School of Civil and Mechanical Engineering

Characterisation of Cement Treated Crushed Rock Basecourse for Western Australian Roads

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DECLARATION

This dissertation contains no material which has been accepted for the award of any other degree or diploma in any university.

To the best of my knowledge and belief, this dissertation contains no material previously published by any other person except where due acknowledgement has been made.

Yeoys

Yang Sheng Yeo

7th October 2011

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ABSTRACT

Cement treatment for pavement basecourse materials results in the creation of cement treated basecourse which is either classified as “modified” or “stabilised”. The two classifications perform differently in service, with susceptibility to fatigue being the most obvious characteristic delineating the two. This classification methodology is currently quantified based on Unconfined Compressive Strength (UCS) ranges.

Throughout the 1990s to the early 2000s, Main Roads Western Australia (MRWA) has investigated various basecourse products created by adding cement. However, due to unexpected behaviour when applying the cement treated basecourse materials, MRWA sanctioned restrictions of its use in Western Australia. “Stabilised” basecourse were not to be used on roads and strength gained from “modified” basecourse is to be ignored. This is quantified by measuring the UCS gained from the cement treatment.

Nevertheless, using UCS to classify cement treated crushed rock is injudicious and does not portray the insitu behaviour and characteristics of cement treated crushed rock basecourse. This dissertation therefore investigates the characteristics of cement treated crushed rock basecourse for Western Australian roads. This is to determine a better method to quantify the various classification of cement treated basecourse and understand its insitu behaviour. Subsequently, the result provides encouragement to reintroduce the use of cement treated crushed rocks in Western Australia.

Amongst the characteristics investigated are strength, fatigue, shrinkage, durability and erodibility. These characteristics are assessed against varying cement content. The dissertation combines known standard testing methods along with uniquely developed testing methods to establish numerical models for characterising the materials. This includes the application of the Four Point Bending Test for fatigue, Tube Suction Test for durability, Nitrogen Adsoprtion for shrinkage, and Wheel Tracking Test for erodibility. A finite element model is also created to validate the results for fatigue.

This dissertation has improved the understanding of the cement treated crushed rock materials. It proposes a new numerical fatigue model and provides an alternative classification methodology by incorporating the other key characteristics studied by this paper.

The following publications have resulted from the work carried out for this degree.

Peer Reviewed Conference Papers

Yeo, Y. S., P Jitsangiam, and H. Nikraz. 2009. Sustainability assessment of hydrated cement treated crushed rock basecourse (HCTCRB). *Proceedings of the 5th PATREC Research Forum, 1st October 2009*. Perth.

Yeo, Y. S., P Jitsangiam, and H. Nikraz. 2010. Mechanistic classification and characterisation of cement treated base in Western Australia. *Proceedings of the 5th Civil Engineering Conference in the Asian Region and Australasian Structural Engineering Conference, 8th – 12th August 2010*. Sydney.

Yeo, Y. S., P Jitsangiam, and H. Nikraz. 2011. Flexural behaviour of cement treated crushed rock under static and dynamic loads. *Proceedings of the 7th International Conference on Road Pavement and Airfield Pavement Technology*, *3rd – 5th August 2011*. Bangkok. (won best conference paper award and best student paper award)

Yeo, Y. S., P Jitsangiam, and H. Nikraz. 2011. Moisture susceptibility of cement stabilised basecourse. *Proceedings of the 7th International Conference on Road Pavement and Airfield Pavement Technology*, *3rd – 5th August 2011*. Bangkok.

Yeo, Y.S. P. Jitsangiam, and H. Nikraz. 2011. Dynamic effects on fatigue life of cement treated crushed rock. *Proceedings of the International Conference on Advances in Geotechnical Engineering, 7th – 9th November 2011*. Perth. (to be published)

Yeo, Y.S. P. Jitsangiam, and H. Nikraz. 2011. Moisture ingress of cemented basecourse. *Proceedings of the International Conference on Advances in Geotechnical Engineering, 7th – 9th November 2011*. Perth.

Yeo, Y.S. P. Jitsangiam, and H. Nikraz. 2011. Mix design of cementitious basecourse. *Proceedings of the International Conference on Advances in Geotechnical Engineering, 7th – 9th November 2011*. Perth.

Yeo, Y.S. P. Jitsangiam, and H. Nikraz. 2011. Erodibility of stabilised pavements using the wheel tracking test. *Proceedings of the International Conference on Advances in Geotechnical Engineering, 7th – 9th November 2011*. Perth.

Refereed Journal Papers:

Yeo, Y.S. and H. Nikraz. 2011. Cement stabilisastion of road base course: a chronological development in Western Australia. *Australian Geomechanics*. September Ed. pp 53 – 67.

Copies of these papers are presented in Appendix A.

PREFACE

The exploration of the unknown has been the earmark of civilization's greatest, as it resonate the virtues that define humanity, viz. patience, resilience, faith, leadership and astuteness. In an era of rampant technological development, the undertaking of postgraduate research may not be akin to the iconic days of first voyages. Yet, the process demands similar rigour and virtues.

My decision to embark on my own exploration stems from the belief of my parents and family in me. For it was their support, sacrifice and trust that formed my pillars of strength – my success in the past, present and future is attributed to them. A special thanks to my brother Yeo Chou Seng for the proof reading of my manuscript.

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Last but not least, thank you to my lovely wife, Esther, for her unconditional sacrifice, support and love. Undertaking postgraduate research work whilst employed can be a strain on any relationship, but she has instead stood by my side through the entire journey. I am blessed to have someone like Esther as a life partner, and I look forward to our future together.

Thank you.

Yang Sheng Yeo (Bay)
October 2011

To my parents, brothers, sister and wife.

You are my world.

TABLE OF CONTENTS

APPENDICES

LIST OF FIGURES

LIST OF TABLES

1 Introduction

In setting the scene for this dissertation, this chapter provides a succinct chronological account of the background to basecourse cement stabilisation in Western Australia, and the major issues surrounding its use. Although this will be elaborated on in greater detail in subsequent chapters, the account is presented to contextualise the issues and detail the fundamental objectives of the dissertation. It is later followed by a statement of the significance of the research, the research approach and an overview of the structure of the dissertation.

1.1 Background

Roads have always been an integral part of human civilisation. Since the first recorded stone-paved streets of Ur (modern day Iraq) in 4000 B.C., roads have evolved into the primary terrestrial network for freight and commuters and they are now recognised as an icon of human ingenuity; a symbol of the modern built environment. Despite the familiarity of roads to the daily commuter, the science and engineering of roads and pavements is built upon an elaborate scheme of numerical models and empirical data. These include: social science to quantify driver behaviour, material science to investigate material selection, statistical science to model traffic movements, spatial science to determine road alignment, meteorological methods to ascertain drainage conditions, physical science to design pavement integrity as well as countless more disciplines. These sciences form the basis of road and pavement engineering, a field familiar to the civil engineer.

From 1981 to 2010, the Australian population has grown exponentially from 14.9 million to 22.2 million (Australian Bureau of Statistics 2010), translating to a growth of 50%. In Western Australia's (WA) capital city of Perth alone, population growth saw the number of residents in the region increase from 175,000 in 1921 to 715,000 in 1971. By 2009 the population had doubled to reach 1.6 million (Bureau of Infrastructure Transport and Regional Economics 2009). An immediate repercussion of the unprecedented population growth has been the need to extend road networks to connect destinations across wider areas and to upgrade existing roads to accommodate the increased traffic volume.

Roads are constructed from finite resources and optimisation of material use to achieve durability and serviceability is paramount. Cement stabilisation is used widely around the world to allow structurally marginal materials to be used in highway construction. Its use can be traced back to the 1950s when a specialist contractor established itself on Australian shores and this led to the construction of in situ cement stabilised pavements in local government roads in New South Wales (Wilmot 1996; Vorobieff 1998).

However, in Western Australia under Main Roads Western Australia's (MRWA) Engineering Road Note 9 (ERN9) (Main Roads Western Australia 2010), the use of cement stabilised materials is very limited and they do not form part of the structural components of a pavement, as mentioned on clauses 1.1.8 and 4.2 of ERN9 as shown below:

> *Clause 1.1.8:*
>
> *The pavement must not incorporate cemented materials.*
>
> *The pavement must not incorporate any modified granular material that satisfies one or more of the following criteria when tested at its in-service conditions: -*
>
> *(a) 7-day unconfined compressive strength (UCS) of the material exceeds 1.0 MPa;*
> *(b) 28-day UCS of the material exceeds 1.5 MPa; or*
> *(c) Vertical modulus of the material exceeds 1500 MPa*
>
> *Clause 4.2:*
>
> *No reduction in thickness requirements can be made for pavements incorporating granular material modified with cement, lime, bitumen or other similar materials.*

Clause 1.1.8 of ERN9 implies that cement stabilised materials that have developed Unconfined Compressive Strength (UCS) in excess of the limits stated in the clause shall not be used to construct roads in Western Australia. Moreover, even when the UCS of the cement stabilised material is below the limits set out in clause 1.1.8, the strength gained from the stabilised material shall not be accounted for when undertaking the pavement structural design as per clause 4.2.

The basis for the above clauses can be traced to the development of cement stabilisation methodologies in Western Australia. MRWA has in the past used cement stabilised materials for road construction, and it has undertaken various initiatives to develop the use of cement stabilisation practices in WA which include laboratory investigation, construction of trial pavements and in situ monitoring of pavements. However, from its initial inception in 1975 to the present day, results from these initiatives commissioned by MRWA significantly lowered the confidence of MRWA in cement stabilisation. The milestones of the developments are summarised in Figure 1.1 shown below.

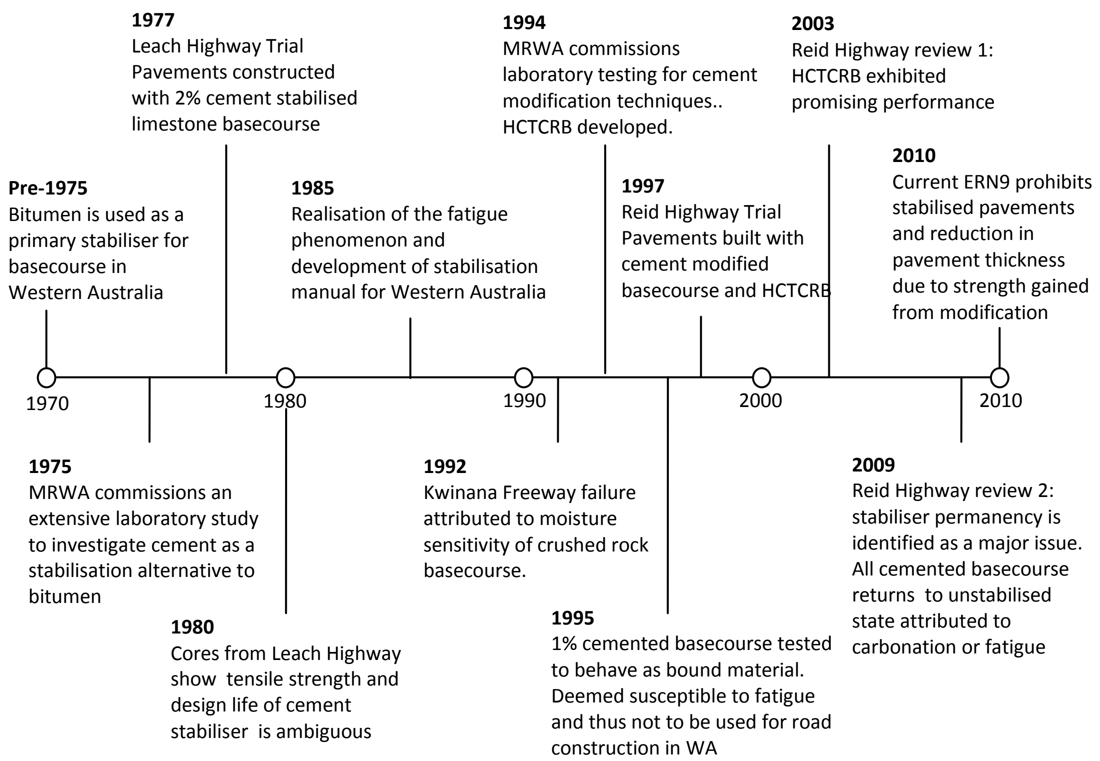


Figure 1.1: Chronological development of cement stabilisation in Western Australia

The chronological outline in Figure 1.1 is covered in greater detail in the next chapter but is presented here to explain the root of the issues surrounding cement stabilisation in WA. Based on the limitations of ERN9 and the milestones shown in the timeline in Figure 1.1, the following issues can be drawn:

i. Ambiguous mechanistic definition of cemented materials means that cement treatment of more than 1% would classify them as heavily stabilised materials

ii. Fatigue and shrinkage cracking of stabilised pavements means that stabilised materials are not to be used

iii. Modified materials have issues with stabiliser permanency and structural benefits from its use are to be disregarded

These issues are further explained in the subsection below.

1.2 Ambiguous Mechanistic Classification of Cemented Materials

The categorisation of cement treated basecourse in Australia is based on the degree of binding of the material, i.e. semi-bound (modified) and bound (stabilised). The delineation between modified and stabilised is crucial in determining the mechanical behaviour of the material and subsequently, the design methodology of the pavement.

Nevertheless, ambiguity exists in the delineation point between modified and stabilised. This is especially valid in WA where the limitations prescribed in clause 1.1.8 of ERN9 have not been verified against characteristic definitions of bound materials, i.e. fatigue and shrinkage. Instead, the delineation point depends solely on Unconfined Compressive Strength (UCS) limits based on “industry customary” empirical observations and not scientific evidence.

The ambiguity is not a local conundrum but a well known issue in pavement engineering as highlighted by several eminent experts in the field as can be seen in the comments below:

"It is important to distinguish between modification and cementation because these terms are used extensively in South Africa....There is no clearly defined boundary between cementation and modification. The one state overlaps the other."

- Jenkins (2006)

"There is no internationally recognised and consistent definition which clearly establishes the difference between a modified and bound pavement material...."

- Vorobieff (2004)

1.3 Fatigue and Shrinkage Cracking

Building upon the previous issue, tests undertaken in 1995 by Lee Goh (Lee Goh and Butkus 1997) showed that the UCS limits of stabilised materials are exceeded when as little as 1% cement is added. This inferred that the composite mix of crushed rocks and minimal cement content would be highly susceptible to fatigue and shrinkage cracking; distinct traits of stabilised materials as described by Austroads (2006). This becomes a disputable contention considering the application of higher cement content is required to achieve stabilisation in pavements in other states in Australia and other major developed countries. The limited scientific understanding of the fatigue and shrinkage behaviour of cement stabilised pavements in WA is the primary reason for the inference regarding fatigue and shrinkage cracking.

