strength

Klieger (1958) showed that curing temperature influences flexural strength development in a similar manner as with compressive strength. Hanson (1963) measured the cylinder-splitting strength and also found similar trends to those obtained for compressive strength. However, the proof load test (SABS 986-1994) applied to box culverts at the manufacturer should ensure that a minimum level of flexural strength is achieved.

4.4.3 Modulus of elasticity

Radjy *et al* (1973) showed that the dynamic modulus of elasticity was irreversibly reduced as the severity of the heat treatment increased. Hanson (1963) and Higginson

(1961) measured the static modulus of elasticity and found that the effects were similar to that of compressive strength.

4.4.4 Creep and shrinkage

Considerable reductions in drying shrinkage have been observed with increasing temperature of steam cure and with increasing length of time of steam cure (Higginson, 1961). Kirkbride (1971) reports that reductions of creep up to 50% have been reported with the use of steam curing. However, the effects of steam curing on both creep and shrinkage have not been isolated from the influence of rapid drying due to steam curing.

4.5 EFFECT OF STEAM CURING ON CONCRETE DURABILITY

Although the effects of elevated temperature curing on concrete strength have long been established, relatively little remains known about its effects on concrete durability. The formation of coarser CSH particles, heterogeneous microstructure, increased porosity and reduced ultimate degree of hydration will undoubtedly result in reduced durability of concrete cured at elevated temperatures. Permeability, absorption and diffusion are the transport mechanisms that are most often measured in order to assess the potential durability of a concrete (Alexander *et al*, 1999a).

Permeability is one of the most important properties of durable concretes as it represents the ability of a particular concrete to resist the ingress of fluids. Higginson (1961) showed that steam curing (with and without additional fog curing) adversely affected the water permeability of concrete. Goto and Roy (1981) examined concretes with water-cement ratios from 0,35 to 0,45 and found that concretes cured at higher temperatures (60°C) had significantly higher water permeability than concretes cured at 27°C.

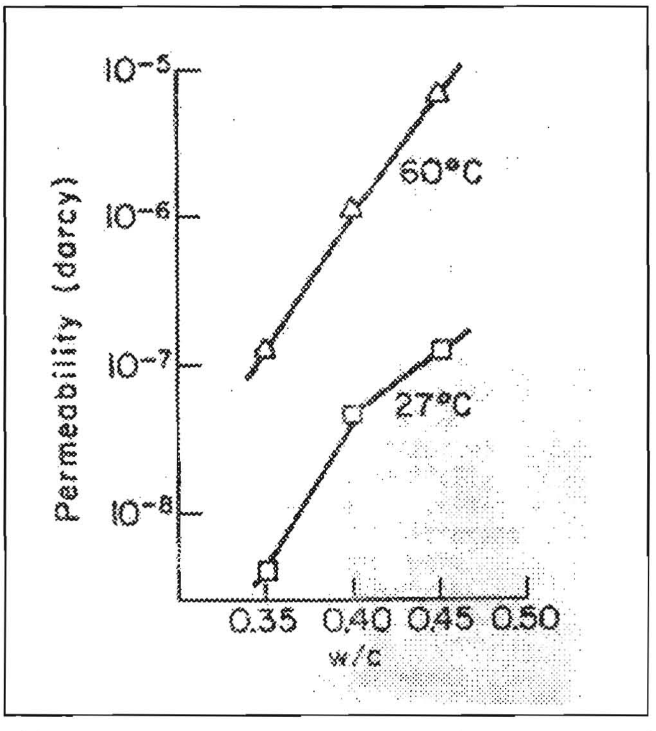


Figure 4.4: Relationship between permeability and w/c ratio at different temperatures (Goto and Roy, 1981).

Diffusivity is the material property that provides resistance to the ingress of aggressive ion species. Detwiler *et al* (1991) make reference to concrete beams of the San Mateo Bridge over San Francisco Bay that are subject to marine exposure. Both steam cured and naturally cured beams of high quality (w/c = 0,45, 375 kg/m^3 cement content) were used. The steam cured beams had to be repaired after 17 years due to corrosion damage. In contrast, Sherman *et al* (1996a) cite both site exposure data and laboratory data of a wet-dry cycle 15% NaCl ponding test conducted over a period of one year. They found, using profiling techniques, that heat-cured conventional concretes had substantially lower chloride ion concentrations at predetermined depth increments.

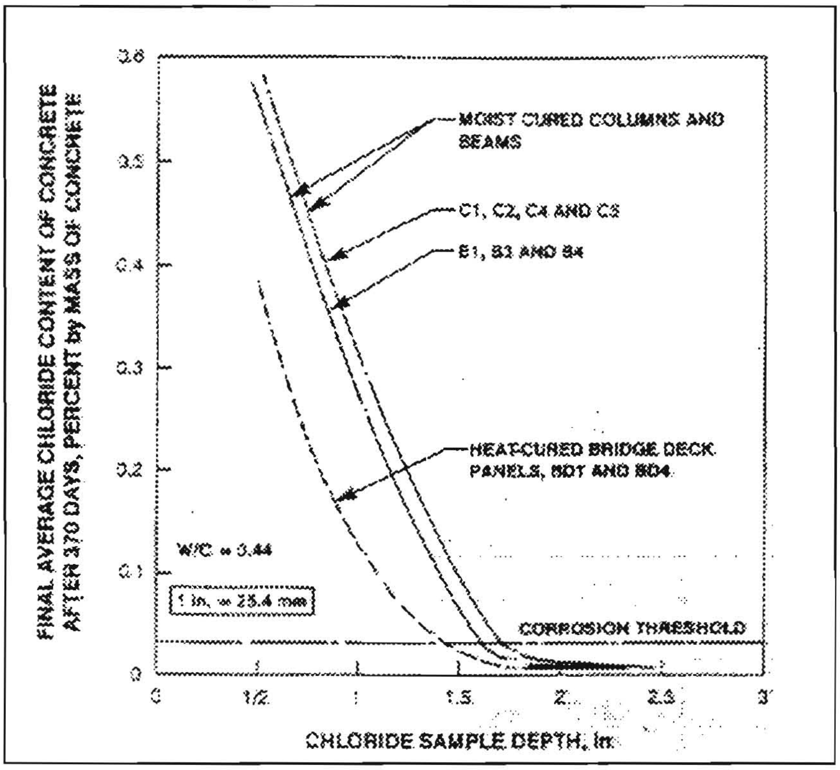


Figure 4.5: Measured chloride profiles in moist-cured and heat-cured concretes (Sherman *et al*, 1996a).

Sherman *et al* (1996b) tested slabs subjected to continual ponding with a chloride solution for a period of one year. They found that heat cured concrete had generally lower acid-soluble chloride ion concentrations at near surface regions than fully wet-cured, burlap-cured and even 5% silica fume mixes. But the gradient of the profiles seem to indicate that the chlorides diffuse into the heat-cured concrete at a similar, if not faster, rate than burlap-cured specimens. Water-cured (at low water-cement ratios) and silica fume (5% and 7,5%) specimens seemed to have the lowest rate of chloride diffusion.

Sherman *et al* (1996a) discuss the shortcomings of the AASHTO T277 rapid chloride permeability test, particularly the arbitrary 1000-Coulomb acceptance criterion and the exaggerated results obtained with silica fume-modified concretes. Although the AASHTO T277 test does not inherently measure chloride permeability, it may be used as an indicator of chloride permeability if the concrete mix parameters are held constant. The chloride permeability of heat cured concretes expressed as a percentage of equivalent wet cured concrete can therefore be used to indicate potential durability. A comparison of the data collected by researchers is presented in table 4.2. It is interesting to note that heat cured mixes with lower water:cement ratios exhibited larger increases in relative chloride ion permeability and therefore larger proportions of microstructural damage.

Table 4.2: Increase in chloride ion permeability (as determined by AASHTO T277) of concretes cured at elevated temperatures expressed as a percentage of fully wet cured samples.

		Percentage increase in chloride permeability			
		Constant Temperature		Steam Temperature	
Material Parameters	W/C	50°C	70°C	65°C	63°C
(After Detwiler et al, 1994)					
Portland Cement	0,40	52,5%	68,3%	52,5%	-
Tested at 28 days	0,50	24,6%	38,8%	34,7%	-
(After Sherman et al, 1996b)					
Portland Cement	0,32	-	-	-	22,3%
Tested at 42 days	0,37	-	-	-	28,3%
	0,46	-	-	-	14,7%

Alternatively, Detwiler *et al* (1994) used the Norwegian chloride test to measure the rate of chloride diffusion and found that the diffusion rate significantly increased with higher curing temperatures. Detwiler *et al* (1991) used a "lollipop" test to determine the time to cracking of concrete specimens. But the ability of a concrete to absorb the expansive forces of the steel corrosion products can be directly related to the modulus of elasticity of that concrete. Since the modulus of elasticity varies with curing temperature, the conclusions drawn from lollipop experiments relating to concrete durability must be disregarded. The conclusions reached by Campbell *et al* (1993) recommending the use of modified binder systems to achieve durability for steam-cured concrete on the basis of comparative AASHTO T277 test results, should also be disregarded.

Higginson (1961) also found that steam cured concrete had improved resistance to sulphate attack, negligible effect on abrasion resistance, but lower freeze-thaw resistance. Sherman *et al* (1996b) found that heat curing decreased the absorption and volume of permeable voids in concrete (ASTM C642).

4.6 USING DURABILITY INDEX APPROACH TO QUANTIFY THE EFFECTS OF HEAT CURING ON CONCRETE DURABILITY

The durability index approach makes use of bulk engineering measurements of the relevant transport mechanisms important to the long-term durability performance of concrete structures. The durability index approach has been previously used to characterise concretes for the roller-compacted pipe industry. These relatively dry mixes with low water-cement ratios and partial binder replacements with cement extenders showed excellent durability properties after direct steam curing (Fourie, 1999). However, the results presented in this section represent relatively wet concrete mixes as used in the manufacture of box culverts. Wet cast mixes are likely to be more susceptible to the steam curing procedure than dry mixes, as water has a relatively high coefficient of thermal expansion. Yet current manufacturing practice suggests that box culvert manufacturers are neglecting recommended steam-curing procedures. Ultimately, this could result in further impacts on durability of the box culvert units.