1.4 Moisture Sensitivity and Stabiliser Permanency

As for modified materials where the cement treatment is comparatively lower in content, issues of stabiliser permanency are prominent. Cored samples and deflection measurements from the Reid Highway Trial Pavement in 2009, as indicated in Figure 1.1 showed that all cement treated pavements exhibited similar performances to untreated material and a retardation of stabiliser content occurred. This observation is also seen in other parts of the world as quoted by leading experts in the field of cement stabilisation:

> *"The permanency of stabilisation is a major concern with all stabilising materials. Many state departments of transportation have experienced problems with stabilisers "disappearing" after a few years in service. While this predicament is more common in layers stabilised with lime and fly ash, cement treated materials have also been found to be susceptible to chemical reversals of the stabilisation process."*
>
> *– Guthrie et al. (2001)*

> *"Contrary to what has often been stated in literature, stabilisation is not always permanent, in spite of many examples of such permanence being cited. Some of the reactions involved are reversible and reaction products are only stable under certain conditions."*
>
> *– Paige-Green et al.(1990)*

Furthermore, the basis for cement modification of crushed rock basecourse in WA is to overcome the moisture sensitivity of material. The laboratory tests undertaken by MRWA thus far have been focussed on the "effect" rather than the "cause", i.e., the effect of the resilient modulus of crushed rock under various moisture conditions rather than the cause or mechanism for moisture intrusion and stabiliser permanency.

As a consequence of the above limitations, the benefits from cement treatment as recognised by industries around the world are not reaped. It is against this background that this dissertation will develop its research aim and attempt to develop new understanding for the purpose of improving the guidelines around pavement engineering, both in Western Australia and Australia wide. Detailed objectives are discussed in the subsequent section.

1.5 Scope and Objective of Research

Based on the limitations of information on cement stabilised materials in WA, as discussed in the previous section, the fundamental objective of this research is to understand the behaviour of the material and subsequently address the major issues limiting its use in pavement construction by providing engineering practitioners with better design guidelines.

In assessing the behaviour of the material, this dissertation will examine the engineering properties that are associated with cement treated basecourse performance. These include compressive strength, tensile strength, fatigue behaviour, shrinkage and moisture sensitivity. By understanding these properties, a formulation of mechanical behaviour models can be established to develop a better classification methodology for cement stabilised basecourse materials.

More specifically, it is hoped to achieve the objective through the detailed scope of work outlined below:

i. develop and undertake a laboratory program to assess the fatigue mechanism of cement treated crushed rock with various cement content

ii. develop a fatigue model of cement treated crushed rock basecourse with various cement content and validate the fatigue model through finite element modelling methodologies

iii. develop and undertake a laboratory program to assess the shrinkage potential of cement treated crushed rock basecourse with various cement content

iv. develop a shrinkage model of cement treated crushed rock basecourse with various cement content

v. develop and undertake a laboratory program to assess the mechanism of moisture ingress into cement treated crushed rock with various cement content and subsequently measure moisture effects against stabiliser permanency

vi. develop a model of moisture ingress into cement treated crushed rock basecourse with various cement content

vii. develop a classification methodology based on the characteristics assessed in the research

viii. develop recommendations for stabilised pavement guidelines for Western Australia and Australia as a whole

The broader objective of the research, as a result of this improved appreciation of cement stabilisation, is the determination of more efficient design procedures for cement stabilised basecourse in road pavements, which would allow engineering practitioners in Western Australia to complete pavement design effectively. Consequently, the sustainable use of the finite source of granular materials in Western Australia should be guaranteed.

The following section provides a general overview of the research philosophy in completing the scope of work defined above.

1.6 Research Approach

The research approach of this dissertation in general involves laboratory programs and numerical modelling.

In the development of the laboratory programs detailed above, the scope of the research will include an extensive literature review, study visits and consultations. The literature review will analyse theoretical concepts to establish the necessary physical parameters that will need to be measured. The literature study will also involve a review of both standard and non-standard tests that have been applied both locally and internationally. This will then be followed by study visits to leading laboratories within Australia to examine the practicality of the current non-standard testing methodologies and to identify the constraints of the tests. Professional correspondence with identified leading researchers around the world will also be followed up from the literature review in order to better understand the fundamentals of the new testing methodologies undertaken.

Upon finalising the laboratory programs, extensive laboratory tests combining a repertoire of standard testing methodologies and non-standard innovative approaches are undertaken to understand the physical characteristics of the material.

Numerical models for fatigue will be developed and validated by using the Finite Element Modelling software package Strand7. The models serve to validate the laboratory data and the laboratory observations can then be extrapolated to the practical arena.

1.7 Significance

The significance of this research is that it highlights, on a scientific basis, the limitations of both the current design methodology adopted in Western Australia for cement treated materials and the testing methodologies adopted in Australia for the classification and characterisation of the cement treated crushed rock basecourse.

On the basis of the findings of this research, road authorities may review their current design guidelines while civil engineers might also apply the material characteristics and numerical models developed to better design roads.

1.8 Structure of Thesis

The following shows the list of chapters presented in this dissertation complemented with a brief description of the contents of each chapter.

- Chapter 1 covers an overview to the dissertation highlighting the background, scope, objectives, approach, significance and outline of the dissertation.
- Chapter 2 presents the history of cement stabilisation practices in Western Australia and its implication to current practices.
- Chapter 3 reviews the fundamental theories for each of the material behaviour covered in this dissertation.
- Chapter 4 details the materials and methodologies used in this dissertation.

- Chapter 5 presents the results of the experiment undertaken and provides an analysis of these results.
- Chapter 6 presents a detailed discussion of the results attained from the experiment and forms numerical models. It also presents the finite element modelling work undertaken.
- Chapter 7 presents the conclusion to the dissertation by reflecting on the objectives of the research. It also presents the limitations and recommendations of this dissertation.

2 Cement Treatment in Practice

2.1 Introduction

Chapter 1 has deliberated briefly on the current issues surrounding cement stabilisation of basecourse in Western Australia as well as setting out the objective of the research. With this in mind, this chapter elaborates further on current industry practices relevant to the research to give a more comprehensive overview of the research topic.

In order to systematically conceptualise the approach of this research, the chapter first provides a reflective review of the current classification criteria of cement treated basecourse from both a local and international perspective to establish the different classification criteria currently adopted. This is followed by a detailed account of the development of cement treated basecourse in Western Australia as covered briefly in Chapter 1 of this dissertation.

2.2 Classification of Stabilised Basecourse Materials

This section provides a review of the current classification of cement treated basecourse, with specific examples from Australian road authorities and several selected countries abroad.

The majority of pavement materials, when treated with additives such as cement, are classified based on their inter-particle behaviour. When cement is treated with basecourse, a matrix is formed between aggregates which alter the physical construct of the mixture. In its simplest definition, the aggregates are by default “unbound”. However, when sufficient cement is treated, the cement forms a significant matrix essentially binding the aggregates to form a “bound” composite. It is this degree of binding that forms the basis of cement treated basecourse classification.

2.2.1 The Australian Context

In Australia, the overarching organisation for road and pavement engineering is known as Austroads. This organisation includes membership by Australian and New Zealand road transport and traffic authorities. Austroads provides design guidelines for the improvement of the engineering of roads and traffic, including the provision of classification methods for cement treated materials based on the principles discussed above.

Austroads (2006) defines the degree of binding to be designated as either being “modified” or “stabilised”. “Modified” refers to the material state resulting from applying small amounts of cement where the minimal treatment does not provide an appreciable increase in mechanical performance (Austroads 2006), such as strength. Instead, other attributes of the basecourse are modified and improvements made such as reducing plasticity and improving moisture sensitivity (Austroads 2006). The improvement of these attributes assists in minimising the potential for surface deformation and in increasing durability.

In comparison, “stabilised” or “bound” refers to material where the addition of larger quantities of cement aims to achieve improvements in mechanical performance, which typically results in the development of appreciable tensile strength (Austroads 2006). The composite material essentially provides a relatively stiffer basecourse to minimise structural deformation of the pavement.

The identification of classification points has generally been empirical in nature and primarily based on experience. However, Austroads typically provides a classification criterion by using the Unconfined Compressive Strength (UCS) measure as a simplified approach to define the degree of binding. The UCS is used due to its familiarity to the road construction industry and its ease to complete (Vorobieff 2004). However its implications from a mechanistic perspective are limited, and these are discussed further in Section 3.23.3.

In order to establish a better perspective of the chronological development of material classification, i.e., the progressive changes of limiting UCS values adopted by Austroads,

Table 2.1 and Table 2.2 are presented below showing the classification criteria used in 1998 and 2004 respectively (Vorobieff 2004).

Table 2.1: Typical properties of cement treated materials adopted in 1998 (Vorobieff 2004)

Degree of binding	Design strength[1] (MPa)	Design Flexural Modulus (MPa)
Modified	UCS < 1.0	$\leq$ 1000
Lightly bound	UCS: 1 - 4	1500 – 3000
Heavily bound	UCS > 4	$\geq$ 5000

Notes: 1. 28 day test results, standard compaction and moist curing to AS1141.51
2. For slow setting binders, the 28 day test results will be less than the values shown but will continue to increase in the field for at least 6 to 12 months.

Table 2.1 above shows the classification criteria before 1994 where the primary defining parameter for the degree of binding between modified and stabilised is a UCS value of 1.0 MPa coupled with a design flexural modulus of less than 1000 MPa. Furthermore, upon attaining a UCS of 4 MPa and flexural modulus of more than 5000 MPa, cement treated basecourse is further classified as “heavily bound”.

This criteria has since then been updated to the current classification criteria as shown in Table 2.2 below.

Table 2.2: Typical properties of cement treated materials adopted in 2004 (Austroads 2006)

Classification	Testing Criteria	Performance Attributes
Modified[1]	0.7 MPa < UCS[2] < 1.5 MPa	Flexible pavement subject to shear failure within pavement layers and/or subgrade deformation.
Lightly Bound (Stabilised)	1.5 MPa < UCS[2] < 3 MPa	Lightly bound pavement which may be subject to tensile fatigue or subgrade deformation.
Bound (Stabilised)	UCS[2] > 3 MPa	Bound pavement which may be subject to tensile fatigue cracking and transverse dry shrinkage cracking.

1. Modification is typically achieved by addition of lime, polymer or chemical binders.
2. Values determined from test specimens stabilised with GP cement and prepared using Standard compactive effort, normal curing for a minimum 28 days and 4 hour soak conditioning.

Table 2.2 above shows the classifications, currently adopted in Australia, of cement treated materials and their corresponding UCS strength and performance attributes.

When compared against values adopted between 1998 and 2004, the UCS values for modified materials broaden to a range of 0.7 to 1.5MPa, while UCS values above 3.0MPa are considered to be bound, a change from the original figure of 4.0 MPa.

Unlike its predecessors, the current classification, established in 2004, provides a performance description of each classification, which highlights the failure mechanism of the material. The flexural modulus has been dropped in the current classification methodology and instead included as part of the design equation (see Section 3.2)

This current classification method marks a critical point in material classification as it is driven by observed performance limitations such as fatigue cracking for heavily bound materials. It is believed that by doing so, most of the concerns of the industry around the classification of cement treated materials have been dispelled (Vorobieff 2006).

The amount of cement treated to basecourse affects the gain in UCS values. Austroads (2009) recommends that cement stabilisation of above 2% application rate by mass is typically identified as the delineation between modified and lightly stabilised materials.

Figure 2.1 illustrates a typical relationship between UCS, binder content and cement treated basecourse classifications (Vorobieff 2004).

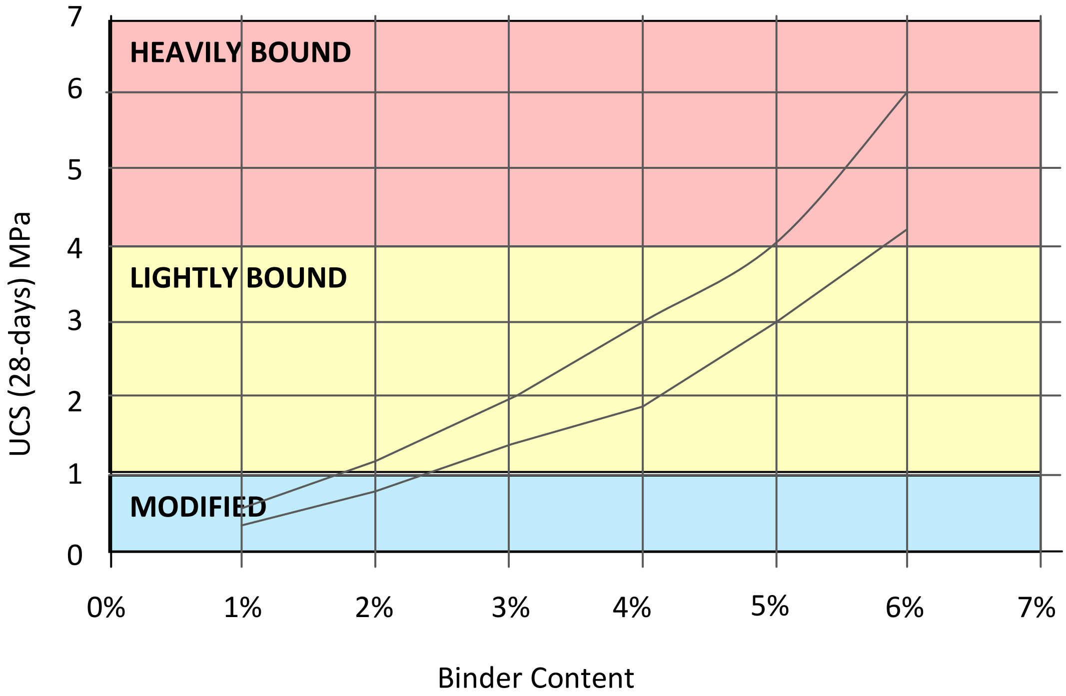


Figure 2.1: Typical relationship between cement content and UCS for material classification (Vorobieff 2004)

Regardless of this overarching approach, each state road authority in Australia adopts slightly varied categorisation approaches for cement treated basecourse. This is presented in Section 2.2.2

Similarly, on an international scale, variability is also evident with different classification criteria being adopted in New Zealand, South Africa, the United States of America and the United Kingdom as presented in Section 2.4.

2.2.2 Local Variations of Australian States

The variability of classification criteria is evident across the states as seen in Table 2.3.