4.6.1 Sample fabrication and treatment

Two 40 MPa concrete mixes were designed that were representative of the box culvert manufacturing industry. The water-cement ratio was fixed at 0,5 and a low slump (30-70 mm) mix with a characteristically high stone content was used. The mix details are listed below in table 4.3

Table 4.3: Concrete mix proportions (per m^3)

Materials:	Series A (kg/m^3)	Series B (kg/m^3)
Water	185	160
Cement (Cem I 42,5)	370	320
19 mm Rheebok Granite Stone	1170	-
19 mm Greywacke Stone	-	1190
Cape Flats Dune Sand	674	759
Sika 163 Superplasticizer (l)	-	4.8
Actual Slump	40-50 mm	20-30 mm

The dry ingredients were added and mixed for 1 minute in a rotary pan mixer. The water was slowly added over the next minute, followed by a further 2 minutes of mixing. The superplasticizer, used in series B, was intermixed with the water before adding into the mixer. Series B was mixed first and the time noted, followed by series A about 10 minutes later. The concrete mixes were cast into ten steel 100 mm cube moulds and compacted on a vibrating table for 30 seconds. The cubes were then placed on racks in the steam chamber in a manner that allowed the steam to pass around all the sides of the steel moulds. The top of the cube was left uncovered to allow rewetting of the finished surface with steam and condensate water. One hour after adding water to the superplasticized concrete mix, the steam supply was activated. Steam was allowed to actively enter the chamber for three hours, after which the steam supply was terminated. The samples were allowed to mildly cool to ambient temperature for approximately 18 hours in the humid chamber. The cubes were then stripped and specimens were air cured for 27 days on slatted wooden racks in the laboratory (20,3°C –26,9°C and 46,2%-73,5% relative humidity).

At 28 days, the specimens were cored to a diameter of 68 mm and two nominally 25 mm slices were cut from either side of the centre of the core. The samples were placed in a drying oven at 50°C for a minimum of 7 days. Also at 28 days, two cubes from each mix (not subject to steam treatment) that were fully wet cured at 22°C were tested to determine the compressive strength in accordance with the relevant provisions of SABS Standard Method 863. Since the focus of this study was durability and not strength, compressive strength values were used only to verify the integrity of concrete mix ingredients. Due to variability in the compressive strength with each bag of cement used, the index values have been normalised to 40 MPa. At least one set of controls was cast with each bag of cement. Fully wet cured (22°C) controls and air-cured controls (placed in steam chamber, steam supply not activated) were used for this purpose.

One cube from each mix was sacrificed and used to monitor temperature development in the individual concrete mixes and the steam temperature. Prefabricated PVC insulated thermocouple wires were inserted in the cube centre and steam chamber and connected to a continuous chart recorder. A consistent trend was observed for the temperature development of the concrete mixes. The maximum temperature recorded for series A was 1-1,5°C above the maximum steam temperature and the maximum temperature recorded for series B was 0,5-1°C below the maximum steam temperature. This difference is most likely due to the differing cement contents of the individual mixes.

The durability indexes were determined to categorise the general effects of heat curing, especially the maximum curing temperature, on the gaseous permeability, water absorption and chloride diffusion properties of concrete.

4.6.2 Oxygen permeability index (OPI)

OPI values were essentially used to characterise the microstructure after steam curing. The test measures the capacity the ability of the concrete, as affected by the curing procedure, to transfer fluids by permeation. Since correlations exist between the depth of carbonation and OPI values, the influence of steam curing may provide further insight into the performance of steam cured box culverts in carbonation susceptible environments. The main variable investigated was the maximum steam curing temperature attained in the steam chamber. Testing was conducted in accordance with test methods and apparatus detailed in *Concrete durability index testing manual* (Alexander, 1999b). OPI test results represent the average of at least 7 samples (out of 9 samples tested) from a particular curing regime and series. Linear regression analysis was conducted on recorded pressure readings (expressed as ln(Po/Pt)) with time and samples with a minimum correlation coefficient of 0,999 were used to determine OPI results. The results of OPI testing in relation maximum steam temperature are presented graphically in figure 4.6.

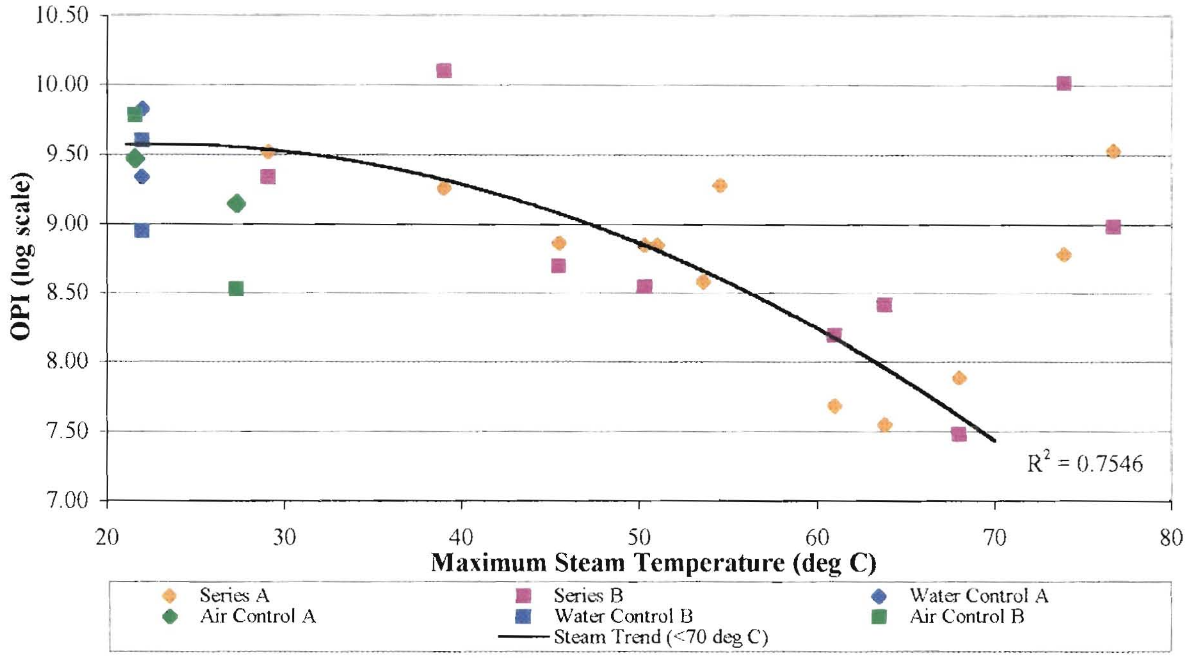


Figure 4.6: Relationship between normalised (40 MPa) OPI values and maximum steam curing temperature.

A general mix-independent relationship is established between the maximum steam curing temperature and the OPI for steam temperatures up to 70°C. The trend, comprising data points from both steam-cured concrete series, indicates that permeability is adversely affected by increases in maximum steam curing temperature. A more permeable and open microstructure, measured by decreasing OPI values, develops with increases in the maximum temperature. Since the OPI represents a negative log scale index, significant increase in permeability result from increases in the indirect steam curing temperature. Further analysis of the data (up to 70°C) shows that the trend represents a second order polynomial with a correlation coefficient of 0,869 (R^2 = 0,755). Since the OPI is particularly sensitive to variations in the degree of compaction, the trend is considered to be strong.

However a marked increase in the OPI values was observed for samples cured above 70°C. An increase in the degree of hydration at temperatures above 70°C may be capable of partially masking the adverse effects of heat curing. However concretes heat-cured at temperatures of this order may be susceptible to attack by DEF (see subsection 3.1.2).

4.6.3 Water sorptivity

Water sorptivity testing is used to measure the rate at which water is absorbed into concrete under the action of capillary forces. The capillary suction is a function of the pore geometry, and is a useful indicator of the nature and extent of early curing on the concrete. Water sorptivity testing was conducted in accordance with test methods and apparatus detailed in *Concrete durability index testing manual* (Alexander, 1999b). The sorptivity test results represent the average of at least 5 samples (out of 6 possible samples) from a particular curing regime and series. Linear regression analysis was conducted on recorded mass readings (expressed as mass of water absorbed (g)) with the square root of time and samples with a minimum correlation coefficient of 0,99 were used to determine sorptivity results. Test data is shown graphically in figure 4.7.

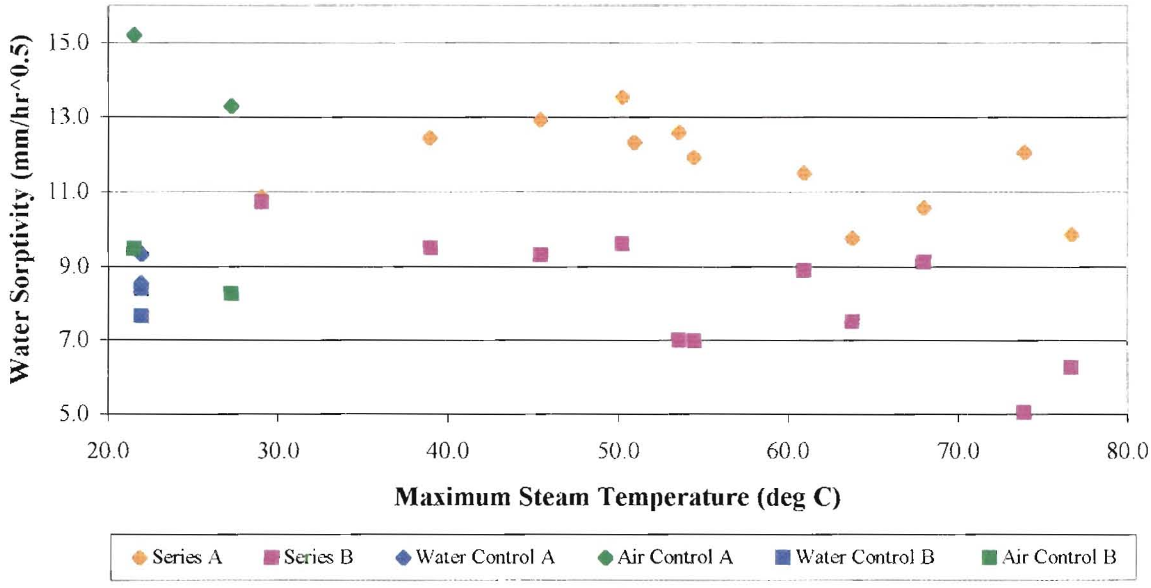


Figure 4.7: Relationship between normalised (40 MPa) water sorptivity values and maximum steam curing temperature.