Table 2.3: Regional cement treated basecourse classification (Austroads 2002; Road and Transport Authority 2002; Department of Main Roads 2006; Main Roads Western Australia 2010)

Road Agency	Criteria	Modified	Stabilised	UCS Test Method
Road and Traffic Authority, New South Wales (RTA)	UCS	N/A	GP Cement: 4.0 MPa Slow Setting[1]: 3.0 MPa	100% Standard MDD 7-day cure
Road Corporation Victoria (VicRoads)	Cement Content UCS	2.0% (min.) - .3% (max.) GP Cement: 2 MPa GB Cement: 1.5 MPa Other cementitious: 1.0 MPa	4.5% (min.) - .5.5% (max.) GP Cement: 5.0 MPa GB Cement[2]: 3.5 MPa Other cementitious: 3.0 MPa	100% Modified MDD 7-day cure
Queensland Department of Transport and Main Roads (Queensland DTMR)	UCS Design Modulus	1.5 MPa ± 0.5 MPa 600 MPa[3]	Category 1: 3.0 MPa Category 2: 2.0MPa Category 1: 3500 MPa Category 2: 2000 MPa	100% Standard MDD 7-day cure
Department of Transport, Energy and Infrastructure, South Australia	Cement Content UCS Resilient Modulus	1.0% GB Cement: 2.7 MPa[4] 1000 MPa	3.0% typically 4% for virgin quarried materials 4.5% for recycled materials GB Cement: 4.0 MPa 2000 MPa	96% Modified MDD 28-day cure
Main Roads Western Australia[5]	Cement Content UCS Vertical Modulus	0.5% - 0.75% typically 2.0% for HCTCRB[6] < stabilised limit < stabilised limit	1.0% 1.0 MPa (7-day) 1.5 MPa (28-day) 1500 MPa	100% Modified MDD 7-day and 28-day cure

1. RTA has a preference for slow setting binders in lieu of General Purpose (GP) cement for the added working period during construction. 3.0 MPa limit applies provided at least 1.0 MPa UCS strength gain between 7 days and 27 days cure or between 7 day accelerated test and 7 day normal curing test (Vorobieff 2004; Yeo 2008)
2. The minimum 7-day UCS is based on a cementitious blend of 75% cement and 25% fly ash. Where other combinations of stabilising agents is used, the minimum 7-day UCS is to be determined based on laboratory testing to ensure a 1-year UCS equivalent to the 75/25 cement/flyash blend is achieved.
3. 350 MPa to be used when deflection assessment not undertaken
4. Assessed based on relationship between resilient modulus and UCS given by, resilient modulus, M_R = 1245 x UCS 300 (DTEI 2006)
5. Main Roads Western Australia do not allow cement treatment for structural purposes, refer Section 2.5.3 for further details
6. Hydrated Cement Treated Crushed Rock Basecourse, refer Section 2.5.2 for further details

2.3 Curing Regime of Cement Treated Basecourse

Further to the difference in classification criteria, another criterion to be investigated is the sample preparation adopted by the different road authorities. The recent publication of AS5101.4 (Australian Standards 2008) has allowed the standardisation of methods to prepare stabilised material samples but the standards do not cover the specifics of the curing regime of the test specimens, and suggests a 7 day or 28 day curing period. Table 2.4 shows the curing methods for UCS tests in various locations (Deparment of Transport 1986; Austroads 2002; Road and Transport Authority 2002; Department of Main Roads 2006; Halstred et al. 2006; Main Roads Western Australia 2010)

Table 2.4: Curing regime for UCS specimens (Deparment of Transport 1986; Austroads 2002; Road and Transport Authority 2002; Department of Main Roads 2006; Halstred et al. 2006; Main Roads Western Australia 2010)

Investigator / Author	Curing regime for UCS
AustStab	28 days at 23°C ± 2°C or 7 days at 65°C ± 5°C
VicRoads & NSW Local Governments	7 days at 23°C ± 2°C
QDMR	3 days at 40°C
MRWA	28 days “wrapped cured” at room temperature
Portland Cement Association	7 days in wet room
South Africa	7 days at 100% Mod AASHTO Compaction

The development of 7 day treatments have generally been encouraged in the industry due to tender deadlines and cost (Vorobieff 2004). Comparative testing has shown that the 7 day tested values ranged from 70% to 80% of the 28 day testing value (Vorobieff 2004).

2.4 International cement treatment categorisation

In order to develop a better understanding of the classification practices, an international perspective has been gathered from New Zealand, South Africa, the

European Union and the United States of America as discussed in the following subsections.

2.4.1.1 New Zealand

New Zealand adopts the Austroads design guidelines for pavement design. However, stabilisation is identified as being impractical in New Zealand due to issues pertaining to fatigue and shrinkage.

Instead, the practice of modification is used to improve performance of basecourse materials. It is identified that the distinction between the two materials is difficult to define and generally limited to a maximum of 28-day UCS of 1 MPa or 7-day UCS of 0.7 MPa (Transit New Zealand 2007).

2.4.1.2 South Africa

Similar to Australia, UCS is used as the defining categorisation method for modified and cemented soil in South Africa. Figure 2.2 below shows the classification of binder treated materials in South Africa.

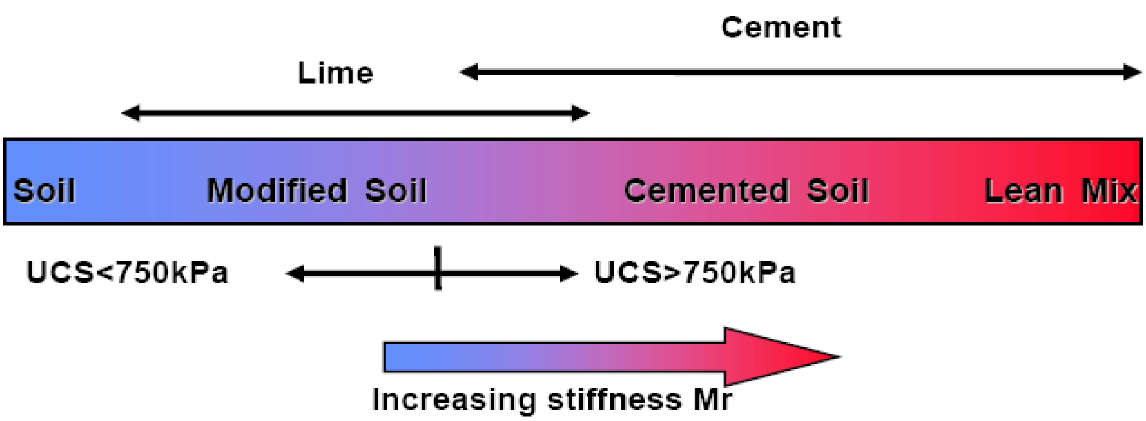


Figure 2.2: Different type and degrees of treatment (Jenkins 2006)

The colour coding shown in Figure 2.2 above is selected to intentionally depict the overlapping traits of the interrelationship between each category. The defining point for stabilised materials is based on a UCS of 0.75 MPa. Lime is also used, as opposed to cement, for modification purposes as lime is deemed to be practical and accurate when used in low application rates (0.5% to 0.75%) as it is half the specific weight of cement, and thus will result in a more consistent blend.

South Africa also goes a step further by classifying stabilised materials into four levels. The different classification types of pavement materials used in South Africa are found in Table 2.5 below (Deparment of Transport 1986; Jenkins 2006)

Table 2.5: Cement treated basecourse classification in South Africa (Deparment of Transport 1986; Jenkins 2006)

Classification	UCS – 7 days at 100% MMD (MPa)	UCS – 7 days at 97% MMDD (MPa)	Minimum ITS (kPa)	Typical Material Used
C1[1]	6 – 12	4 – 8	N/A	Crushed stone
C2	3 – 6	2 – 4	N/A	Stone / Gravel
C3	1.5 – 3	1 – 2	200	Gravel
C4	0.75 – 1.5	0.5 – 1	120	Gravel

[1] C1 materials are no longer used due to its propensity to cause reflective cracking

As noted from Table 2.5 above, the classification of cemented basecourse is differentiated by the degree of binding characterised by the UCS. As seen, a UCS of above 0.75 is the delineation point where the material starts to be classified as bound/stabilised.

Nevertheless, a minimum Indirect Tensile Strength measure is also used in the classification of cement treated materials of lighter bound material, where a minimum of 120kPa is required for classification as C4 material. The typical material used by South Africa is also presented to frame the context of typical applications. It is also prudent to note that C1 material is no longer used, due to its propensity to cause reflective cracking (as discussed further in Section 3.8).

A design model for determining the flexural modulus of field beams has also been developed in South Africa to define the relationship between UCS and flexural modulus, as shown below:

$$\text{Flexural modulus, } E_{FLEX} = k \text{ UCS} \tag{2.1}$$

where, k = values of 1000 to 1250 for GP cements, depending on the testing practices and construction specifications. This is reminiscent of the practice found in the Department of Transport, Energy and Infrastructure, South Australia and Austroads (2002). Higher k values of above 1250 are typically adopted in Australia (Austroads 2002) } to allow for strength gain, post 28 days of curing. This measure potentially reduces the need for unnecessary over application which may result in over stiffening, leading to early failure.

With regard to modifications to the material, the primary purpose is to reduce the plasticity of the soil to conform either to the maximum limits allowable for untreated material, or to a maximum Plasticity Index of 4% (Deparment of Transport 1986).

2.4.1.3 European Union

Both British and the now European Standard(s) have categorised Hydraulically Bound Materials in terms of compressive strengths at 28 days, as well as classifying the material's static stiffness modulus and direct tensile strength. The EU standard also specifies a minimum application rate for hydraulically bound granular material, as per BS EN 14227-1. The minimum application rate requirement for cement treated basecourse is approximately 1% (Specification 800).

The following Table 2.6 has been provided as a guideline to road agencies for (treated) cement treatment for basecourse only, constructed with different layers (Kennedy 2006).

Table 2.6: Cement treatment for basecourse in European Union (Kennedy 2006)

Material	Indicative Compressive Strength (MPa)	Indicative Laboratory Static Elastic Modulus (GPa)	Indicative additions (kg/m^3) assuming components are added separtely
Well-graded & hard	> 8	15	90
Well-graded sand	> 8	10	140
Poorly-graded	> 8	10	200
Weak rocks	> 8	10	200
Brickearth	> 4	5	170

A critical note regarding the European Union standard is the factor of material selection which also affects the performance of cement treated basecourse generally. This is evident from the various different cement content required to achieve a similar minimum indicative compressive strength.

2.4.1.4 Cement Organisation, United States of America

The categorisation of cement treated pavement materials has also been studied extensively in the United States of America. Showing a preference for the typical categorisation of the mechanical state of pavement materials, the Portland Cement Association and Texas Department of Transport, both leading agencies in championing cement treated materials, have typically categorised cement treated material based on application, viz. cement modified soil (CMS), cement treated base (CTB) and Full-depth reclamation (FDR) (Halstred et al. 2006).

The application of CMS resembles that of the modified materials classification adopted in Australia. CMS is treated with a relatively low application rate, typically 3% - 5%. Its purpose is typically to achieve improved plasticity and volume change, while providing added bearing strength (Halstred et al. 2006).

Cement treated base, as its name implies, resembles stabilised or bound cemented material. It has been identified as a material with a compressive strength of between 2.1 MPa and 5.5 MPa, and a modulus of elasticity of between 4100 MPa and 6900 MPa. A typical application rate ranges from 3% to 10%, where increased mechanical performance, i.e., durability, strength, and frost resistance is required (Portland Cement Association 2005; Halstred et al. 2006). Full-depth reclamation is a subset of CTB in which a cement treatment is applied along with the pulverisation of existing pavements.

As can be seen, the minimum application rate for CMS, CTB and FDR are the same, where the minimum cement application rate is 3% (Portland Cement Association 2005; Halstred et al. 2006). This again highlights the overlapping traits of cement treated basecourse with cement treatment in South Africa as discussed previously.

Furthermore, rather than the classification based on UCS, Portland Cement Australia (PCA) suggests that the various classifications of cement treated soil can be plotted against cement content and water content as shown in Figure 2.3 below (Portland Cement Association 2005; Halstred et al. 2006).

Unlike traditional concrete, where water content is primarily driven by the workability of mixture and strength, water in the mix design of cement treated basecourse serves a third purpose of ensuring that compaction of the basecourse is achieved. The workability required for the mixture also differs to that of traditional concrete, since the material is placed via roller-compacting efforts instead of by wet-forming (Portland Cement Association 2005; Austroads 2009). Hence, on top of classifying the different materials, Figure 2.3 below explains the interrelationship of water content and cement content to the workability of the material, denoted as either rolled or cast.

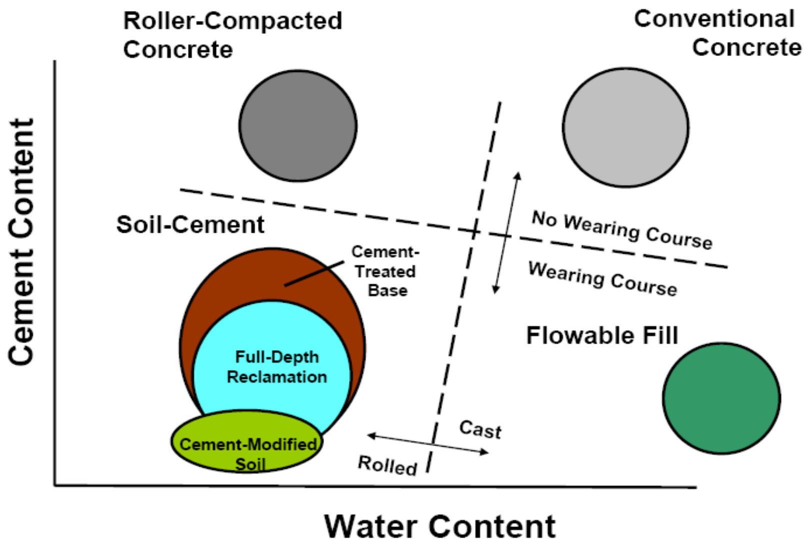


Figure 2.3: Cement treated pavement materials (Portland Cement Association 2005; Halstred et al. 2006)

The gain in strength of a material is primarily a function of cement, but water plays a crucial role in ensuring that the hydration of cement is supplemented with the sufficient amount of water. Without sufficient water, the cement hydration process will not be fully activated (Thom 2010). The minimum water to cement ratio to allow hydration to occur is 0.22 to 0.25 (Hamory and Cocks 1988; Thom 2010). At water to cement ratio in excess of 0.45, the hydration process is also overly diluted and creates porous matrices that are low in strength (Thom 2010). Nevertheless, the increase in water content also reduces the performance properties of the material measured using the Resilient Modulus as shown in Figure 2.5 in Section 2.5.2.1 (Butkus 1985). There is therefore ideal water content for cement treated basecourse.

2.4.2 Review of Existing Classification Methods

The observations from this section substantiate that the Unconfined Compressive Strength (UCS) measure is the universal industry standard for the classification of cement treated materials, due to its ease and the speed at which it can be undertaken, (Vorobieff 2006) rather than its accuracy. The standardisation of procedures in Australia (AS5101.4) and the availability of testing frames or moulding equipment in typical geotechnical laboratories are added advantages for the industry. It can also be seen that the modified proctor maximum dry density is preferred over the standard proctor maximum dry density.

However, significant variability exists between agencies both locally and internationally in terms of the delineating point between modified and stabilised. In addition, there are differences in sample preparation methods. This inconsistency shows that UCS is simply an empirical measurement of the degree of binding achieved for specific materials; UCS cannot explicitly measure the performance traits of cement treated basecourse.

Even from a theoretical perspective, UCS values do not hold any reliable mechanistic inference (Thom 2010) and for this reason have typically not been used directly as a parameter in pavement design (Vorobieff 2006). The inherent structural unreliability of UCS values is further discussed in Section 3.3.

It should however, be recognised that there is an appreciable trend in the defining point of typical basecourse materials, manifested as UCS values of 1 – 2 MPa. This is only true for certain materials, when treated with set amounts of cement, as shown in the preceding tables. It is apparent that blanket rules are not adopted in South Africa and the European Union, where different materials exhibit different UCS values when treated with varying cement content.

A more reasonable definition of cement treated basecourse, as presented by authors in this section, is the gain in tensile strength of the material. The tensile strength thus forms the failure criterion of the material, when used to construct pavements where tensile strains at the bottom of the cement treated pavement layer eventually exhibit noticeable distress in the pavement structure. This is discussed later in Section 3.2.

As opposed to making a distinct differentiating point between a modified and a stabilised condition; there is an inherent benefit in accepting the overlapping traits of cement treated basecourse and designing the layer on the basis of reliable mechanical properties.

In terms of cement content by mass, consistency may be observed when the delineation point between modified and stabilised is found to be in the range of 1% - 2%. Lime is typically recommended for lower binder content requirements (0.5% - 0.75%) to allow better blending, due to its lower specific weight. These observations provide an indicative guideline to the likely ranges that may be expected of this research.