No definitive trends could be established for the water sorptivity test data. Series A performed consistently worse than series B when compared to the water cured control representative of each mix. Lower sorptivity values than the water controls were measured for series B at curing temperatures above 70°C.

However further observations suggest the presence of two mechanisms with a likely inversion point occurring at 50°C. Between 30°C and 50°C a steady increase in the water sorptivity with increasing temperature is noted. It is known that absorption is related to capillary porosity. Seemingly the increase in water sorptivity may be related to a corresponding alteration in the pore structure. Yet at temperatures above 50°C, a decrease in water sorptivity is noted with increasing temperatures. This improvement, especially at temperatures above 65°C, suggests that the microstructural change occurring at lower temperatures has been overcome. The manner in which the samples have been steam cured allowed some variation in curing effects, particularly the maturity and the degree of hydration. Since the water sorptivity is sensitive to curing effects, it is likely that the increased degree of hydration at higher temperatures is directly responsible for the observed improvement.

When comparisons are made relative to air-cured controls, it is evident from both mixes that between 30°C and 50°C that sorptivity values approach that of the air-cured controls with increasing temperature. However at temperatures higher than 50°C, a steady improvement in sorptivity values relative to air-cured specimens is noted. This suggests that a preferential drying mechanism is dominant up to 50°C, but at higher temperatures the increase in the hydration rate is sufficient to overcome the adverse effect of drying.

4.6.4 Chloride conductivity

The chloride conductivity test is used to assess the resistance of concrete to ingress by chloride ions and is sensitive to binder types and contents. Chloride conductivity testing was conducted in accordance with test methods and apparatus detailed in *Concrete durability index testing manual* (Alexander, 1999b). The conductivity test results represent the average of at least 4 samples (out of 5 possible samples) from a particular curing regime and mix. Variability analysis was conducted on calculated conductivity values for each mix and curing regime. Tests with coefficients of variation of less than 15% were used to determine conductivity results. Statistical outliers were removed if the coefficient of variation was larger than 15%. The coefficient of variation was typically between 4% and 8%. Test data is presented graphically in figure 4.8.

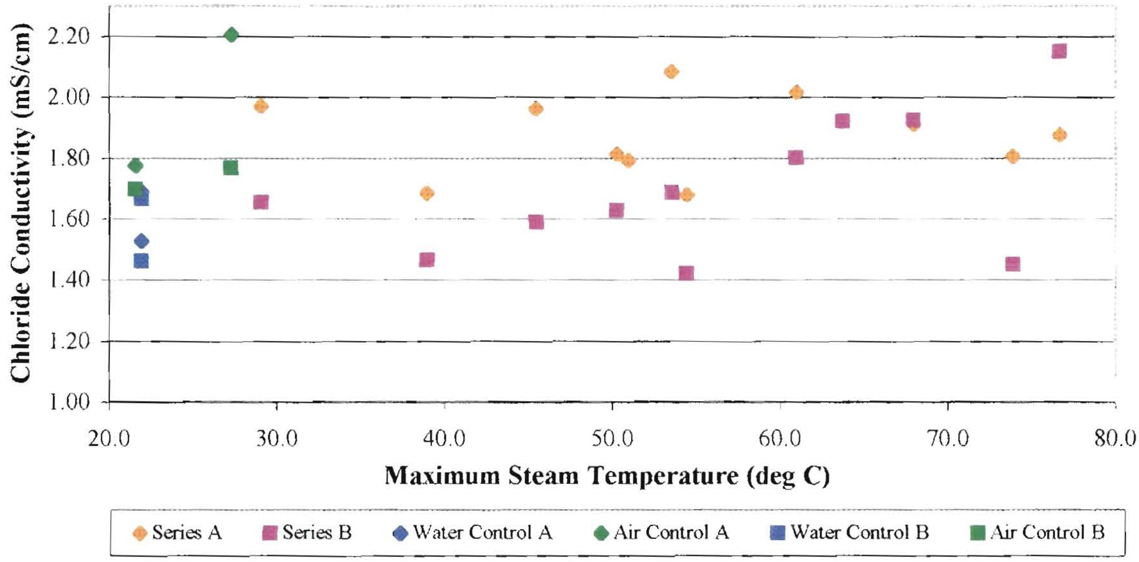


Figure 4.8: Relationship between normalised (40 MPa) chloride conductivity values and maximum steam curing temperature.

No definitive trend could be established for the chloride conductivity test data. Steam cured series A samples generally had conductivity test results greater than the water control. Steam cured series B samples had similar conductivity values as the water control up to 55°C. Thereafter a marked increase in the conductivity is noted for series B samples with increasing steam temperatures.

It was evident that series A had variable results, while series B followed a weak trend. A marginal increase in chloride conductivity values was observed for series B, particularly at higher temperatures (excluding low result at 75°C). The conductivity value is dependent on the chloride binding ability of a concrete. The increased degree of hydration at higher temperatures should result in more sites available for chloride binding, thereby reducing the conductivity value. Conversely researchers have measured an increase in surface area of the HCP in steam-cured samples; hence fewer sites will be available for chloride binding. A corresponding increase in chloride conductivity values should be observed. It is likely that an increase in the porosity of the samples cured at higher temperatures will also account for increases in measured chloride conductivity values.

4.6.5 Porosity

Following the durability index testing, a dependence on the degree of cement hydration and porosity is suspected for steam temperatures above 50°C. Initially a bulk water-filled porosity was calculated from the raw data collected from the sorptivity test. The net mass difference between oven-dried and saturated sample was expressed as a percentage of the approximate volume of the sample. The results are presented graphically in figure 4.9.

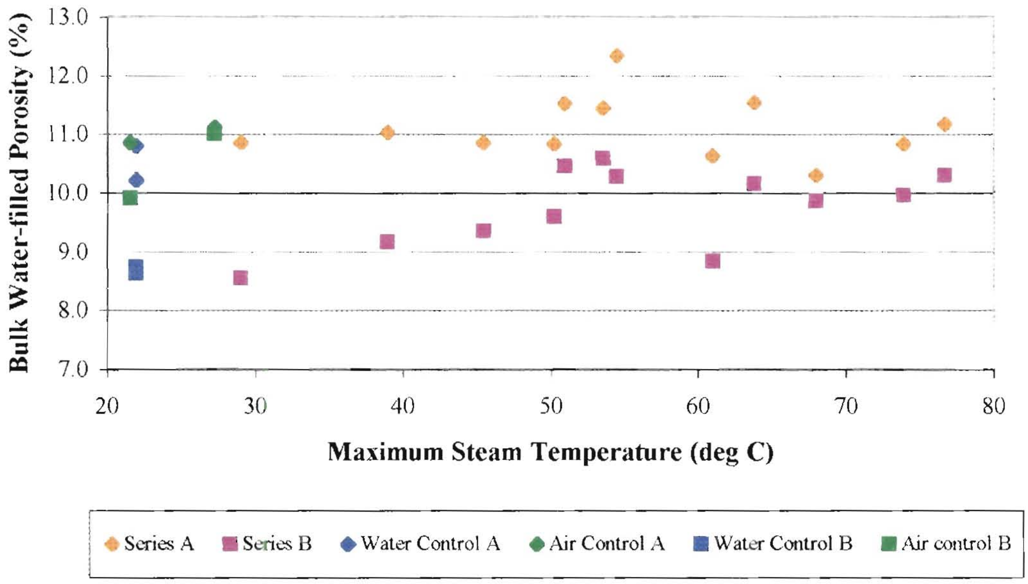


Figure 4.9: Relationship between bulk water-filled porosity and maximum steam temperature.

It is evident from figure 4.9 that an increase in water-filled porosity was observed with increases in temperature up to 55°C, especially for series B. However at higher temperatures, the water-filled porosity marginally decreased with increasing temperature. This evidence suggests that two interdependent mechanisms are indeed present and the inversion point occurs at approximately 50°C (for 3 hour steam curing). It is likely that the initial increase in porosity is due to preferential drying and corresponding lack of adequate wet curing, while the subsequent decrease in porosity occurs probably due to the effects of increased degree of hydration.

However further testing was needed to attempt to verify the effects of increase in the degree of hydration. Electron microscope techniques were favoured as comparisons of the degree of hydration made through visual observation and the microstructural porosity could be quantified. Indications of microstructural changes could also be inferred. Kjellson (1990) observed increases in the micro porosity with increasing hydration temperature and the presence of hydration shells for specimens hydrated to 70%. The sample curing procedure used in section 4.6 did not allow control over the degree of hydration. The durability index test and water-filled porosity are bulk measurements of the properties of the concrete sample. However electron microscope techniques represent a measurement specific to the diminutive test area. This limitation may be partially overcome by making a large number of determinations. Due to the limited samples available, time constraints and the confirmatory nature of the study, only one area was selected for electron imaging.

Testing was conducted on six samples taken from series A representing five different steam curing temperatures and one air-cured control sample. The discarded end slices of the core were left on the air-curing rack until the preparation of the samples. Small sections of mortar were removed, and vacuum dried for 24 hours. The samples were then embedded in Spur's resin under vacuum for 8 hours. The resin was cured for 14 hours at 60°C. The resin embedded samples were incrementally polished down to 0.3 μm using a diamond paste. A highly polished surface was necessary to remove noise from the images and is consistent with the practice used by Kjellson. The samples were then coated with carbon.

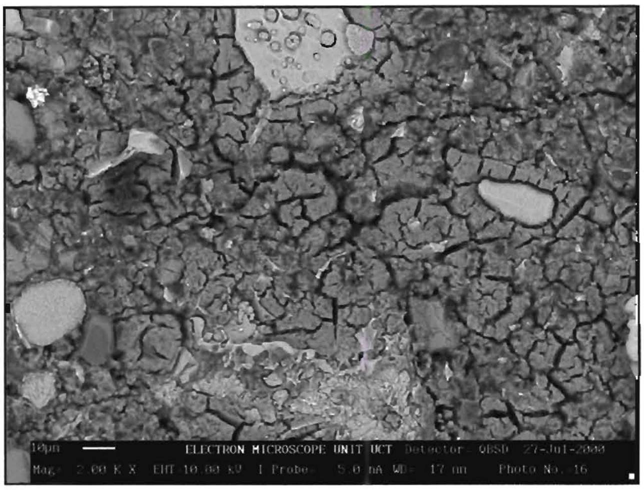
The microscopic study was conducted at the Electron Microscope Unit of the University of Cape Town. The instrument used was a Leo S440 analytical scanning electron microscope equipped with a Lab 6 filament. A solid state four quadrant backscattered detector (KE developments) and Leica specimen current amplifier were used to produce the relevant sets of images. Backscattered electron images, consistent with the method as used by Kjellson (1990), were used in an attempt to identify differences in the hydration products. Backscattered electron images are most often used to provide information relating to the mean atomic number (and thereby density) of the regions being studied. Specimen current images were produced for the identical region with precisely the same microscope parameters. The specimen current images are images produced representing the electrical conductivity of the sample or the amount of charge being leaked to earth and therefore adsorbed by the sample. Although specimen current images (SC) lack the atomic number contrast of backscattered electron images (BS), the porosity is likely to be more clearly defined. Image analysis to determine porosity fractions may then be simpler.

The microscope parameters for both image sets were held constant as follows:

- accelerating voltage = 10 kV
- beam current = 70 μA
- probe current = 5.0 nA
- brightness = 48.0 % (BS) or 38.3 % (SC)
- contrast = 77.0 % (BS) or 71.1 % (SC)
- gain = 38.7 % (BS) or 47.2 % (SC)

The backscattered image sets for the six samples are presented in figure 4.10 and the specimen current image of the identical region is presented in figure 4.11.

Following this the porosity fraction of the exposed area of the sample was calculated by digitally processing the image. Computer image analysis software (Visilog 5.0 by Noesis, 1996) was used to quantify the porosity fraction of the image area. A suitable greyscale interval (0 to 96) was chosen that was most representative of the porosity fraction of the all the backscattered images. The same greyscale interval was used for the specimen current images. The image was binarised and the one pixel noise was removed through erosion and dilation graphic manipulation processes. The number of pixels representing the chosen porosity interval was the expressed as a percentage of the total pixel count to represent the image porosity. These values were then compared with the maximum steam curing temperature and represented graphically in figure 4.12.



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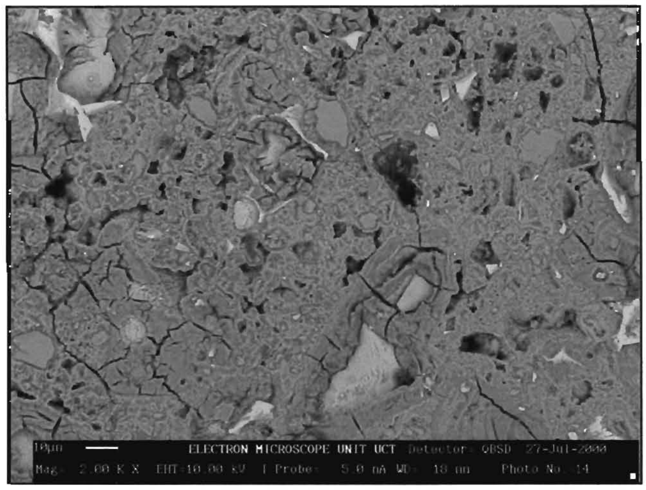

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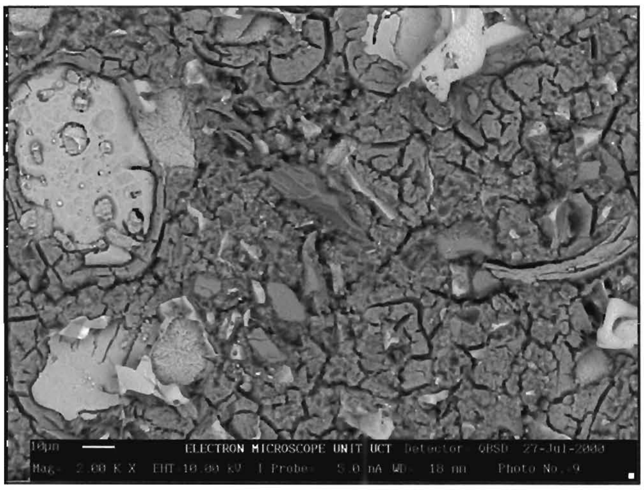

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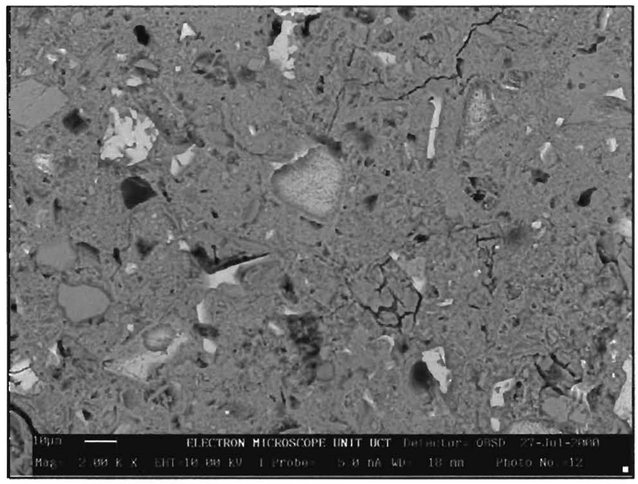

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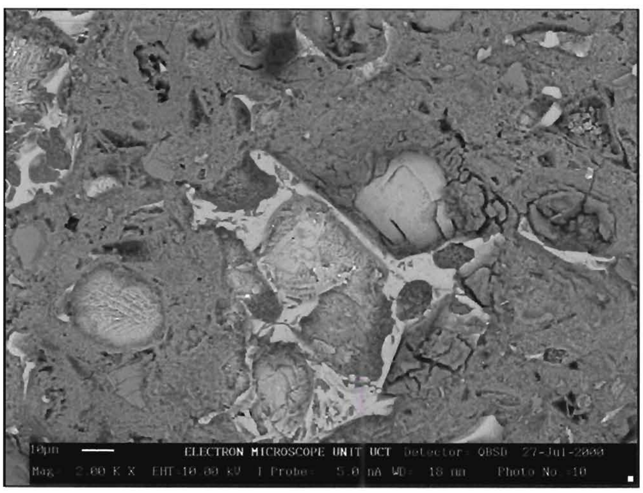
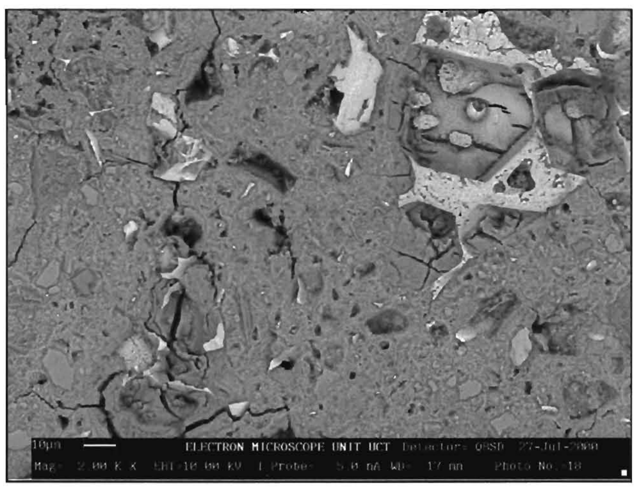
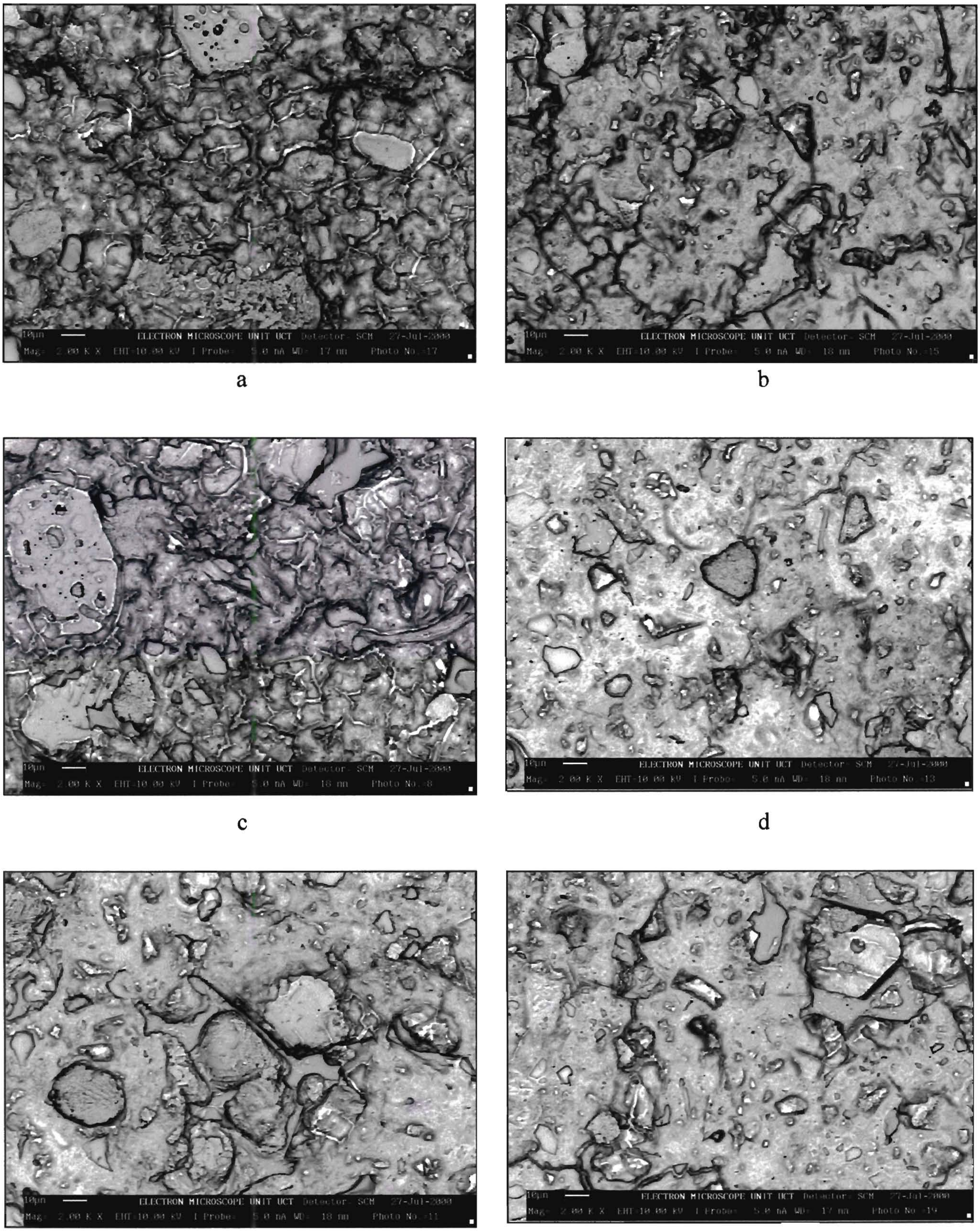


e



f

Figure 4.10: Backscattered electron images of series A concrete samples steam cured at (a) 21.6°C (air-cured control), (b) 29.1°C, (c) 50.3°C, (d) 61.0°C, (e) 68.0°C and (f) 76.7°C.



a b c d e f

Figure 4.11: Specimen current images of series A concrete samples steam cured at (a) 21.6°C (air-cured control), (b) 29.1°C, (c) 50.3°C, (d) 61.0°C, (e) 68.0°C and (f) 76.7°C.