It is important to note that the approach taken by the Portland Cement Association (Portland Cement Association 2005) in its categorisation, which is based on water/ cement ratio and application, may prove to be more useful to the industry compared to a classification system based on the degree of binding. The notion is not discussed at any great length in this section, due to the limitations of the scope of the section, but it supports the need to ascertain a meaningful method of defining the applicability of cement treated basecourse that meets the needs of the industry.

The critical concept to be drawn from this section is that industries will be prone to use simplified methods for classifying materials, as construction is driven by time and money. The UCS measurement is capable of providing this information, but because it is dependent on the cement type selected, aggregate type used, specimen preparation adopted, etc, it is imperative that a direct relationship to actual material specific performance criteria is established. This shapes the way forward for laboratory work, as covered in the subsequent chapter, where other mechanical parameters are shown to be interrelated in one way or another.

Another point that can be inferred from the study is a geographical one. There is obvious variability between states and nations in a geological context. This geological context dictates the type of basecourse virgin material available for construction, the typical sub grade conditions and the quality of cement. This further suggests that Western Australia should be potentially viewed as a unique state compared to other states and territories, as Western Australia has significantly different geological conditions.

With the broad overview of cement classification established, the following section discusses in more specificity, the development of cement basecourse in Western Australia.

2.5 Development of Cemented Basecourse in Western Australia

Cement became a mainstream stabilising agent in eastern Australian states as early as 1950, through the establishment of a specialist contractor, leading to the construction of in-situ stabilisation of local government roads in the 1960s (Wilmot 1996; Vorobieff 1998). The use of cement continues to the present day and is recognised as a potentially cost effective solution for rural road construction (Smith 2005; Austroads 2010). Despite the myriad of documentation surrounding the development of stabilisation in eastern Australian states, as seen in the previous Section 2.2, literature in the public arena on the development of cement stabilisation techniques in Western Australia is limited.

This section therefore presents a more detailed chronological review of Main Roads Western Australia (MRWA), expanding on the brief timeline presented in Chapter 1. The presented information is based on literature on cement treatment in Western Australia which includes technical reports, reports on trial pavements and performance reviews. This is followed by a critical review of this development against the current design methodology adopted by Main Roads Western Australia.

2.5.1 Cement for Stabilisation

With the success of cement stabilisation in the Eastern States, MRWA initiated extensive research works to assess its viability in Western Australia. This included laboratory investigations followed by the construction of Leach Highway Trial Pavements (discussed in subsequent sections).

2.5.1.1 Initial Laboratory Investigation of Stabilisation Options

The typical application of 3% bitumen for basecourse stabilisation was adopted in Western Australia (Hamory 1980) in the 1970s. However with increasing prices of bitumen, more economical options were sought. Subsequently in 1975, a detailed laboratory investigation of limestone stabilised with bitumen and cement in Western Australia was urged by Main Roads Western Australia (MRWA) (Hamory 1977).

The investigation involved testing specimens treated with cement and bitumen, ranging from 1 to 6% each. The significant gain in strength and stiffness from relatively low quantities of cement treatment was noted from the study, where strength gained from 2% of cement was equivalent to that typically achieved for 3% of bitumen treatment (Hamory 1977; Hamory 1980). The results of this investigation are summarised in Table 2.7 and

Table 2.8 below.

Table 2.7: Test results of soaked cement stabilised limestone (Hamory 1977; Hamory 1980)

Cement	1%	2%	3%	4%	5%	6%
Peak Unconfiend Compressive (MPa)	0.755	1.702	2.642	1.860	3.760	4.940
Unconfined Compressive Modulus (MPa)	60	160	340	235	470	680
Tensile Strength (kPa)	68	151	252	329	593	569
Cohesion (kPa)	109	243	464	361	693	794
$\emptyset^{o}$	58	58	56	48	49	54
WACTT Class Number	1.2	0.3	0.0	0.0	0.0	0.0

Table 2.8: Strength parameters of cemented limestone compacted at 0 & 24 hrs delay (Hamory 1977; Hamory 1980)

Cement Content	2%							
Curing Time pre-compaction (hours)	0				24			
Curing Time After Compaction (days)	0	7	14	21	0	7	14	21
Peak Unconfined Compressive (kPa)	69	635	820	770	243	446	585	600
Unconfined Compressive Modulus (MPa)	5	56	80	57	23	28	34	27
Tensile Strength (kPa)	8	54	70	71	18	45	64	55
Cohesion (kPa)	11	89	115	116	32	67	92	87
Internal Angle of Friction Ø (^{o})	54	58	59	68	61	56	55	58
WACTT Class Number	3.6	1.4	1.0	1.0	2.4	1.8	1.6	1.4

As can be seen from the results above, the observations from the laboratory study concluded that a 2% cement mix showed some potential as a superior road stabilising agent as its strength parameters were comparable to a 3% bitumen treatment. The results prompted a further need to assess in-service conditions of cement stabilised limestone and this was realised through the construction of trial pavements on Leach Highway.

2.5.1.2 Leach Highway Trial Pavements

Leading on from the preliminary laboratory investigation undertaken in 1975, trial pavements were constructed on Leach Highway in 1977 using 1% and 2% bitumen stabilised limestone basecourses and a 2% cement stabilised limestone basecourse. (Hamory 1980). The pavement was designed based on the 1977 NAASRA pavement design procedures. Details of the trial pavements are shown in Figure 2.4 below.

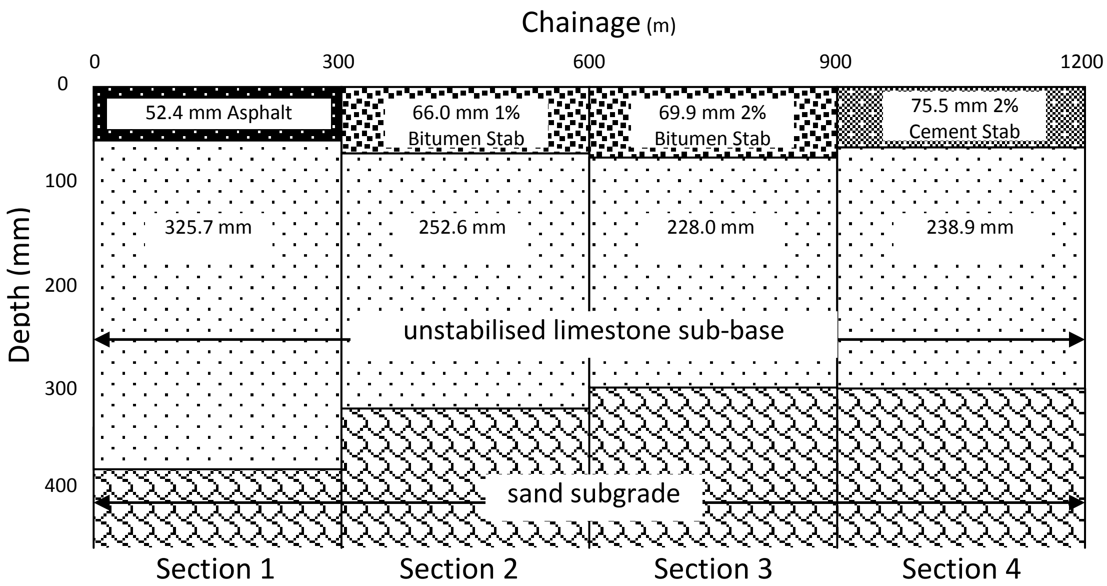


Figure 2.4: Profiles of Leach Highway trial sections measured in 1980/81 (Hamory 1981)

The trial pavements were tested and measured for indirect tensile strength, field moisture and surface deflection. The results were published in comprehensive reports in 1980 and 1981; details are presented in the subsections below.

2.5.1.3 Development of Tensile Strength

Indirect tensile strength tests were undertaken from 25 typically random cores, selected from the trial sections in each assessment period. These results are presented in Table 2.9 below (Hamory 1981)

Table 2.9: Tensile Strength and Moisture Ratio of Leach Highway Trial Pavement (Hamory 1981)

Time of Test	2 (1% Bitumen Stabilised)		3 (2% Bitumen Stabilised)		4 (2% Cement Stabilised)	
	Tensile Strength (kPa)	Moisture Ratio (%)	Tensile Strength (kPa)	Moisture Ratio (%)	Tensile Strength (kPa)	Moisture Ratio (%)
Dec 1977	42	29	55	36	59	65
Aug 1978	72	20	113	23	-	-
Nov 1978	-	-	-	-	57	59
Oct 1979	36	34	52	39	26	66
Dec 1980	50	32	73	42	46	51

As the results indicate, a significant decrease in tensile strength of the cement stabilised basecourse was measured after 2 years in service (1979), which MRWA could not explain at the time (Hamory 1980; Hamory 1981). However, measurements undertaken in 1980 showed a rebound in tensile strength which suggested that the blink in the measurement can be attributed to changes in testing procedures (Hamory 1981). After these measurements, ongoing monitoring of tensile strength showed it as remaining essentially stable (Cocks 1987; Hamory and Cocks 1988). Moreover, by 1980, with the exception of section 3, tested specimens showed that the tensile strength of more than 50% of the cores was lower than the minimum performance criteria at the time, of 55kPa (Hamory 1980).

2.5.1.4 Implications regarding Deflection and Service Life

The low tensile strength measurement was subsequently dismissed, based on deflection monitoring using the Benkelman Beam. Deflection measured during the assessment of the trial pavements concluded that the deflection experienced by the four sections was similar, ranging from 0.06 to 0.10mm as shown in Table 2.10 below (Hamory 1981).

Table 2.10: Equivalent Single Axle and Deflection of Trial Sections (Hamory 1981)

Time of Measurement	Equivalent Standard Axle, ESA ($x10^6$)				Deflection (mm)		
	LH	Centre	RH	Total	Section 2	Section 3	Section 4
30/10/1977	Construction Traffic Only				0.63	0.63	0.56
26/04/1978	0.08	0.09	0.04	0.21	0.48	0.50	0.54
15/05/1979	0.17	0.19	0.08	0.44	0.49	0.51	0.57
23/10/1979	0.26	0.29	0.12	0.67	0.45	0.54	0.55
15/04/1980	-0.34	0.38	0.15	0.87	0.52	0.59	0.59
27/11/1980	0.42	0.47	0.19	1.08	0.43	0.42	0.47
30/04/1981	0.51	0.58	0.24	1.33	0.49	0.53	0.56

The results were then used to estimate the service life of the pavement based on NAASRA 1979 design guidelines which provided the results presented in Table 2.11 :

Table 2.11: Calculated pavement life (Hamory 1981)

Section	NAASRA (1979) Deflection Design Line	Total ESAs in One Direction	Calculated Pavement Life (years)
1	2	$> 3 \times 10^7$	44
2	1	$> 3 \times 10^7$	44
3	1	$> 3 \times 10^7$	44
4	4	1.1×10^6	5

It was argued at the time that the compactions works, during the construction of Section 4, were not completed until 24 hours after the addition of cement (Hamory 1980), which meant that a reduction of 60% could have occurred in the compressive modulus, (Hamory 1977) as seen in laboratory results presented in Table 2.9. This in turn implied that the NAASRA 1979 design line to be adopted could vary from 4 to 2, giving a calculated pavement life of approximately 20 years, (Hamory 1980) rather than the 5 years calculated with Deflection Design Line 4.

In comparing the design life and the deflections measured, the pavement was capable of providing adequate performance for the life of the pavement (Hamory 1980). It was later understood that the material had started to behave as unbound material due to extensive cracking (Hamory and Cocks 1988), a point discussed in a subsequent section.

In validating the results, the Australian Road Research Board (ARRB) was also engaged to conduct test tracks to assess the four trial sections using a full scale test known as the Accelerated Loading Facility (ALF). The assessment ranked limestone stabilised with 2% cement as the best performing basecourse material particularly when it was used in poorly drained areas (Hamory and Cocks 1988).

Further to these measurements, initial observations carried out during the construction of the trial pavements also pointed out that the cement treated limestone exhibited adequate strength with significantly high CBR values, doubling that of bitumen treated sections (Hamory 1980).

2.5.1.5 The Realisation of Fatigue Cracking

The first appreciation of issues pertaining to materials with high stiffness was reported by Hamory (1977) during preparation of cement treated limestone specimens for laboratory investigations. It was noted that a potential risk of cracking existed for cement stabilised limestone basecourse due to the stiff behaviour of the material. However, its correlation to fatigue phenomenon was not yet established. The theoretical discussion of the fatigue phenomenon is presented further in Section 3.4, while this section focuses primarily on the experience of MRWA.

It was only in the mid 1980s that the concept of fatigue cracking of bound material was introduced regarding pavements in Western Australia. The concept was substantiated by the difficulty experienced when obtaining intact cores from cement stabilised basecourse from Leach Highway trial pavements, an observation attributed to the development of extensive microcracks at the time (Cocks 1987; Hamory and Cocks 1988).

Moreover, measurements of trial pavements up to 1986 indicated that the calculated allowable ESA for sub grade deformation based on NAASRA 1986 design charts is 0.75×10^6, which contradicts both observed deflections and in-service road conditions. The combined observations suggested that the design life calculations used were unsuitable. The following conclusions were also drawn:

i. the cement material had undergone extensive fatigue cracking and was now acting in “blocks” of unbound granular material, which when calculated as such would provide a more realistic allowable ESAs limit; and

ii. the sub base characteristics of Western Australia were not compatible with the design equations used by NAASA.

In 1987, through the documented works of Sales (1987) and Cocks (1987), the dependent relationship between the bound behaviour of materials and fatigue was established. The development of CIRCLY and the inclusion of fatigue criterion in NAASRA 1986 design guidelines meant that a mechanistic analysis of pavements with bound layers was assessable. Such an assessment was undertaken by MRWA to back-calculate the load-deflection relationship measured from the trial sections on Leach Highway. It was concluded from the report that the NAASRA methodology was "dubious" and did not provide any conclusive relationship (Sales 1987). Furthermore, Sales (1987) noted that the inconclusiveness was likely to be caused by the sand sub grade underlying Leach Highway, providing adequate support for the stabilised layer "blocks".

In the same year, the *Pavement Design using Bound (Stabilised) Materials* was developed by MRWA (Cocks 1987). In the guidelines, the post-cracking phase concept of pavements was introduced and a suggestion made that in scenarios when the failure criterion of fatigue is not specified, the design of the basecourse shall be considered as unbound granular material. This concept is also further discussed in Section 3.4.

2.5.2 Cement as a Modification Method

Cement modification is not a new technology in Western Australia. Cement treatment of limestone basecourse has been used for some time in Western Australia as a modification method to reduce the moisture sensitivity of basecourse through the lowering of the Plasticity Index (PI) and the Linear Shrinkage (LS) for constructing floodways and other moisture sensitive structures.

For example, cement treatment was carried out on gravel basecourse as part of the Great Northern Highway at Sandfire (Hamory 1979). Samples collected from construction showed that the PI was reduced by 44% and 60% in the samples collected from the windrow and pavement respectively. LS on the other hand was reduced by 33% and 52% respectively. The difference was associated with the non-uniform distribution of cement treatment and the limitations of the testing methodology.

The use of cement as a modification method later came under further scrutiny due to its limitations as a stabilisation agent. The development of cement modification is presented in the subsequent sections.

2.5.2.1 Kwinana Freeway and Crushed Rock Basecourse

In the early 1990s, there was an increase in the use of crushed rock as a basecourse material in projects such as the Kwinana Freeway. However, during the construction of the Kwinana Freeway, several sections between Yangebup Road and Farrington Road, Welshpool Road, were noted to have failed (Watson 1995). The failure of the roads was associated with the sensitivity of the crushed rock basecourse to moisture. This prompted an urgent need to better understand the behaviour of crushed rock basecourse.

An extensive testing program was thus initiated by Main Roads Western Australia to analyse the response of crushed rocks to varying conditions of moisture, compaction, and modification techniques (Watson 1995). Cement use was included for investigation among the possible modification methods.

This investigative work, commissioned by MRWA to assess crushed rock basecourse, involved as its primary objective the assessment of the effects of density and moisture on the resilient modulus. This was achieved by testing samples of varying densities and moisture content, and these included specimens prepared to 100/80, 98/60, 98/50, 96/80 and 96/60 (dry density ratio/moisture ratio) to represent the in-service conditions of basecourses typically found in Western Australia.

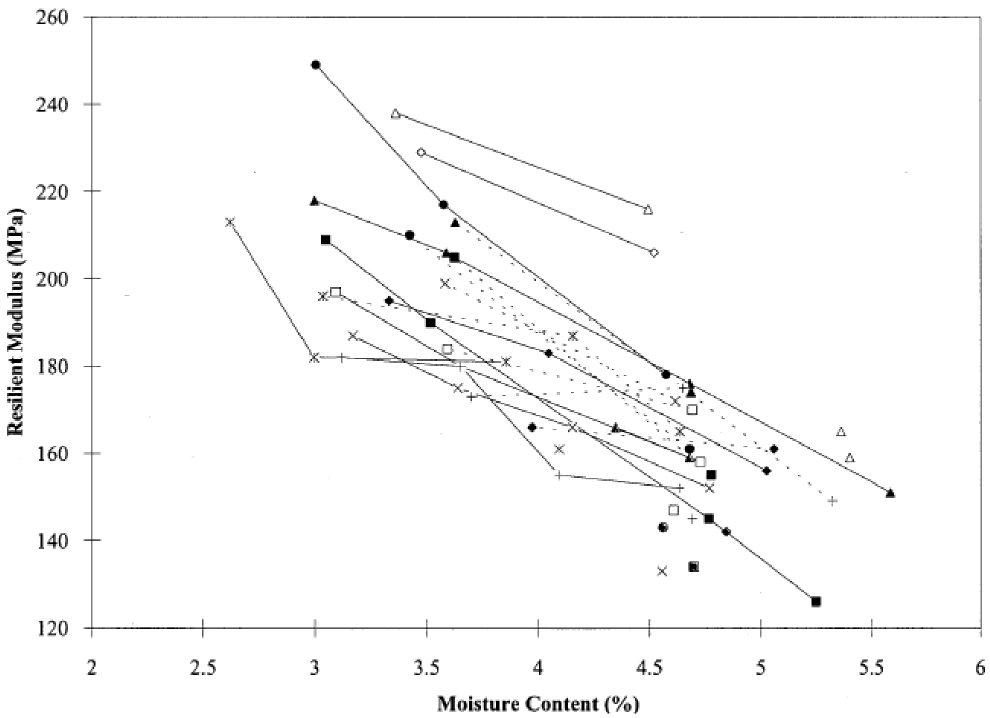


Figure 2.5: Relationship between Resilient Modulus and Moisture Ratio (Butkus and Lee-Goh 1997)

As seen from the results in Figure 2.5, crushed rock basecourse is highly sensitive to moisture ingress, where moisture ratios in excess of 60% may result in poor performance due to the reduction in resilient modulus (Butkus and Lee-Goh 1997). This was also supported by results from the testing of in-situ crushed rock basecourse. Following this, further tests were undertaken to assess the sensitivity of crushed rock base treated with cement, to assess the modified behaviour of crushed rock basecourse.

The cement treated crushed rock was tested for its performance against various cement content (0.5%, 1%, 2% and 3%), the cement setting time prior to compaction, curing time, and a hydration test. The hydration test was designed by MRWA to assess whether part of the improvement of crushed rock base was due to factors other than the cementation process (Watson 1995; Butkus and Lee-Goh 1997). The test involved an interruption of the cementation process by regularly remixing the material prior to compaction. These properties and the conclusions from Butkus and Lee Goh (1997) are summarised in Table 2.12 below.

Table 2.12: Observations of cement modification on crushed rock basecourse (1997)

Properties	Cement content	Observation
Resilient modulus	0.5, 1, 2 and 3	• Increased performance generally (increased resilient modulus, strain rate and permanent strain) • Reduced sensitivity to moisture
Cement setting time	2	• Decrease performance with increasing set time
Curing Time	2	• Increased performance with curing time • Reduced sensitivity to moisture
Hydration Test	2	• Reduced sensitivity to moisture

From the test results shown in Table 2.12, it was concluded that the treatment of cement generally provided better performance against repeat loading and that it showed reduced sensitivity to moisture. However, unconfined compressive strength (UCS) tests undertaken by Lee Goh in 1995 (Butkus and Lee-Goh 1997) suggested that crushed rock basecourse would behave as a stabilised (bound) material when as little as 1% cement was applied; purely because the measured UCS was more than 1 MPa. This implied that the material would undergo fatigue damage and shrinkage cracking; characteristics unfavourable to the road construction strategy in Western Australia.

More significantly, the disturbed product of the hydration test led to the development of Hydrated Cement Treated Crushed Rock Base (HCTCRB) (Butkus and Lee-Goh 1997). HCTCRB is a modified material which is produced by remixing stockpiles of cement treated crushed rock to physically break the cementitious bond. The 2% HCTCRB was deemed at the time to be a superior mix as it exhibited an improvement against the influence of moisture, without the developing the characteristics of stabilised/bound materials.

Following the test, modification by treatment of cement less than 1% and HCTCRB was deemed as a potential option to reduce the moisture sensitivity of crushed rock basecourse. These tests were conducted on trial pavements constructed on Reid Highway.

2.5.2.2 Reid Highway Trial Pavements

The Reid Highway Pavements Trial was constructed as an outcome of the laboratory work discussed in the previous section. Its purpose was to investigate modified basecourse materials, with specific attention being given to HCTCRB. The trial pavements consisted of 9 sections located between West Swan Road and Bennett Brook Bridge in Caversham, totalling approximately 860m in length (Butkus 2004; Harris and Lockwood 2009) as summarised in Table 2.13 below:

Table 2.13: Reid Highway trial section basecourse material and thickness (Butkus 2004)

Section	Modified Basecourse Material	Measured Depth (2009)
1	2% HCTCRB	123mm
2	2% Bitumen Stabilised Limestone	113mm
3	Crushed Rock Base	90mm
4	Crushed Rock Base	211mm
5	1% HCTCRB	210mm
6	2% HCTCRB	211mm
7	0.75% GGBFS Stabilised Crushed Rock Base	231mm
8	2% GGBFS Stabilised Limestone	182mm
9	LIMUD	214mm

GGBFS = Ground Granulated Blast Furnace Slag
HCTCRB = Hydrated Cement Treated Crushed Rock Base
LIMUD = Lime stabilised basecourse
CRB = Crushed rock basecourse

Detailed information regarding the trial pavements was provided in two major reporting periods by Butkus (2004) and Harris and Lockwood (2009). In summary, the Reid Highway pavements trial concluded with the following observations pertinent to the two cement treatment options (Harris and Lockwood 2009):

- Section 7 showed that low cement options gave rise to issues of homogenous distribution and large initial deflections before returning to the performance levels of untreated CRB
- HCTCRB Sections 1, 5 and 6 initially showed marked improvement to moisture sensitivity and against moisture ingress but later assessment showed that they returned to the performance levels of untreated CRB
- the binder content of Sections 1, 5, 6 and 7 was noted to have "disappeared", potentially due to carbonation, as shown in the carbonation test results in Table 8 below
- the deficiency in performance could not be attributed to either loss of stabilisation or fatigue cracking
- transverse cracks were observed on the centreline and shoulders of the HCTCRB sections and were believed to be caused by shrinkage cracks which in turn were caused by the presence of the heavy binding of 2% of cement content (refer Figure 2.6 below).
- limestone stabilised pavements, as tested in Section 8, showed high curvature and this suggests that limestone stabilised pavements are incompatible with cementitious treatment
- thicker pavements generally perform better in terms of the Benkelman beam curvature.

The severity of carbonation for the various cement treated trial sections were tested with phenolphthalein, phenol red and HCl acid tests. These tests were empirical, but it was deemed that the combined testing would provide a reasonable indication of whether cement was still present in the pavements. The results are summarised in as follows:

Table 2.14: Results of Carbonation Test for Cement Treated Sections (Harris and Lockwood 2009)

Sections		1		5		6		7	
Basecourse Description		2% HCTCRB 123mm		1% HCTCRB 210mm		2% HCTCRB 211mm		0.75% GGBFS Stabilised CRB 231 mm	
Sample Chainage		11570	11600	10520	10550	10430	10460	10330	10360
Test Solution	Phenolphthalein	N	N	Y	N	Y	Y	N	N
	Phenol Red	N	N	-	N	-	-	N	N
	HCl Acid	Y	Y	Y	Y	Y	Y	Y	Y
Carbonation Result		Full	Full	Partial	Full	Partial	Partial	Full	Full

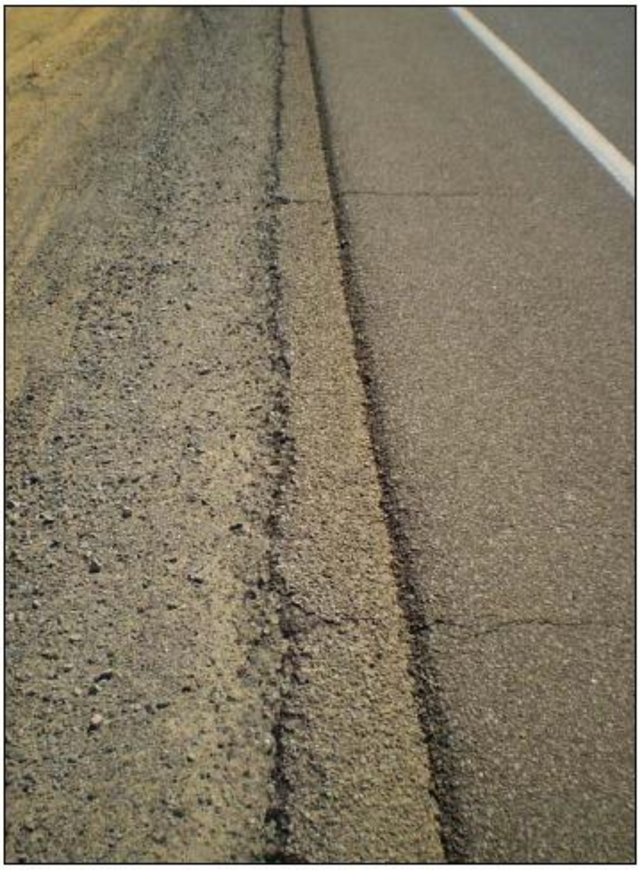


Figure 2.6: Transverse cracking of trial pavements with HCTCRB basecourse (Harris and Lockwood 2009)

2.5.3 Pavement Design in Western Australia - Engineering Road Note 9

The culmination of the laboratory tests and pavement trials resulted in the production of the current pavement design guidelines, Engineering Road Note 9 2010 (Main Roads Western Australia 2011), released by MRWA. The clauses relevant to this limitation has been covered in Section 1.1, however for continuity of this narration, these clauses are again presented below:

Clause 1.1.8:

The pavement must not incorporate cemented materials.

Clause 1.1.8:

The pavement must not incorporate any modified granular material that satisfies one or more of the following criteria when tested at its in-service conditions: -

(d) 7-day unconfined compressive strength (UCS) of the material exceeds 1.0 MPa;
(e) 28-day UCS of the material exceeds 1.5 MPa; or
(f) Vertical modulus of the material exceeds 1500 MPa

Clause 4.2:

No reduction in thickness requirements can be made for pavements incorporating granular material modified with cement, lime, bitumen or other similar materials.

The guideline states that the use of bound materials as structural components is prohibited and modifications shall be limited to UCS values below bound conditions. It is implied that the definition of “bound” is contained in the conditions outlined in (a), (b), and (c) of Clause 1.1.8 shown above. The guidelines are such that even when modified materials are used, any resultant strength gain is not to be deemed to be a structural improvement.

2.5.4 Implications of Past Experiences of Cement Treatment of Basecourse Materials in Western Australia

2.5.4.1 Defining Bound Pavements and Fatigue

The literature presented in this section suggests that the current design clauses within ERN9 are immediate reactions to the observations from the trial pavements that produced limited test data regarding the behaviour of bound materials.

The non-inclusion of cemented materials stems predominantly from UCS tests of 1% cemented crushed rock, tested by Lee Goh in 1995, where material exhibiting UCS values of more than 1.0 MPa was associated with the development of fatigue cracking. Although the limit complements the suggested bound behaviour definition provided by Austroads (2008), it supports the argument presented in Section 3.3 that the UCS is but an empirical reference and does not necessarily represent the actual mechanistic response of the material.

Furthermore, the observations from the Reid Highway Trial Pavements which showed a reduction in stiffness, manifested as increased curvatures measured by the Benkelman Beam, were not conclusively associated with fatigue cracking. The curvatures are deemed to be either a result of fatigue cracking of the pavements or a loss of binder content. It is also important to note that the primary material investigated in the trials was HCTCRB. Some doubt is cast on whether the issues related to fatigue cracking are relevant to the cement matrix of HCTCRB in an undisturbed state.

With fatigue being the primary defining criterion for the limitation of the use of bound material as part of road networks in Western Australia, it is prudent to reassess the definition criteria, especially when no recorded fatigue tests have as yet been initiated by MRWA to confirm the UCS limits suggested in ERN9. Notwithstanding the fact that an overlap exists between the mechanistic behaviour of modified and stabilised materials, a more definitive delineation between modified and stabilised conditions is required to efficiently design pavements, rather than making a blanket rule around the non-inclusion of bound materials. This is one of the primary objectives of this research (covered in Section 1.5).

2.5.4.2 Binder Permanency and Moisture Ingress

The prohibition around reducing the structural thickness of a pavement, from the strength gained during modification by cement treated materials, is a result of problems with the permanency of cement binder content. The “disappearance” of the cement binder content in modified pavements used in the past by MRWA is linked to the carbonation process of binders. Carbonation occurs in the presence of carbon dioxide infused water where the hydrated cement paste undergoes a reverse chemical reaction and dissolves into the water. In a practical sense, when water enters pavements through groundwater intrusion, lateral seepage, etc, carbonation occurs and the pavement returns to an unbound state. This carbonation or impermanency of binder contents therefore limits the durability of the pavement. Further details of carbonation and the mechanics of moisture ingress are discussed in Section 3.7.

Of considerable concern, given the premise that cement modification should reduce the moisture fluctuation sensitivity of pavements, is the presence of a non-durable stabiliser which defeats the stabiliser’s very purpose. It would not only be detrimental to the integrity of the road, exposing the pavement to moisture, but would also become an economic burden, as the benefits of the modification costs would not be realised to their full potential.