It is evident from the backscattered images that the CSH becomes more uniformly grey and take a smoother appearance with increasing steam temperature. The specimen current images show that the same CSH becomes more intensely white with increasing steam temperature. The effects observed in the backscattered images and the increase in the specimen current intensity, or the whiteness of the image, can be attributed to increases in the degree of hydration with increasing temperature. Although the specimen current images are not available on most microscopes, they seem particularly suited for observing variations in the degree of hydration.

The presence of hydration shells could not be confirmed for this investigation. This is due a relatively low degree of hydration being achieved, even for samples cured at high temperatures. Kjellson only observed the hydration shell phenomena at advanced degrees of hydration (70%). Since the samples were cured at elevated temperatures for only three hours, the degree of hydration is expected to be significantly lower. Data presented by Idorn (1968) shows large variations in the degree of hydration of C_3S after 3 hours: 54% at 70°C, 30% at 50°C and only 10% at 20°C. However 40% hydration is expected for C_3S hydrated for 24 hours at 20°C.

Voids filled with needle-shaped ettringite are present in all backscattered images, but the relative size of these objects increases with increasing steam temperature. The specimen current images show increased zones of porosity around the edges of these objects. The cracking appearance of samples cured at 21,6 °C and 50,3°C are probably due to the extended drying period of the samples (approximately five months). The other samples were dried on the air-curing rack for only two months after slicing and coring. Observations of porosity decrease with increasing steam temperatures for both sets of images. The results of digitally analysed porosity for both data sets are shown graphically in figure 4.12.

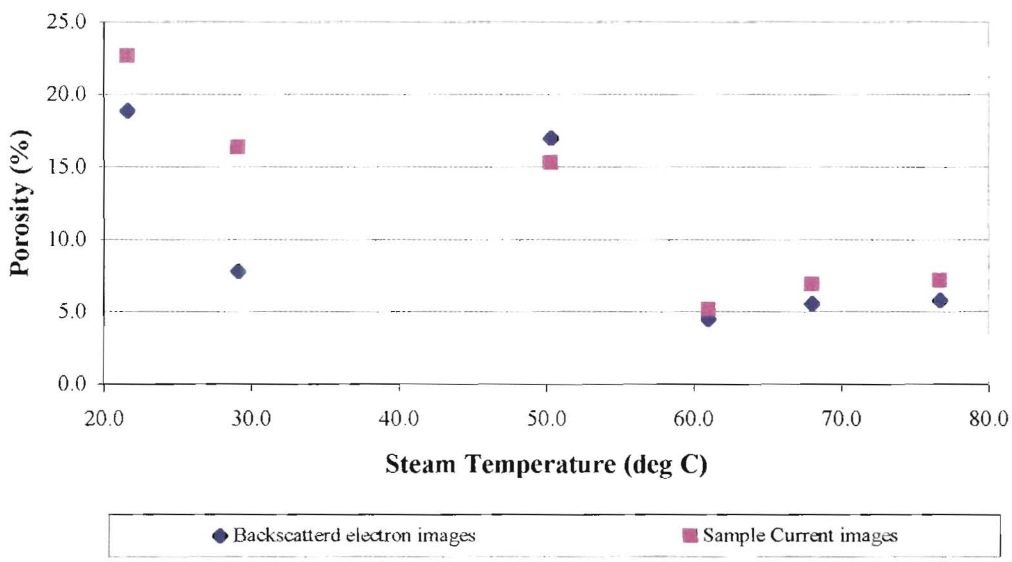


Figure 4.12: Porosity of series A concrete samples determined from digital image analysis using two independent electron microscope techniques.

The first observation is that both microscopic methods yield similar porosity fractions. The use of the specimen current image is recommended for the ease at which the porosity fraction can be determined and for the technical ability to identify porous rims around hydrating cement grains.

Secondly, the porosity fraction decreases with increases in steam temperature up to 60°C. However the porosity increases at temperatures in excess of 60°C, but remain somewhat low. These contradict those obtained for bulk water-filled porosity. This may be due to the fact that a very small region is examined using electron microscope techniques. It is more likely that the degree of hydration strongly influences the micro porosity measured using microscopic techniques. The advancement of the degree of hydration is likely to result in the production of more hydration products, thereby reducing the micro porosity. The apparent contradiction of the porosity trends established for the water-filled and microscopic techniques suggests that a shift in the average size of the pores has taken place.

4.6.6 Holistic discussion of results

The decrease in OPI values and corresponding increases in permeability are consistent with the observations made by Goto and Roy (1981) and Higginson (1961), namely that increases in permeability are measured for concretes cured at elevated temperatures. The water sorptivity and water-filled porosity data show that both the rate and volume of calcium hydroxide-saturated water absorbed generally increases for concretes cured at elevated temperatures. This data represents contrasting trends to observations made by Sherman *et al* (1996b). The increase in chloride conductivity values for concretes cured at elevated temperatures is consistent with the trend established for AASTHO T277 results inferred from data generated by Detwiler *et al* (1994) and Sherman *et al* (1996b). It is likely that heat cured concrete will have higher chloride ion diffusion coefficients and hence more susceptible to attack by aggressive ion species. The occurrence of reduced chloride ion concentrations at the exposed surface as determined by Sherman *et al* (1996b) is typical of concretes with poor ability to effectively bind chloride ions. Similar observations have been made for cements with low C_3A contents and poorly cured Portland cement concretes (Mackechnie, 1997).

The increases in permeability with increasing steam temperatures are indicative that a more open pore structure with increased connectivity, reduced constrictivity and reduced tortuosity is formed. The increases in water sorptivity values, chloride conductivity and bulk water-filled porosity with increasing curing temperature support this idea. The observed reduction in the durability indexes with increases in the steam temperature correlates with increases in the volume of macro pores. However the micro porosity measured using electron microscope techniques reduces with increasing curing temperature. Although the advancement of the degree of hydration does influence the durability indexes and the porosity measurements, the data set conforms to the model proposed by Radjy *et al* (1973). An increase in macro pore volumes, observed through the effects on the bulk water-filled porosity, sorptivity and conductivity values, occurs at the expense of micro pore volume, measured using electron microscope techniques, at higher curing temperatures.

4.7 SUMMARY

The current state of knowledge suggests that a shift from through-solution to topochemical cement hydration mechanism occurs earlier at elevated curing temperatures. At more advanced degrees of hydration, microstructural changes occur in concrete samples cured at elevated temperatures. Dense hydration shells, as well as increased heterogeneity of the paste, are increasingly observed at higher curing temperatures for more mature samples. Coarser CSH particles with increasing porosity were measured for samples cured at higher temperatures. These microstructural changes have adverse affects on mechanical properties. Relative reductions of the long-term strength are an acceptable consequence of high early age strength. However significant microstructural, and possibly physical, changes are likely to have adverse effects on durability. Durability index testing shows that curing at elevated temperatures have an adverse affect on concrete durability. A second order polynomial relationship, with a correlation coefficient of 0,869, was established between OPI values and maximum steam curing temperatures. This suggests that a more permeable microstructure is developed at elevated temperatures. Increases in water sorptivity, chloride conductivity and bulk water-filled porosity suggests that concretes developed larger volumes of macro pores at elevated temperatures. Micro porosity, measured using two electron microscope techniques, decreases for increasing curing temperatures. These findings support the idea that larger pores are formed at the expense of smaller pores for concretes cured at elevated temperatures. Confirmatory studies demonstrated that the durability indexes are sensitive to the bulk macro porosity and the degree of hydration.

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Chapter 5:

Conclusions and Recommendations

Precast box culverts are structurally significant units, which have historically been allowed large reductions in cover to reinforcing steel in comparison to cast-in-place box culverts. The reduction of cover is based on the perception that the precast units are of superior quality. The increased quality stems from increased control over the materials used, the mixing procedures, reinforcing steel arrangement and placement, and especially control over the concrete casting, compacting and curing operations that should be achieved in a precast factory. However the reduction in cover to 20 mm (from 40 mm for cast-in-place culverts), suggests that precast concrete should have as much as four times higher quality than cast-in-place concrete. Despite the anticipated high levels of concrete quality, the current specification (SABS 986-1994) does not provide adequate guidance relating to concrete quality. The current specification is of the "recipe" type and requires concrete with a minimum binder content of 350 kg/m^3 and minimum 28-day cube strength of 40 MPa. However, these requirements are not sufficient to ensure the durability of box culvert units.

The durability performance of a structure in a particular environment is a function of the thickness of cover and the quality of the concrete in the cover region. Thus the quality of the concrete and the actual cover achieved are critical factors in ensuring the durability performance of precast box culverts units. The current specification does not provide adequate guidance to control the concrete quality or the actual cover achieved.

The manufacturers construction process affects the quality of the concrete in the actual unit. Compaction and curing procedures have the most significant effect on final concrete quality.

5.1 CONCLUSIONS

The durability performance of precast box culverts was quantified and was found to be dependant on the aggressiveness of the environment and the manufacturing quality of the units. Both visual and analytical evidence has been presented in support of the observed lack of durability. The minimum cover, critical to the durability performance of the precast culverts, has frequently not been achieved. A nominal reduction in cover with precast concrete is justified by showing that the quality of precast concrete is superior to cast-in-place concrete.

The steam curing procedures currently used in box culvert construction was found to be severe and often exceeded the recommended limits. The concrete used in box culvert construction had good to excellent durability characteristics. The required cover to steel is not currently being achieved.

A review of the effect of elevated temperature curing on the cement hydration mechanism, microstructural development and mechanical properties of concrete was undertaken. The results from a laboratory study show that the maximum steam curing temperature had a significant effect on concrete durability. Generally, the concrete durability was adversely affected by increases in the maximum steam curing temperature.

5.1.1 The in-service performance of box culverts

The in-service performance of precast box culverts was found to be variable. A large proportion, 38%, of the precast culvert units surveyed exhibited some degree of deterioration. Incidence of deterioration increased with the aggressiveness of the environment. The durability of units in mild chloride-and sulphate-bearing environments was poor. Softwater attack contributed a significant portion of the deterioration. Carbonation-induced corrosion was an important deterioration mechanism due to frequent exposure to wetting and drying cycles. The occurrence of alkali-aggregate reaction was also observed.

A lack of adequate manufacturing quality was established as a large number of reinforcing bars were measured at covers less than 20 mm. Large variations in the covers achieved for individual units were also observed. Carbonation rates were mild, but increased with a corresponding decrease in the concrete quality.

A reduction in cover to reinforcing for precast concrete is justified given the superior quality of the precast concrete in relation to cast-in place concrete. Yet covers as low as 20 mm seem extreme in relation to other national and international building codes and the potential risk of failure of the structure. Before recommendations linking concrete quality to design covers could be made, the current manufacturing quality was established.

5.1.2 The current manufacturing quality of precast box culvert units

The steam curing procedure used to cure precast box culverts in the Western Cape was found to be severe. Recommended and durability limits are often exceeded, and premature drying, as well as cracking, were observed due to the use of harsh steam curing practices.

The concrete used in precast box culvert construction is typically ordinary structural concrete having good to excellent durability characteristics. The use of fly ash (30% replacement) considerably improves the durability of the concrete. The concrete quality is shown to decrease in the actual units, probably as a result of less effective compaction, but the influence of accelerated curing procedures cannot be excluded.

The lack of adequate manufacturing quality, particularly the lack of achievement of the required cover, remains prevalent in the industry. Since the achievement of the required minimum cover for a specific concrete quality is a critical durability factor, recommendations are suggested to improve reinforcing steel arrangements. Finally, a rational basis for the reduction of cover to steel, in terms of the quality of the concrete, is proposed.

5.1.3 The effects of elevated temperature curing on concrete

A review of the available literature suggests that a shift from through-solution to topochemical cement hydration mechanism occurs earlier at elevated temperatures. At more advanced degrees of hydration, microstructural changes become apparent as a result of the altered hydration mechanism. Dense hydration shells, as well as increased heterogeneity of the paste, are increasingly observed at higher curing temperatures for more mature samples. Coarser cement hydration products and corresponding increases in porosity have been measured for samples cured at elevated temperatures. The microstructural changes have been used to explain the adverse affects of elevated temperature curing on the mechanical properties of concretes. More importantly, significant microstructural and physical changes have adverse affects on durability.

Durability index testing shows that curing at elevated temperatures does have an adverse effect on concrete durability. A good statistical relationship was established between oxygen permeability measurements and the maximum steam temperature, suggesting that a concrete of increasing permeability is formed at increasing temperatures. Measured increases in the water sorptivity, chloride conductivity and bulk water-filled porosity suggests that concretes developed larger volumes of macro pores at elevated temperatures. But micro porosity, measured using two electron microscope techniques, decreases for increasing curing temperatures. Thus supporting an established suggestion that larger pores are formed at the expense of smaller pores for concretes cured at elevated temperatures. By inference, the durability indices are sensitive to the bulk macro porosity and the degree of hydration.

5.2 RECOMMENDATIONS

Recommendations are made with the aim of further controlling the manufacturing quality of precast concrete box culverts and thereby improving the in-service performance. Suggestions relating to the current SABS 986-1994 specification and improvements to current precast practice are presented separately. It is important to note that any changes in the construction process need to be incorporated with some degree of flexibility. This will allow the individual precast manufacturer to develop unique solutions to the proposed changes and the competitive nature of the industry can thereby be maintained.

5.2.1 Suggestions for inclusion in the SABS 986 specification

The current SABS 986-1994 specification does not provide adequate guidance relating to manufacturing quality issues. The specification, and thereby the quality of the final product, can be significantly improved by providing adequate comment on steam curing procedures, ensuring the achievement of the required minimum cover to steel, and suggesting levels of concrete quality.

Steam curing procedures as presented in subsection 3.1.2 should be included in the SABS 986 specification. The maximum concrete temperature should be limited to 65°C. A suitable delay period that facilitates the required 8 hour mould turnover period should be recommended. Further testing relating the concrete durability to delay period should be conducted for this purpose.

Concrete materials exhibiting excellent durability are of little use unless the required minimum cover to reinforcing steel is consistently achieved. A suitable method of determining the cover should be clearly defined in the SABS 986 specification. The method used in section 3.4 represents the minimum number and relevant positions that should be considered. Additional positions for measuring covers could be included, but the excessive measurements will be time consuming and should be avoided. The sampling rate, possibly 1 unit from every 10 units produced, needs to be established and records of the testing presented for SABS certification purposes.

The precast manufacturers currently make use of a product testing procedure to ensure the structural performance of the box culvert units. It is recommended that performance based testing be introduced to ensure the quality of the concrete used. A convenient and rational way of allowing reductions of cover to steel is through the implementation of a set of performance levels required for various cover depths (as shown in table 3.10). However nationwide testing of all aggregate and binder types is required to prove the relevance of the suggested table. In addition the reduction in performance levels applicable to concrete in the final product remains to be confirmed.

5.2.2 Suggested improvements to the manufacturing process

Attention should be given to the adequate placement of cover blocks. In addition, more care should be exercised in concrete casting operations. This should help mitigate some of the current problems concerning the achievement of cover to steel.

Improvements to general concrete practice can be made by adequately covering products during heat treatment. The premature drying and cracking observed in some units must be prevented.

The most significant improvement of the manufacturing process can be made by amending current reinforcing steel arrangements to ensure the achievement of minimum cover to steel. The covers for a representative sample of units needs to determined and the degree of control over steel placement assessed. The required nominal cover (evaluated from table 3.5) should be used to amend the reinforcing steel layout, thereby ensuring that the minimum cover is achieved.

Finally, it is recommended that most manufacturers increase the cover to 30 mm and use concrete mixes that are similar to those currently produced in the Western Cape. Should a minimum design cover of 20 mm be used, at least a grade 65 concrete mix containing 30% fly ash will be required. This mix is likely to prove to be uneconomical in the Western Cape and particular care needs to be taken to ensure consistently high levels of quality.