The tests that have since been carried out by MRWA provide data on the structural performance of pavements when a certain volume of water enters the pavement. The literature shows that studies have focused on working out the resilient modulus of materials against various moisture ratios of pavement materials. The mechanism by which the moisture enters the pavement has not yet been fully investigated. This presents an opportunity for the design of laboratory tests to understand the mechanisms of moisture ingress and subsequently the rate binder content carbonation.

2.5.4.3 Shrinkage and Transverse Cracking

Shrinkage cracking, evident from the observation of transverse cracking on the centreline and shoulders of seals (as per Figure 2.6) is presented by MRWA as a stabilisation problem. Although a plausible observation, again, no validation work has been commissioned or undertaken to substantiate this. It is however, recognised by the author that there is limited technology available to assess this observation.

It is reported in Harris and Lockwood (2009) that cracking is mostly found in pavements with Section 6 - 2% HCTCRB 211mm, at constant spacings of 2.5 to 3m. The consistent spacing of the cracks and their location suggest that volumetric change may be the cause. Nevertheless, the ageing of the pavement is potentially caused more by fluctuations in the moisture intruding at the edge of the pavement, rather than the shrinkage cracking from the cement hydration process. With HCTCRB pavements, where the cement bonds are broken after 7 days of hydration, the likelihood of their effect on cracks generated after 9 years of service is debatable. Further details about the development of shrinkage cracks are provided in Section 3.8.

In retrospect, the suggestion by MRWA to increase the disturbance period may hold some merit but further laboratory analysis should be undertaken to assess the product, considering that any appreciable modification may be destroyed from late re-mixing activities.

2.5.5 Summary of Western Australian Stabilisation History

As presented earlier, Main Roads Western Australia has been using cement as a stabilisation and modification binder since the early 1970s. However, there are still gaps with regard to the mechanistic behaviour of the material and the resolution of these issues is vital in order to allow the application of the material in Western Australia.

In summary, the key points made here capture the principal research objectives of this dissertation (as discussed in Section 1.5), i.e.

1) there is limited understanding regarding the fatigue behaviour of cement treated crushed rock in Western Australia. The fatigue phenomenon is thus covered later in Section 3.4

2) the sole reliance on UCS tests for definition criteria, without substantial laboratory or field results, has suggested ambiguous classification, supporting the arguments presented in Section 3.3

3) the mechanism of moisture ingress is critical in understanding the durability of cement modified and cement stabilised materials. This will be later covered in Section 3.7

4) limited studies on the phenomenon of the effects of shrinkage and the effects of cement content have been conducted. These will be covered in a later section (3.8).

The previous section has provided a detailed insight into the development of cement treated basecourse in Western Australia.

2.6 Unsealed Roads and Erodibility

Due to the geographical vastness of Australia and the marginally low population density of rural locations in general, unsealed roads form approximately 500,000 km or 65% of the roads in Australia (Australian Road Research Board 1993). The development of the Australian commodity sector requires unsealed road networks to be developed in order to access remote areas.

The network of unsealed roads in Australia comprises built-up gravel roads, graded tracks or unformed roads on natural surfaces. Due to nature of these roads, more than $1 billion is spent each year on the construction and maintenance of unsealed roads (Australian Road Research Board 1993). This suggests some urgency in ensuring a reasonable and sustainable service life for these pavements.

Defects requiring maintenance works on unsealed roads are generally categorised as either surface or structural. Structural defects involve failure of sub grades which result in permanent deformation of the road. On the other hand, surface defects include corrugations, potholes, slippery surfaces, rutting, ice formations, scouring, loose material and loss of surface material (Australian Road Research Board 1993); these defects are generally localised on the surface of pavements and are typically treated by employing re-grading works.

Furthermore, a critical issue regarding unsealed roads is the generation of dust which is a fundamental environmental issue; dust proliferation severely reduces the visibility of trailing vehicles, increases wear and tear of vehicles and is detrimental to health.

2.6.1 Stabilising Unsealed Pavements with Cement

With the issues highlighted above, stabilisation techniques are typically used to improve the serviceability of unsealed pavements. However, stabilisation philosophy in the past regarding unsealed pavements, has generally been to avoid the use of cement binders as they are not compatible with the maintenance regime typically applied to unsealed pavements. Cement stabilisation results in stiff, bound surfaces which precludes routine grading and periodic shaping (Australian Road Research Board 1993).

However, in recent times the use of a cement and slag blend as a stabilisation option in rural Australia has gained momentum, due to its ability to minimise dust generation, reduce reliance on the development of material sources and considerably decrease maintenance frequency on unsealed low traffic roads (Auststab 2009). Potentially, this will reduce the-whole-of-life costs of these roads. In New South Wales, 5 trial sections of unsealed pavement with various stabilising agents were constructed with promising results (Auststab 2009). These are summarised below in Table 2.15.

Table 2.15: AustStab unsealed pavement trial details (Auststab 2009)

Road Name	Town	Reference Density (t/m^3)	Stabilisation Agent Tested
Barber	Griffith	2.2	Quicklime
Woodlands	Wombat	2.2	Cement/slag blend (70:30) and polymer based binder
Old Corowa	Jerilderie	2.05	Cement/slag (80:20)
Four Corners	Jerilderie	-	Quicklime
Back Mimosa	Temora	2.09	Quicklime

All stabilised unsealed pavements trialled showed adequate performance in wet weather conditions except the pavement with polymer based binder which became too slippery when wet. The cost of stabilisation per kilometre worked out to be $22,500 to $39,000 (Auststab 2009).

Since the issues associated with the use of bound pavements for low volume roads, i.e. fatigue cracking, can be avoided, the performance criteria for stabilised unsealed pavements are therefore durability and propensity to generate dust, both manifested by the erosion of the pavement.

2.7 Chapter Summary

This chapter has presented the industrial practices relevant to the classification of cement treated basecourse, from a local, interstate and international perspective. It has shown that the emphasis on UCS as a benchmark in the industry exists mainly because of the ease of, and familiarity with the measurement.

The chapter has also presented an account of the development of cement treated basecourse in Western Australia, providing a key insight into the limitations and challenges faced in the use of cement treatment in the state. It has also accentuated the significance of the research as covered in Section 1.7.

Finally, it has provided an insight into a different use for cement treated basecourse, i.e. in the stabilisation of unsealed pavements. With this understanding, the following chapter therefore examines the fundamental theories of cement treated basecourse.

3 Fundamental Theories of Cement Treated Basecourse

3.1 Introduction to Fundamental Theories of Cement Treated Basecourse

The previous sections have reflected on past experiences with, and the current industrial regime for cement treated basecourse in Western Australia. They have provided an introduction to the issues that have led to the development of the objectives of this dissertation. Consequently, to provide the theoretical background required to supplement discussions presented in this dissertation, this chapter focuses on informing the reader on the engineering and fundamental theories of cement treated basecourse materials.

This chapter's main purpose is to develop appropriate tools and techniques to achieve the objectives outlined in Section 1.5, i.e. to establish which laboratory programs are efficient in characterising cement treated basecourse, and to identify effective approaches to meet the objectives.

Steyn (2007) has identified three broad classifications of material properties, i.e. performance properties, engineering properties, and fundamental properties. Performance and engineering properties have been well understood in the investigation of pavement materials through mechanical testing. However, the fundamental properties of materials are characteristics that are not determined by external conditions but instead remain unchanged while directly influencing engineering and performance properties. An example is the gradation of materials where the moisture content and various other engineering properties can be inferred (Steyn 2007).

Based on the above principles, this section first provides a brief introduction to the current pavement design model adopted in Australia, to provide a framework for the mechanical properties investigated. Subsequently, detailed theoretical discussions are presented regarding the following engineering properties and fundamentals of cement treated basecourse:

Engineering properties

i. Unconfined Compressive Strength, Indirect Tensile Strength

ii. Shear Parameters (internal angle of friction and cohesion)

iii. Flexural Behaviour and Fatigue Phenomenon

Fundamental Properties

i. Cement Microstructure

ii. Moisture Ingress

iii. Shrinkage

iv. Erodibility

3.2 Pavement Design in Australia – Idealised Layered Elastic Design

Austroads' mechanistic design of pavements idealises the structural analysis of pavements in a multi-layered model (Austroads 2008) as shown in Figure 3.1. Within the model, cemented basecourses are characterised as bound materials having developed tensile strength from the formation of interlocking cement matrices between aggregates. Therefore, the critical response of this layer is a deemed a failure when the cement matrix undergoes excessive tensile stresses. The tensile stresses are generated from applied traffic loadings where the underlying sub grade gives way thus creating deformation of the cemented basecourse layer. As a result, tensile strains at the base of the layer are formed, and cracks propagate as bottom-up fractures.

The design approach adopted by Austroads is built on the premise that it assumes a level of idealisation of the true mechanical behaviour of cement treated basecourse materials, i.e. the model idealises the cement basecourse layer as homogeneous, elastic and isotropic (Austroads 2008). This approach simplifies the characterisation of the actual behaviour of the material (Austroads 2008) but in doing so, the versatility of the design approach is lost and does not allow the inclusion of the full spectrum of considerations typically given to pavements.

Although such versatility is often sought after in general design models, to enable broader use by practitioners in various conditions, it is deemed to overcomplicate design models and becomes a deterrent to adoption (Austroads 2008). In other words, fully mechanistic models are not developed, arguably because their intricacy makes it development impossible. An extensive consideration of particulate behaviour and microstructural behaviour (Austroads 2008) becomes a requirement. Instead, due to the averaging of the total volume of cemented materials against local severe deterioration (Austroads 2008), the idealised models currently adopted by Austroads are deemed capable of representing the collective distress of pavements. Nevertheless, in order to reduce the models do not represent the collective distress, a "reliability factor", RF is introduced to calibrate the model against the reliability of the parameters used. This model is shown in Figure 3.1 below.

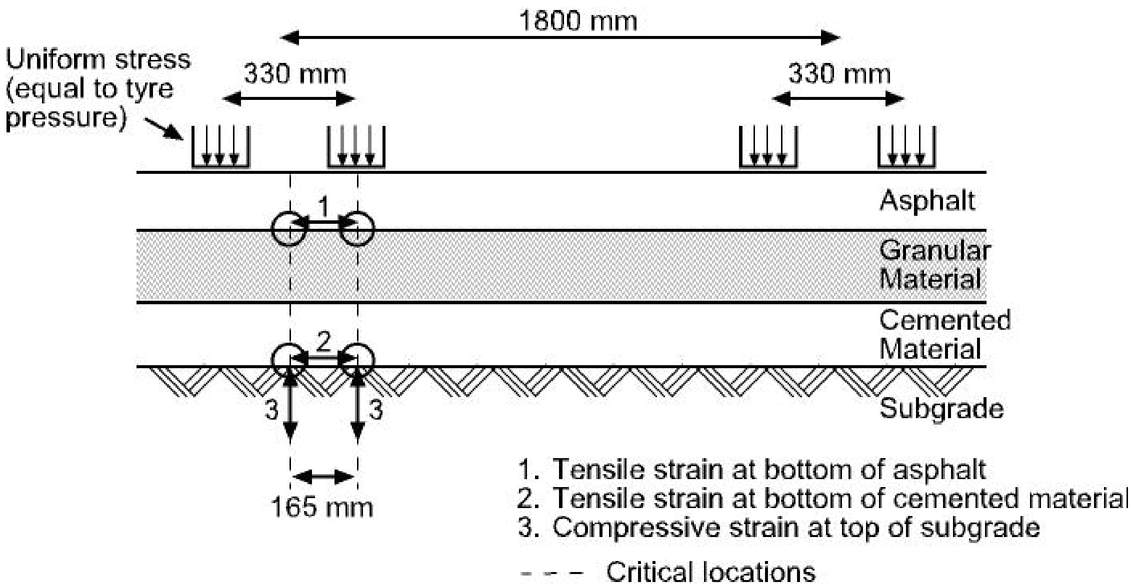


Figure 3.1: Austroads layered pavement model (Austroads 2008)

In the design of the cemented basecourse layer, where basic elastic layered software packages such as CIRCLY are used, the cemented basecourse layer is governed by equation below. As presented in Section 2.5.1.5, from 1998 to 2006 the flexural modulus has since been dropped as a classification criterion and is used now as part of the numerical design calculation of cement treated basecourse.

$$N = RF\left[\frac{11300/E^{0.804}+191}{\mu\varepsilon}\right]^{12} \tag{3.1}$$

where,	N	=	fatigue
	E	=	flexural modulus
	$\mu\varepsilon$	=	load-induced strain
	RF	=	reliability factor for cemented materials fatigue

The equation allows for the calculation of the total number of repeated loads, prior to the failure of the cemented material by fatigue, as a function of the elastic modulus and the applied microstrain. By adopting Miner's hypotheses (Austroads 2008) the life of a cemented basecourse layer can be assessed, where

$$\frac{\sum n}{N} < 1 \tag{3.2}$$

Where	$\sum n$	=	summation of number of equivalent single axle loads
	N	=	number of equivalent single axle loads before fatigue occurs

The summation of the number of equivalent single axle loads (ESA) is a science on its own and is not discussed in this dissertation. Such information is readily available (Austroads 2010). Upon exceeding $\frac{n}{N}$ =1, the cemented layer enters a post-cracking phase where it has the propensity to cause reflective cracking. This is typically mitigated by the construction of an equivalent 175mm thick asphalt layer (Austroads 2008). Due to this requirement, a cemented basecourse layer is often not chosen due to the high costs associated with ensuring that reflective cracking does not occur (Austroads 2008).

The development of this numerical relationship is further discussed in Section 3.4.1. For the purposes of this section, it is taken that the equation is essentially a mechanistic-empirical relationship built upon laboratory results and calibrated against in-situ performance.

It is evident that the above equation offers a simplified approach for practitioners to use. However, for the purposes of this research and in order to achieve the objectives of this dissertation, a more comprehensive and efficient mechanistic analysis is required. This includes the examination an array of engineering and fundamental properties; covered in the subsequent sections.

3.3 Indirect Tensile Strength and Unconfined Compressive Strength

This section discusses the two primary properties of cement treated materials, i.e. compressive and tensile strength, and their combined use to estimate the shear strength parameters of the material, based on a stress envelope. The engineering properties are interpreted based on the Unconfined Compressive Strength test and the Indirect Tensile Strength Test. Following this, the flexural and fatigue behaviour of the material is presented.

3.3.1 Indirect Tensile Strength Test

In order to fully appreciate the implications of an indirect tensile strength test, the typical tensile stress-strain relationship in cement treated basecourse material is to be first understood. As per conventional concrete, the tensile characteristics of cement treated basecourse are non-linear, once the elastic limit is exceeded; as shown in the stress-strain relationship in Figure 3.2. This is because cement treated basecourse is essentially a composite material comprising aggregates and a cement matrix. When continuously loaded, the strain incompatibility of the aggregates and cement results in distress to the cement matrix first, causing a redistribution of stresses to non-damaged areas and thus elastic linearity ceases.

The typical limit of linearity for cement treated basecourse is found to be at a 35% of the maximum strength and 25% of the breaking strain (Deparment of Transport 1986).