Physical Site Information

Appendix A1

Ref No.	Total units	Survey date	Construction date	Environment 1 vs	1 s	1 m	2	3	Manufacturer	Size (spxht)	Notes	SV	Route Description
1	40	08-Feb-00	25-Oct-79					3	Rocla	3600x1800		33.0	R45: R46 junction to Paarl
2	32	08-Feb-00	15-Jun-78					3	Rocla	1800x1200		35.1	R45: R46 junction to Paarl
3	15	08-Feb-00	28-Feb-80					3	Rocla	2100x1200	Special	36.2	R45: R46 junction to Paarl
4	15	09-Feb-00	12-Jun-95				2		Fyfe	1500x1200		10.5	R45: R44 junction to Franshoek
5	14	09-Feb-00	21-Aug-95				2		Fyfe	2000x2000		11.5	R45: R44 junction to Franshoek
6	16	09-Feb-00	12-Jul-91				2		Fyfe	3000x2400		12.0	R45: R44 junction to Franshoek
7	13	09-Feb-00	15-Feb-96				2		Fyfe	2000x2000		12.1	R45: R44 junction to Franshoek
8	15	09-Feb-00	02-Aug-95				2		Fyfe	2400x2100		14.3	R45: R44 junction to Franshoek
9	14	09-Feb-00	01-Aug-95				2		Fyfe	2400x2400		14.6	R45: R44 junction to Franshoek
10	17	09-Feb-00	24-Mar-86				2		Rocla	3600x3000		30.0	R45: Franshoek to Villiersdorp
11	14	15-Mar-00	08-Dec-78				2		Rocla	3000x900		28.2	R43: Villiersdorp to Worcester
12	10	09-Feb-00	22-Oct-78				2		Rocla	2400x2400		34.4	R43: Villiersdorp to Worcester
13	11	09-Feb-00	23-Oct-78				2		Rocla	2400x2400		37.5	R43: Villiersdorp to Worcester
14	26	09-Feb-00	29-May-79					3	Rocla	3600x1800		42.0	R43: Villiersdorp to Worcester
15	28	09-Feb-00	18-May-79					3	Rocla	3600x1800		42.2	R43: Villiersdorp to Worcester
16	12	15-Mar-00	20-Oct-78				2		Rocla	2100x2100	Sulphachem	44.2	R43: Villiersdorp to Worcester
17	13	15-Mar-00	07-Mar-79				2		Rocla	2100x2100	Sulphachem	45.0	R43: Villiersdorp to Worcester
18	15	12-Jul-99	08-Feb-89					3	Concrete Units	2400x1200	SATS 1	11.9	N1: Du Toits Tunnel to Worcester
19	24	12-Jul-99	13-Apr-89					3	Concrete Units	3000x1500	SATS 1	15.9	N1: Du Toits Tunnel to Worcester
20	75	03-Feb-00	02-Mar-89				2		Concrete Units	3000x1500	SATS 1	16.0	N1: Tunnel to Worcester (under bridge B463A)
21	13	12-Jul-99	17-Feb-89				2		Concrete Units	2400x1500	SATS 1	17.6	N1: Du Toits Tunnel to Worcester
22	34	01-Feb-00	14-Mar-78				2		Rocla	3000x2400		24.8	R44: Porterville to Gouda
23	11	01-Feb-00	06-Mar-78				2		Rocla	2100x2100		22.8	R44: Porterville to Gouda
24	13	01-Feb-00	09-Mar-92				2		Rocla	3000x1200		13.4	R44: Porterville to Gouda
25	70	08-Feb-00	24-Apr-91				2		Fyfe	3000x2400		69.9	R44: Paarl to Wellington
26	135	08-Feb-00	25-Jul-91				2		Fyfe	3000x1800		66.0	R44: Paarl to Wellington
27	15	09-Feb-00	26-Jul-78				2		Rocla	1800x1800		46.3	R44: N1 to Stellenbosch
28	41	09-Feb-00	15-Sep-78					3	Rocla	3600x2400		44.3	R44: N1 to Stellenbosch
29	32	09-Feb-00	27-Feb-85					3	Vianini	3600x2400		-	R44: Stellenbosch to Franshoek: US Voedselwet.
30	9	28-Feb-00	25-Feb-86			1c			Vianini	2400x1000		29.2	R44: Pringle Bay to Stony Point
31	13	13-Jul-99	19-Feb-86			1c			Vianini	2400x1000		29.2	R44: Stony Point to Betty's bay
32	13	13-Jul-99	11-Mar-86			1c			Vianini	2400x1000		29.1	R44: Stony Point to Betty's bay
33	15	13-Jul-99	02-Apr-86			1c			Vianini	2400x1000		28.8	R44: Stony Point to Betty's bay
34	22	28-Feb-00	24-Apr-86			1c			Vianini	3000x1000		28.2	R44: Stony Point to Betty's bay
35	9	13-Jul-99	12-Feb-86			1c			Vianini	2400x1500		27.4	R44: Stony Point to Betty's bay
36	14	13-Jul-99	23-Mar-86			1c			Vianini	2400x1500		27.3	R44: Stony Point to Betty's bay
37	42	13-Jul-99	22-Feb-86			1c			Vianini	1200x1200		26.1	R44: Betty's Bay
38	16	28-Feb-00	22-Jul-86			1c			Vianini	3000x1200		25.3	R44: Betty's Bay to Kleinmond
39	13	13-Jul-99	05-Feb-87			1c			Vianini	1200x1200		18.6	R44: Betty's Bay to Kleinmond
40	13	13-Jul-99	20-Mar-86			1c			Vianini	1200x1200		18.5	R44: Betty's Bay to Kleinmond
41	32	13-Jul-99	02-Apr-87			1c			Vianini	1200x1200		18.2	R44: Betty's Bay to Kleinmond
42	22	13-Jul-99	16-Mar-87			1c			Vianini	1200x1200		17.9	R44: Betty's Bay to Kleinmond
43	12	13-Jul-99	02-Apr-87			1c			Vianini	1200x1200		16.8	R44: Betty's Bay to Kleinmond
44	14	15-May-87	28-Feb-00			1c			Vianini	1200x1200		2.7	R44: Kleinmond to Hermanus
45	14	04-Apr-87	28-Feb-00			1c			Vianini	1200x1200		2.7	R44: Kleinmond to Hermanus
46	22	13-Jul-99	26-Mar-87			1c			Vianini	1200x1200		1.7	R44: Kleinmond to Hermanus
47	42	13-Jul-99	18-Jul-79			1c			Rocla	1200x1200		9.0	R43: R44 junction
48	42	03-Feb-00	11-Jul-89					3	Rocla	2400x2400		5.0	R303: Ceres to Wolseley
49	6	03-Feb-00	27-Sep-89					3	Fyfe	1200x1200		5.2	R303: Ceres to Wolseley
50	75	03-Feb-00	21-Nov-89					3	Fyfe	2400x1800		6.2	R303: Ceres to Wolseley
51	2	23-Jul-99	03-May-89			1c			SAR/SAS	2000x1500		6.7	R27: Milnerton to Table View
52	50	29-Feb-00	03-Nov-76			1c			Rocla	1200x1200		13.0	R27: Milnerton to Table View
53	6	15-Mar-00	03-Nov-92					3	Rocla	1200x1200		17.0	R102: Kuilsriver to Eersteriver
54	12	15-Mar-00	21-Sep-92					3	Rocla	2100x900		18.6	R102: Kuilsriver to Eersteriver
	1313												
Insitu:	Most culverts are very old, and made with timber shuttering, these have been ignored as they do not reflect modern insitu construction techniques. See N7, N1, N15, R27, R43 Worcester to Ceres												
I1		12-Jul-00						3	Timber panels	-		68.0	N1: Past tunnel out of Cape Town
I2		17-May-00	S 1664					3	Steel Shuttered	4000x4500		18.0	N1: Cattle Creep btw wine farms
I3		16-May-00						3	Steel Shuttered extension	1800x1800		19.5	N7: Melkbos turnoff to Malmesbury
I4		15-May-00				1c			Steel Shuttered	3000X1500		18.5	R44: Betty's Bay to Kleinmond

Site Deterioration

Appendix A2

Ref No.	# of Units	Abrasion	Stains	Cracks & major direction	Spalling, # of units	Likely Cause	AAR
Precast:							
1	2			1.5mm Transverse			Possible
5	14	50mm				Softwater	
7	4	Holes				Poor Finishing	
10	17	1000mm				Softwater	Possible
11	4			Map, Longitudinal, Transve	Yes, 1		Yes
12	3		Corrosion	1.5mm Longitudinal	Yes, 3	Aggres GW	
13	3		Corrosion	6.7mm Longitudinal			
14	26	700mm		0.6mm Transverse		Softwater	Possible
15	1	500mm		0.3mm Longitudinal, Transve		Softwater	Possible
16	2		Corrosion		Yes, 2		
17	3		Corrosion	2.0mm Longitudinal	Yes, 6		
18	1		Corrosion			Carbonation	
18	2					Poor Finishing	
19	3					Poor Finishing	
20	75	500mm				Softwater	
21	13	1000mm	Humic Deposit			Aggres GW	
22	19	200mm	Corrosion	2.5mm Longitudinal	Yes, 6	Carbonation	
23	6		Corrosion	2.0mm Longitudinal		Carbonation	
26	3			2.5mm Transverse		Aggres GW	
27	1	Corner mis	Corrosion			Installation Err	
28	1		Corrosion			Low cover, 6mm	
29	5	120mm	efflorescence		Yes, widespread	Sulphate attack	
30	9	10mm				Softwater	
31	13	50mm				Softwater	
32	13	50mm				Softwater	
33	15	200mm	Corrosion		Yes, 1	Carbonation	
35	9	150mm	Corrosion	0.6mm Transverse		Low cover, 0mm	
36	14	500mm				Softwater	
37	42	20mm				Softwater	
38	13	30mm				Softwater	
39	13	50mm				Softwater	
41	32	110mm				Poor Finishing	
42	22	30mm				Softwater	
43	1		Corrosion			Carbonation	
44	14	15mm				Softwater	
46	1		Corrosion			Carbonation	
47	3		Corrosion		Yes, 3	Carbonation	
48	42	40mm				Softwater	
51	2		Corrosion	6.0mm		Aggres GW	
52	20		Corrosion	3.1mm Longitudinal	Yes, 10	Chlorides	
52	12			0.15mm Transverse		Thermal	Possible
	498						
Insitu:							
12	Insitu	Honeycombing				Poor Manufact	

Site Manufacturing Quality

Appendix A3

Ref No.	Carbonation Model		Distribution of reinforcing steel				Distribution of reinforcing steel			
	Depth (mm)	Coefficient (mm/year^0.4)	Mean mm	Std Dev mm	Co-var %	# bars < 20 mm	Mean mm	Std Dev mm	Co-var %	# bars < 20 mm
Precast:										
1	8.8	2.6	31.1	21.0	67.5	7	44.3	14.0	31.7	0
2	9.4	2.7	31.6	8.0	25.3	1	23.9	4.7	19.7	3
3	9.3	2.8	20.1	2.4	11.8	5	25.6	4.4	17.3	0
4	3.8	2.0	20.4	7.6	37.0	6	21.2	7.5	35.1	6
5	4.2	2.2	27.0	7.5	27.6	1	22.3	2.7	12.2	2
6	7.0	2.9	42.3	13.7	32.4	0	45.9	7.3	16.0	0
7	7.2	4.1	21.7	8.7	40.3	9	23.0	6.2	27.0	8
8	2.3	1.2	24.0	7.0	29.2	8	22.3	4.7	21.2	8
9	5.0	2.6	27.6	8.2	29.8	0	31.9	8.5	26.6	0
10	3.9	1.4	19.5	4.5	22.8	9	25.4	7.1	28.0	5
11	14.7	4.3	29.3	12.5	42.6	0	27.8	11.0	39.5	0
12			40.4	6.8	16.9	0	41.2	12.1	29.4	0
13	4.5	1.3	25.3	2.5	9.9	0	25.8	2.4	9.4	0
14	8.3	2.5	32.8	6.5	19.9	0	33.0	7.7	23.4	0
15	8.0	2.4	25.5	5.0	19.4	3	26.0	8.6	32.9	5
16	6.8	2.0	26.5	4.1	15.6	0	29.2	4.2	14.4	0
17	9.7	2.9	25.0	2.2	9.0	0	Water too deep and dirty!			-
18	18.8	7.5	31.5	4.2	13.3	0	33.6	4.7	14.1	0
19	9.4	3.7	28.6	3.3	11.4	0	26.8	2.3	8.7	0
20	18.4	7.1	32.2	13.2	40.9	0	40.6	6.2	15.2	0
21	5.5	2.2	32.5	2.9	8.9	0	30.9	4.2	13.6	0
22	18.7	5.4	25.7	7.4	28.7	3	26.4	7.6	28.7	2
23	5.8	1.7	25.4	8.6	34.0	6	33.0	5.6	17.1	1
24	3.6	1.6	21.7	4.2	19.3	1	28.2	4.6	16.3	0
25	5.2	2.2	31.3	11.6	37.1	0	33.4	13.8	41.4	2
26	5.1	2.1	35.7	3.5	9.9	0	31.9	12.2	38.1	0
27	4.6	1.3	22.7	2.6	11.4	2	20.6	2.0	9.5	9
28	3.9	1.1	33.0	12.2	36.8	1	31.9	5.0	15.6	0
29	8.4	2.8	51.7	8.7	16.7	0	65.1	19.3	29.6	0
30	8.7	3.0	28.5	7.1	24.7	0	29.9	16.4	54.8	5
31	6.6	2.4	26.8	7.7	28.6	1	31.6	4.2	13.2	0
32	4.7	1.7	26.5	3.5	13.0	0	28.0	2.7	9.8	0
33	20.4	7.3	28.1	3.9	13.9	0	28.8	6.5	22.5	0
34	5.9	2.1	29.0	5.5	19.0	0	30.6	9.5	31.2	0
35	10.6	3.8	25.6	2.0	7.9	0	17.9	12.5	70.0	6
36	6.5	2.3	31.3	8.2	26.2	1	24.3	3.7	15.3	1
37	7.3	2.6	23.9	5.7	24.0	2	21.9	2.6	11.8	2
38	8.0	2.8	28.9	10.1	35.0	0	28.6	6.6	23.0	0
39	8.4	3.1	18.1	3.4	19.0	4	21.3	10.4	49.1	4
40	8.2	2.9	23.6	6.0	25.5	2	16.8	3.9	23.0	6
41	13.1	4.8	20.4	4.1	20.3	4	16.4	11.1	67.6	5
42	6.0	2.2	18.6	4.8	25.8	3	23.4	11.6	49.6	0
43	16.3	6.0	15.8	5.5	35.0	5	21.6	3.5	16.0	2
44	6.9	2.5	27.0	1.6	5.7	0	26.7	5.0	18.6	0
45	6.1	2.2	26.1	5.2	20.1	2	25.5	3.4	13.3	2
46	16.8	6.2	16.9	2.6	15.3	7	21.1	2.8	13.0	1
47	18.0	5.4	23.6	1.7	7.0	0	23.2	1.4	6.2	0
48	7.0	2.7	18.4	7.3	39.6	6	26.2	4.5	17.3	0
49	7.6	2.9	57.0	3.7	6.4	0	40.5	13.0	32.1	0
50	9.3	3.6	51.4	7.7	15.0	0	55.0	13.8	25.0	0
51	10.7	4.3	27.7	5.3	19.2	2	26.1	5.4	20.6	3
52	5.9	1.7	21.6	4.6	21.1	6	24.7	4.6	18.6	2
53	4.4	1.9	20.4	1.7	8.4	9	20.2	3.1	15.2	11
54	5.7	2.5	27.8	4.6	16.6	0	27.1	3.9	14.2	0
	Average K =	**3.0**								
Insitu:										
I1	4.4		59.8	7.7	12.9	-	54.6	14.1	25.9	-
I2	30.0		61.7	11.4	18.5	-	-	-	-	-
I3	13.7 (old=20.5)		32.0	5.1	15.8	-	41.4	2.4	5.7	-
I4	3.5		40.3	7.0	17.4	-	-	-	-	-

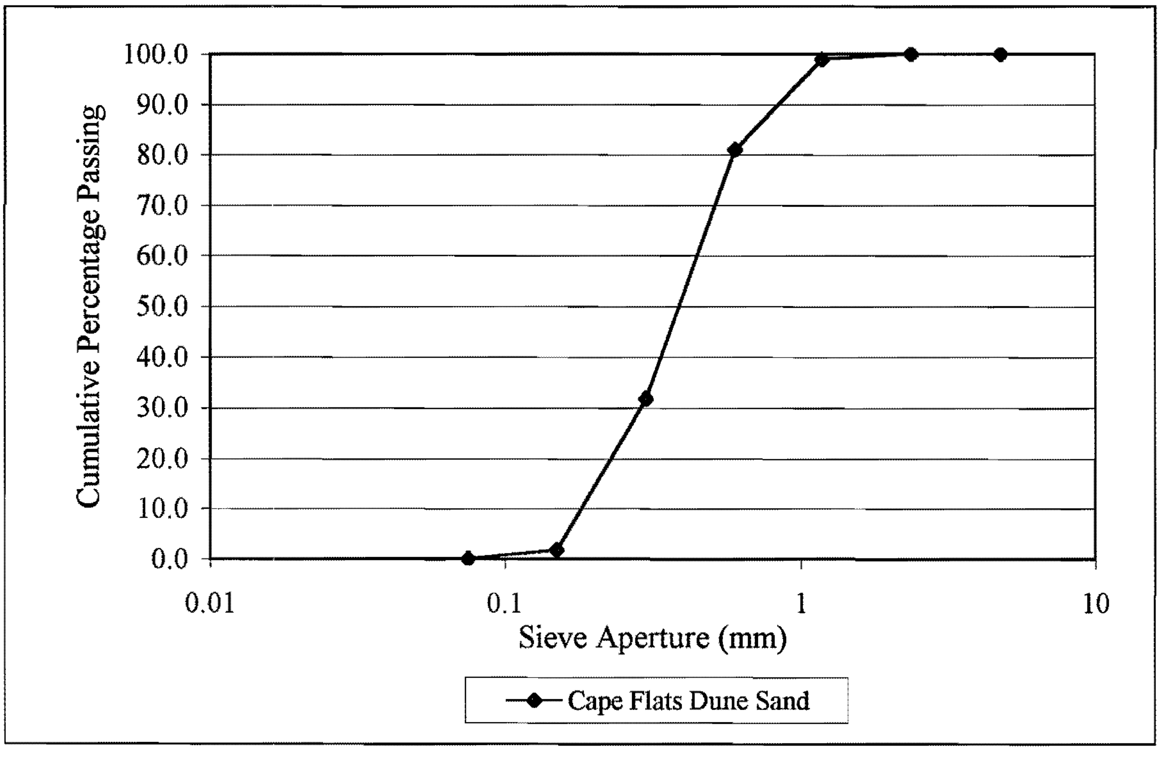
Material Properties (for Section 4.6)

Appendix B1

XRF analysis of CEM I 42,5:

Chemical Oxide	**Proportion of sample (%)**
CaO	64.72
SiO_2	20.56
Al_2O_3	3.46
Fe_2O_3	3.78
Mn_2O_3	0.06
TiO_2	0.18
MgO	1.13
P_2O_5	0.00
Cl	0.00
SO_3	2.18
K_2O	0.64
Na_2O	0.33
Na_2O eq	0.76

Grading analysis of Cape Flats Dune sand:



Steam Curing Project Data

Appendix B2

Concrete Mix	Max Steam Temp (deg C)	fcu orig. (MPa)	OPI orig. (log scale)	N-OPI (40 MPa)	Sorptivity orig. (mm/hr^0.5)	N-Sorp (40 MPa)	Cl Conductivity orig. (mS/cm)	N-Cond (40 MPa)	Porosity orig (% by vol.)
Water control A.1	22.0	41.0	9.58	9.34	8.3	8.5	1.49	1.53	10.8
A.1	50.3	41.0	9.08	8.85	13.2	13.5	1.77	1.82	10.9
A.2	51.0	40.1	8.86	8.85	12.3	12.3	1.79	1.79	11.5
A.3	54.5	38.4	8.92	9.28	12.4	11.9	1.75	1.68	12.4
A.4	73.9	40.2	8.83	8.79	12.0	12.1	1.80	1.81	10.9
A.5	45.5	39.5	8.75	8.86	13.1	12.9	1.99	1.96	10.9
A.6	68.0	47.2	9.32	7.89	9.0	10.6	1.62	1.91	10.3
Air control A.7	27.3	52.9	9.35	7.08	10.1	13.3	1.67	2.21	11.1
A.8	63.8	51.1	9.64	7.55	7.6	9.8	1.96	2.50	11.6
A.9	61.0	49.2	9.45	7.68	9.4	11.5	1.64	2.02	10.6
Water control A.10	22.0	39.1	9.60	9.83	9.6	9.3	1.73	1.69	10.2
A.10	29.1	39.1	9.30	9.52	11.1	10.9	2.02	1.97	10.9
A.11	76.7	38.9	9.28	9.54	10.1	9.9	1.93	1.88	11.2
A.12	53.6	43.2	9.28	8.58	11.7	12.6	1.93	2.09	11.5
A.13	39.0	38.1	8.82	9.26	13.1	12.5	1.77	1.69	11.0
Air control A.14	21.6	37.8	8.95	9.47	16.1	15.2	1.88	1.78	10.9
Water control B.1	22.0	44.3	9.91	8.95	6.9	7.6	1.32	1.46	8.6
B.1	50.3	44.3	9.47	8.55	8.7	9.6	1.47	1.63	9.6
B.2	51.0	29.2	8.43	11.56	7.5	5.4	1.98	1.44	10.5
B.3	54.5	31.1	8.73	11.24	9.0	7.0	1.83	1.42	10.3
B.4	73.9	33.7	8.45	10.02	6.0	5.1	1.72	1.45	10.0
B.5	45.5	44.2	9.61	8.70	8.4	9.3	1.44	1.59	9.4
B.6	68.0	49.4	9.24	7.48	7.4	9.1	1.56	1.93	9.9
Air control B.7	27.3	44.5	9.48	8.53	7.4	8.3	1.59	1.77	11.0
B.8	63.8	44.2	9.30	8.42	6.8	7.5	1.74	1.92	10.2
B.9	61.0	45.9	9.40	8.19	7.7	8.9	1.57	1.80	8.8
Water control B.10	22.0	40.1	9.63	9.60	8.4	8.4	1.66	1.67	8.7
B.10	29.1	40.1	9.37	9.34	10.7	10.7	1.65	1.66	8.6
B.11	76.7	39.0	8.75	8.99	6.5	6.3	2.21	2.15	10.3
B.12	53.6	36.3	9.53	10.51	7.7	7.0	1.86	1.69	10.6
B.13	39.0	36.9	9.32	10.11	10.3	9.5	1.59	1.47	9.2
Air control B.14	21.6	37.8	9.23	9.78	10.0	9.5	1.80	1.70	9.9

Note:

1) Mix B.2 retarded due to overdose of superplasticizer (2%).
2) Mix B.3 and B12 normalized OPI values excluded as OPI values above 10.50 are unrealistic for this mix.
3) fcu variations attributed to variations in cement quality.