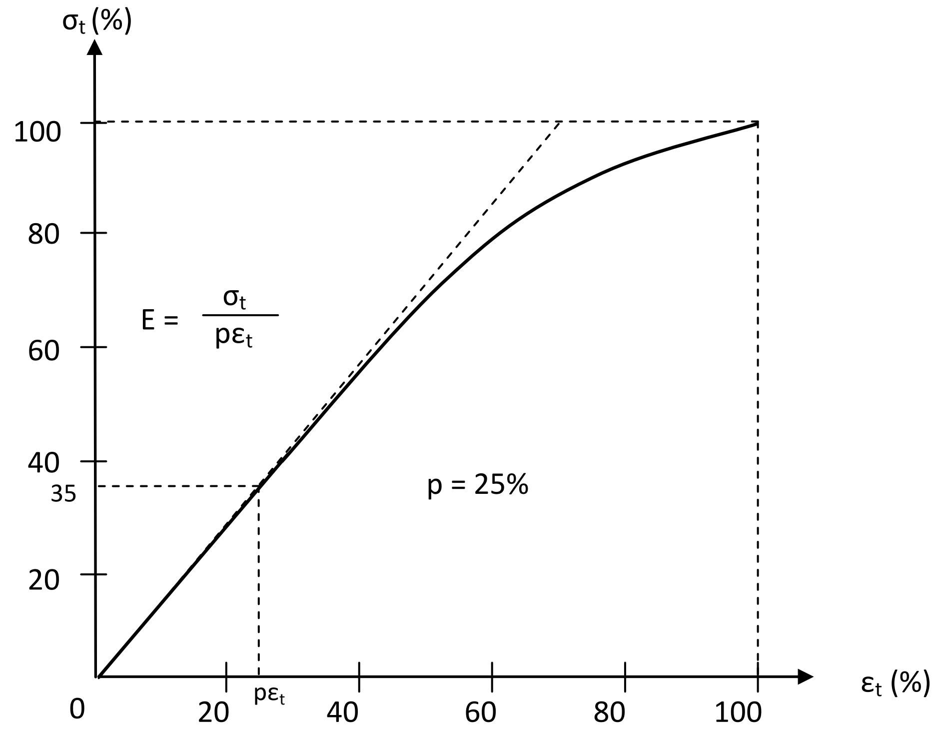


Figure 3.2: Typical tensile stress-strain curve

Due to the impracticality of directly measuring the tensile strength of pavement materials, the Indirect Tensile Strength test is the most commonly used tensile strength testing method. It involves applying a splitting force to cylindrical specimens which thus allows cored samples from in-situ pavements to be used for testing.

The splitting force is generated by applying a compressive force over a small portion of the circumference of the specimen. This is believed to emulate pavement conditions where tensile stresses are only experienced through compressive loads (Thom 2010). Figure 3.3 below shows a typical test setup.

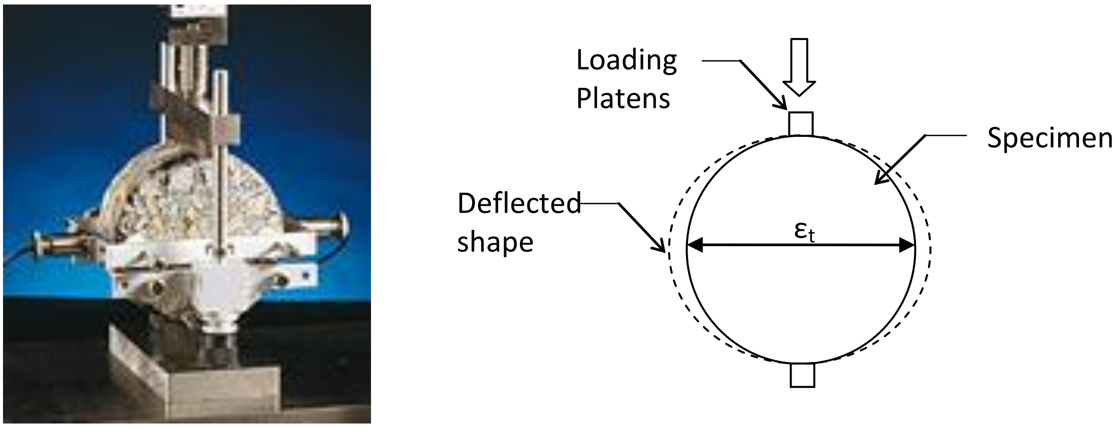


Figure 3.3: Indirect Tensile Strength (ITS) test setup and diagram (Thom 2010)

The measuring tests do not represent a departure from the non-linearity concept discussed earlier. Through detailed analysis of the stresses induced by the test (Thom 2010), the following equation has been developed to explain the indirect tensile strength of tested specimens.

$$\sigma_t = \frac{2P}{\pi D t} \tag{3.3}$$

where P = applied load

D = diameter

t = thickness of specimen

3.3.2 Unconfined Compressive Strength

As highlighted extensively in Chapter 2.5.4.1, the Unconfined Compressive Strength (UCS) Test is the universal empirical classification measurement for cement treated basecourse materials. However, it is inherently just an anecdotal representation of the degree of binding of specific materials and it does not represent a distinct definition of the mechanistic response of pavement (Jenkins 2006; Vorobieff 2006; Thom 2010). This section provides the theoretical evidence to show this limitation.

A typical UCS test and free body diagram is as shown in Figure 3.4 below (Thom 2010).

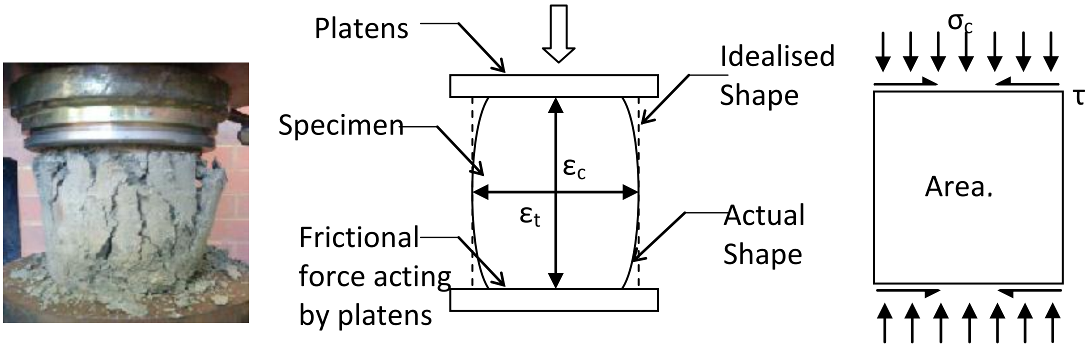


Figure 3.4: Unconfined Compressive Strength (UCS) test (Thom 2010)

Based on the free body diagram, the following calculations can be derived:

$$\text{shear stress, } \tau = \sigma_c \tan \varnothing \tag{3.4}$$

where σ_c = compressive stress

$\varnothing$ = internal angle of friction

Therefore, the average horizontal stress

$$\tau = \tfrac{1}{2}\sigma_c \tan\phi \tag{3.5}$$

By assuming failure of specimen occurs when ε_t is exceeded,

$$\varepsilon_t = \frac{v\sigma_c}{E} - \frac{\tfrac{1}{2}\sigma_c \tan\phi}{E} \tag{3.6}$$

$$\sigma_c = \frac{\sigma_t}{p\left(v - \tfrac{1}{2}\sigma_c \tan\phi\right)} \tag{3.7}$$

Due to the variability of Poisson's ratio, the internal angle of friction, the frictional resistance of platens and the non-linearity of the stress-strain relationship, the measurement of UCS does not hold any true meaning regarding the mechanical property of materials when it is used as the sole measurement.

3.3.3 Relationship between UCS And ITS and the Stress Envelope

Various studies have been undertaken both locally and internationally which have shown a distinct relationship to exist between UCS, ITS and binder content (Andrews 1998; Mohammad et al. 2000; Guthrie et al. 2001; Chakrabati and Kodikara 2007).

Furthermore, the combination of UCS and ITS, (Mohammad et al. 2000) has been successful in estimating the shear strength parameters of the samples, based on the Mohr-Coulomb strength envelopes. A typical Mohr-Coulomb strength envelope and its corresponding equations, used to determine shear strength parameters, i.e. cohesion (c) and the internal angle of friction (Ø) is shown below.

$$\sigma_c = \frac{2c \cdot \cos\phi}{1 - \sin\phi} \tag{3.8}$$

$$\sigma_t = \frac{2c \cdot \cos\phi}{1 + \sin\phi} \tag{3.9}$$

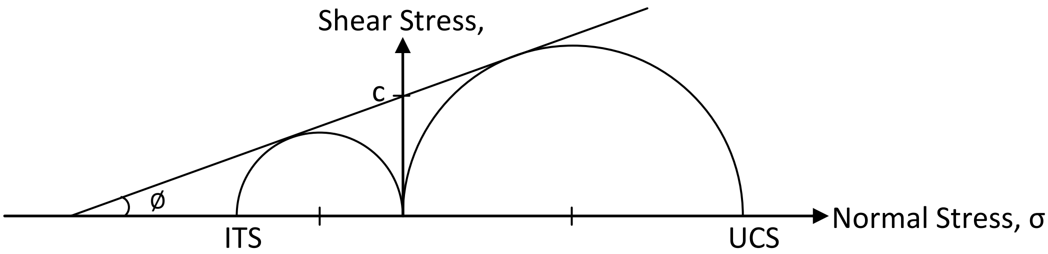


Figure 3.5: Stress envelope based on UCS and ITS

Given the limitations of the UCS test, this simplification process provides an indication of shear strength parameters, i.e. the cohesion, c and the internal angle of friction, Ø. This provides a relatively faster method for the assessment of shear parameters compared with the undertaking of triaxial tests.

This section has provided a theoretical basis for presenting the predicament posed by UCS testing and ITS testing, as well as showing how its combined use can predict the shear strength parameters of cement treated basecourse materials. With this understanding, the following section presents a more representative account of the material properties of cement treated basecourse under traffic loading, i.e. flexural and fatigue behaviours.

3.4 Flexural Fatigue Phenomenon of Cement Treated Basecourse

Flexural fatigue was first discovered after World War 2 and it is a phenomenon that is not well understood in the engineering world. Even so, the study of fatigue predominantly revolves around homogenous and isotropic materials such as steel and other metals. Comparatively, the rigour of research on pavement materials is limited despite the fact that fatigue damage is the fundamental cause of failure in stiff pavement layers, e.g. asphaltic concrete seals and cemented basecourse layers.

This lack of fundamental research into the fatigue of pavement materials is likely to be due to the fact they are composite materials with three different phases, i.e. binder, aggregate and inter-binder aggregate. This adds considerable complexity to providing a mechanistic explanation for their structural behaviour.

It is however, known that the fatigue mechanism of cemented material is characterised as a reduction in stiffness (Austroads 2010) caused by an accumulation of damage at locations of inhomogeneities (Balbo and Cintra 1996) rather than the distinct transverse rupture usually seen in ultimate loadings, as shown in the idealised model in Section 3.2. An apparent limitation and mismatch with the idealised structural model is evident.

The typical behaviour of cement treated basecourse under repeated traffic loading is summarised in Figure 3.6, shown below (Theyse et al. 1996).

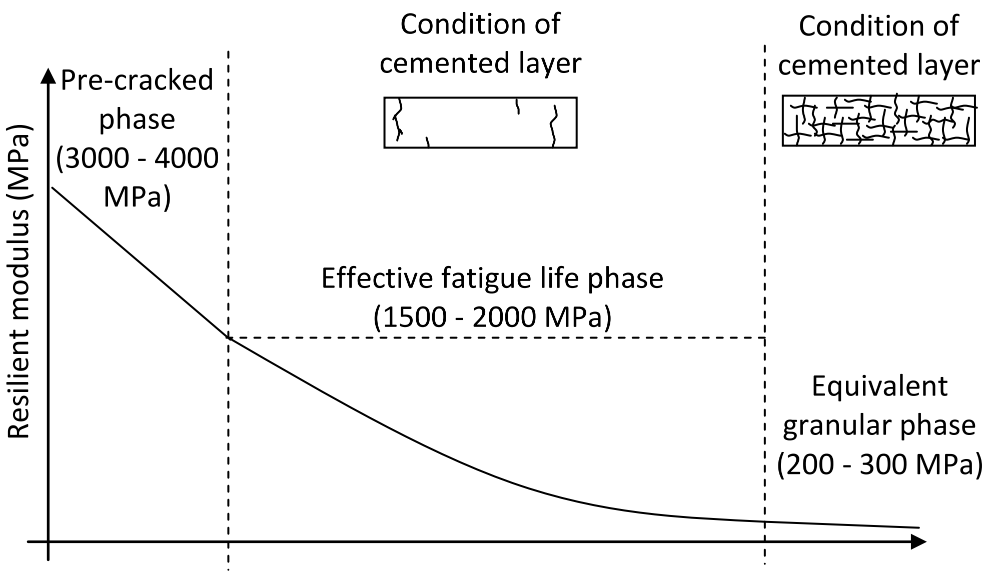


Figure 3.6: Long-term behaviour of lightly cemented material (Theyse et al. 1996)

As seen in Figure 3.6, the accumulation of damage in the pavement structure, manifested as the propagation of cracks within the cement matrix, once averaged across the volume of material affected by repeated traffic loads, will produce a reduction in stiffness. Initially, the first stage of fatigue, known as the pre-cracked phase, is characterised by a sharp and typically linear reduction in stiffness, since significant damage of the material has yet to be applied. A distinct relationship can be drawn between the elastic limit and the pre-cracked fatigue phase of the cement material.

This is followed by an effective fatigue life phase where cracks within the cement matrix have occurred and stresses are constantly redistributed to other sections of the material. As the name implies, this phase of fatigue life is deemed to be the effective design life for cement treated basecourse (Austroads 2010).

Upon further accumulation of cracks within the cement matrix, which can be measured by a continual reduction in stiffness, the material ultimately reaches a distress limit whereby the localised cement matrices are considered to have disintegrated, returning the material to its original unbound mechanical state, a service stage known as equivalent granular phase (Austroads 2008; Austroads 2008; Austroads 2010).

The Australian Road Research Board (ARRB) have been the leaders of research into fatigue characterisation in Australia, and they have been instrumental in the development of the theory of fatigue relationships in the design of cement treated basecourse in Australia. The subsections below provide a brief outline of the development of fatigue design criteria in Australia.

3.4.1 Development of Fatigue Design Criteria in Australia

Austroads design criteria published by NAASRA in 1987 took the form of the equation below (Austroads 2008; Austroads 2010):

$$N = \left(\frac{K}{\mu\varepsilon}\right)^{LDE} \tag{3.10}$$

where, N = number of load repetitions

$\mu\varepsilon$ = applied microstrain

K = material constant as a function of modulus of cemented materials

LDE = load damage exponent = 18

The K values based on the above equations are presented in Table 3.1 below:

Table 3.1: K values for Austroads 1987 fatigue model (Austroads 2010)

Modulus of cemented material (MPa)	Value of K
2000	280
5000	200
10000	150

The interrelationship of microstrain and modulus was adopted from works completed by Otte (1982) which in turn were based largely on works by Pretorius (1969) in the USA (Austroads 2010). Potter (1999) reported that Pretorius developed the model through the testing of high strength materials; significantly different materials to the cement treated basecourses used in Australia (Austroads 2010). Subsequent to this, more research was undertaken in Australia (Angell 1988; Jameson et al. 1992; Jameson et al. 1995) and this led to the development of an upgraded model by NAASRA (1987) in the form of the equation below in 1997 (Austroads 2010):

$$N = RF\left[\frac{11264/E^{0.804} + 190.7}{\mu\varepsilon}\right]^{12} \qquad (3.11)$$

Based on the above equation, the K values can be calculated for various modulus values, as shown in Table 3.2 below.

Table 3.2: K values for Austroads 1997 fatigue model (Austroads 2010)

Modulus of cemented material (MPa)	Value of K
2000	440
3500	350
5000	310
10000	260
15000	240

In adopting the new model there was found to be a dramatic reduction in LDE from 18 to 12. Furthermore, K values have been revised, as shown in Table 3.2 above, to provide a quick reference guide for practitioners. The K values were determined based on the current design model used by Austroads (2010) with minor alterations by Jameson et al. (1992) and Jameson et al. (1995) which included the introduction of the reliability factor.

An inherent trait of all models discussed above is the use of modulus and microstrain within design models throughout the development of fatigue characterisation. It can be said that an injudicious effort of re-developing known approaches has been undertaken, rather than a reassessment of the fundamentals of the fatigue phenomenon.

It is for this reason that Austroads (2010) reported a new approach to modelling the fatigue life of cement treated materials, based on extensive parametric laboratory investigations, coupled with verification using the Accelerated Loading Facility (ALF) (see Figure 3.7 below - taken from technical visit to ARRB). The work involved the testing of an extensive range of pavement materials sourced from all over Australia. These were tested with 3% cement and 5% cement (respectively), and lean mix materials.

Figure 3.7: Accelerated Loading Facility (taken during technical visit to ARRB)

The parametric assessment took into account an assessment of international fatigue models, as shown in Table 3.3 below (Yeo 2008).

Table 3.3: Summary of fatigue relationships of cemented materials in other countries (Yeo 2008)

Origin	Type	Fatigue Relationship
France	Stress-based relationship	$Log\ N = \frac{1}{B}\left[\frac{\sigma_t}{\sigma_b} - 1\right]$
USA	Stress-based relationship	$Log\ N = \frac{1}{0.0825B_{C2}}\left[-\frac{\sigma_t}{\sigma_b} + 0.972B_{C1}\right]$
South Africa	Strain-based relationship	$Log\ N = 9.1\left[-\frac{\varepsilon_t}{\varepsilon_b} + 1\right]$

From the international models above, it can be clearly seen that either the ratio of applied stress over breaking strain or the applied strain over breaking strain is used as a design parameter. This is in clear contradiction to the models that have been adopted in Australia.

The parametric study therefore investigated the level of reliance of various properties, including the concepts of ratios, using a specially designed flexural beam testing frame (refer Figure 3.8 below). Two general tests were undertaken, i.e. a static load test and a fatigue test.

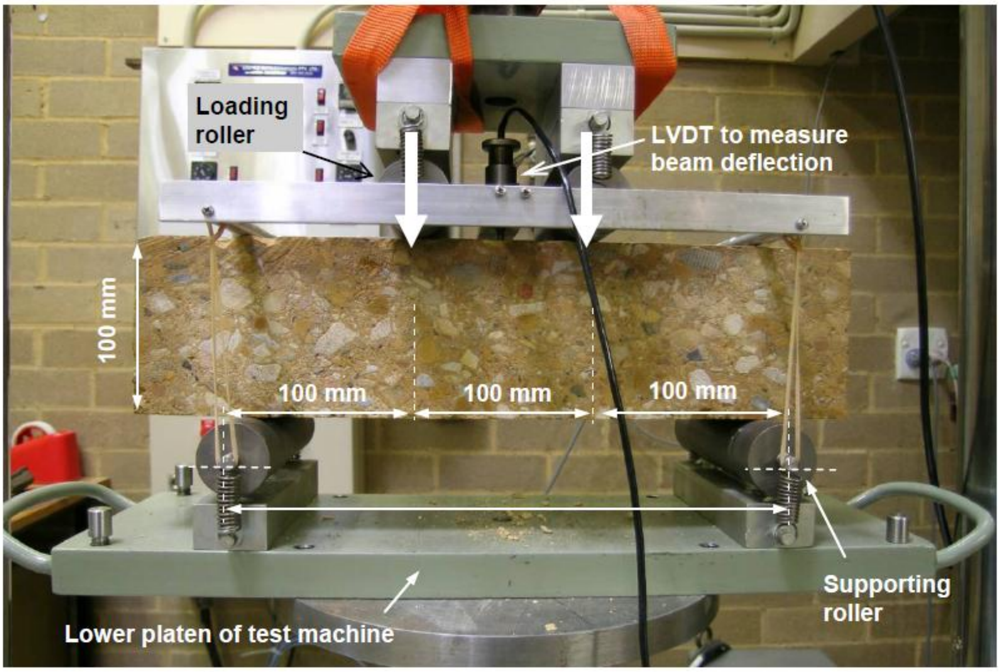


Figure 3.8: Flexural beam testing setup at the Australian Road Research Board

The static load test provided the values of breaking strain, ε_b and modulus, E whilst the fatigue model provided an S-N curve. An initial strain, ε_i was measured from the test during the first 200 cycles of the test. As the frame used by ARRB was in constant load mode (refer Section 3.4.4 for considerations of fatigue testing), the initial strain was deemed to be the elastic strain of the material. The fatigue test undertaken, using the above testing frame, typically generated the following S-N as shown in Figure 3.9. The graph supports the behaviour of cemented materials as discussed earlier in this section.

The parameters studied and their interrelationship is summarised in Table 3.4 shown below.

Table 3.4: Results of parametric study by Australian Road Research Board

Analysis number	Dependent variable	Independent variable	Independent variable	Independen t variable	R-square	Standard error
			Strain based equations			
1	log (N)	μ_t	-	-	<0.25	0.76
2	log (N)	Log (μ_t)	-	-	<0.25	0.77
3	log (N)	μ_t	μ_b	-	0.35	0.67
4	log (N)	log (μ_t)	log(μ_b)	-	0.29	0.70
5	log (N)	$\mu_t/\ \mu_b$	-	-	0.33	0.68
6	log (N)	log($\mu_t/\ \mu_b$)	-	-	0.32	0.68
7	log (N)	μ_t	-	E	<0.25	0.76
8	log (N)	log (μ_t)	-	E	<0.25	0.77
9	log (N)	μ_t	μ_b	E	0.37	0.66
10	log (N)	log (μ_t)	log(μ_b)	E	0.34	0.67
11	log (N)	log($\mu_t/\ \mu_b$)	-	E	0.33	0.68

Analysis number	Dependent variable	Independent variable	Independent variable	Independen t variable	R-square	Standard error
			Stress based equations			
12	log (N)	σ_t	-	-	<0.25	0.81
13	log (N)	Log (σ_t)	-	-	<0.25	0.81
14	log (N)	σ_t	σ_b	-	0.27	0.69
15	log (N)	log (σ_t)	log (σ_b)	-	0.27	0.70
16	log (N)	σ_t/ σ_b	-	-	0.28	0.69
17	log (N)	log (σ_t/ σ_b)	-	-	0.26	0.70
18	log (N)	σ_t	-	E	<0.25	0.79
19	log (N)	log (σ_t)	-	E	<0.25	0.79
20	log (N)	σ_t	σb	E	0.29	0.68
21	log (N)	log (σ_t)	log (σ_b)	E	0.28	0.68
22	log (N)	log (σ_t/ σ_b)	-	E	0.29	0.68

- General form of equation is $y = ax^3 + bx^2 + cx^1 + d$
- N = fatigue life
- μ_t = initial strain
- μ_b = breaking strain
- σ_t = initial strain
- σ_b = breaking strain
- E = flexural modulus
- Breaking strain for the purposes of analysis is defined as strain at 95% breaking load from nine months flexural strength testing

From the results above, a better correlation can be established between strain-based equations and fatigue life (Austroads 2010) where analysis numbers 3, 9, 10 and 11 show the highest least square regression, i.e. R2 values of more than 0.3. More significantly, contrary to the current numerical model presented earlier in this section, the elastic modulus is seen to provide little to nil statistical improvement in fatigue prediction, suggesting that the elastic modulus does not play a significant role in fatigue life prediction. This is consistent with the other international models shown in Table 3.3 (Austroads 2010).

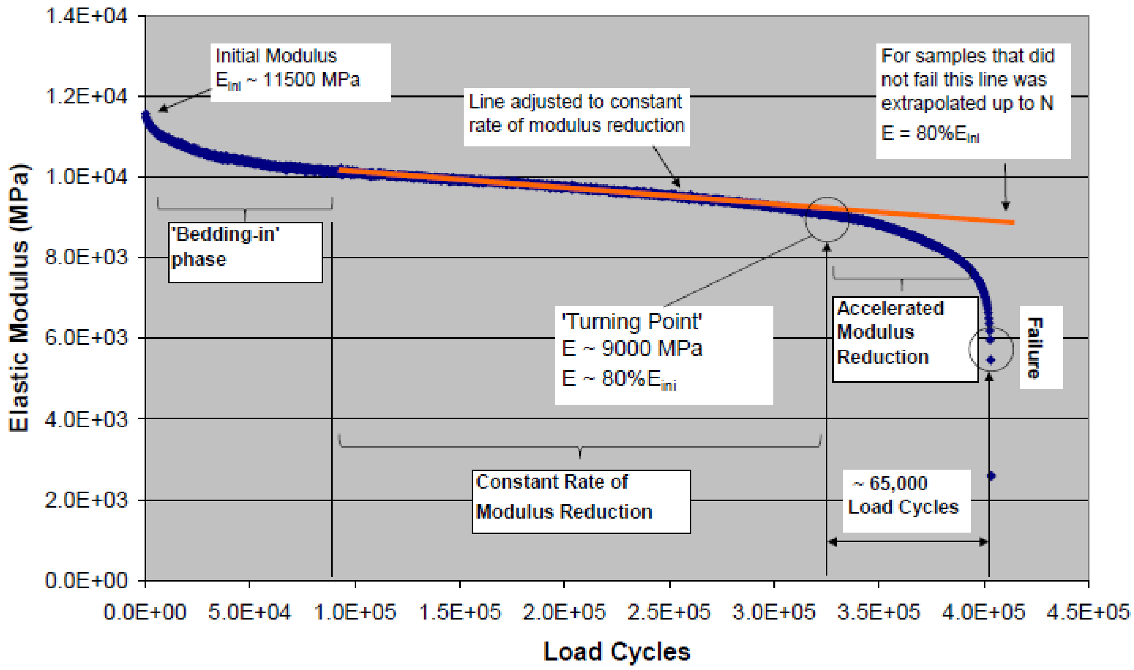


Figure 3.9: Typical modulus variation during fatigue tests (Austroads 2010)

Subsequently, a new fatigue prediction model has been developed, as shown in the equation below:

$$N = B\left[\frac{\mu\varepsilon_{break}}{\mu\varepsilon}\right]^{A} \qquad (3.12)$$

where, N = number of load cycles to failure

$\mu\varepsilon$ = initial elastic strain (microstrains)

$\mu\varepsilon_{break}$ = initial elastic strain (microstrains)

A = damage exponent

B = coefficient

By extrapolating the results of ARRB tests against the adopted model, the results of their analysis are as shown below:

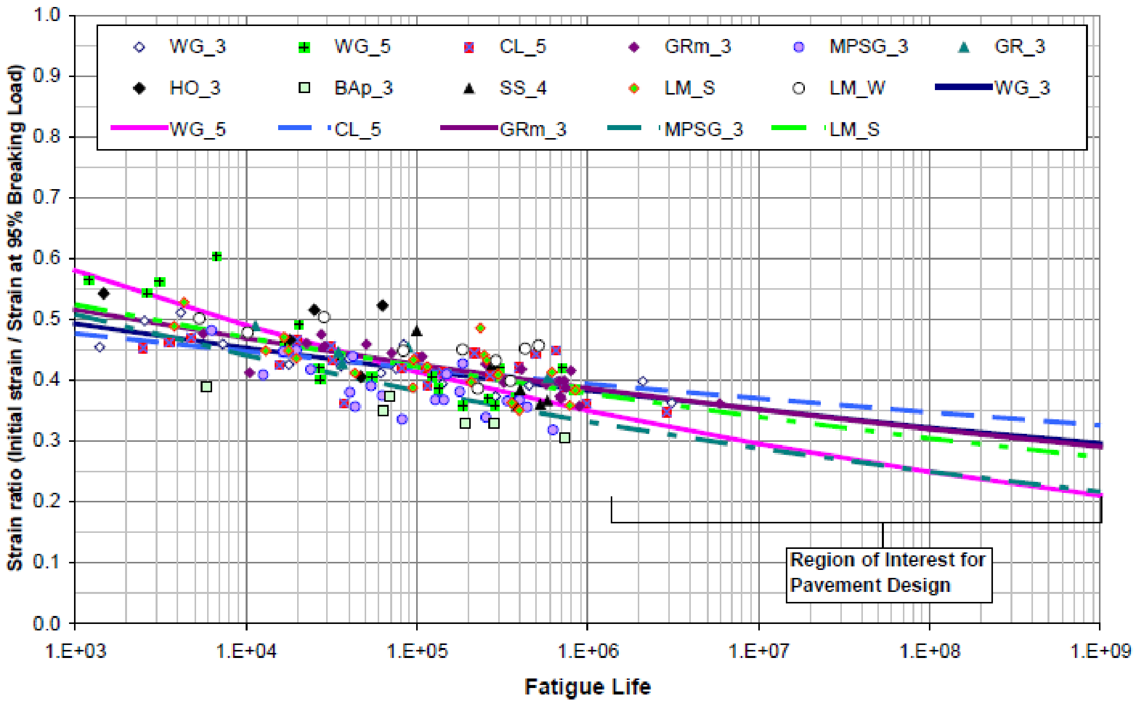


Figure 3.10: Relationship between strain ratio and fatigue life (Austroads 2010)

Figure 3.10 above shows that a distinct relationship between fatigue life and a strain ratio concept is evident. This relationship is the pivotal output of the research, and has been further verified against the ALF, forming a promising new fatigue relationship. A simplification of the design equation is as follows:

$$N = B\left[\frac{\mu\varepsilon_{break}}{\mu\varepsilon}\right]^{A} \quad (3.13)$$

The model is yet to be adopted in Australia due to its infancy, but it is of great interest to this dissertation in that it acknowledges a more bona fide basis to explain fatigue, rather than the observational behaviour that is unrepresentative of the true nature of the material.

In summary, the numerical models of Austroads fatigue life of cemented materials to date have been based on parametric studies that were built upon laboratory data that may not have been compatible in the Australian context. As shown in the Table 3.3, all fatigue models typically adopt a ratio between applied stress or strain and stress or strain at break. This concept can be traced to the rudiments of fracture and damage mechanics, as discussed in the subsequent section.

3.4.2 Fracture and damage mechanics for Cement Treated Basecourse

In the study of continuum damage mechanics, fatigue is characterised by the accumulation of damage on a micro-scale. Similarly, the quasi-brittle nature of cement treated basecourse means that the material undergoes damage in the form of nucleation of voids formed from the coalescence of micro-cracks existing within the cement matrix (Balbo and Cintra 1996). This fracture process involves the creation of new surfaces in the material, a phenomenon much better described by energy principles than by classical mechanics (Alliche and François 1992; Lee et al. 1997).

Based on the strain equivalence principle (Sidoroff 1981; Lee et al. 1997), the concept of damage can thus be represented by introducing a damage function, derived on the basis that the virgin material and its continuum model must contain equal strain energies when subjected to similar global displacements. It is represented based on the degradation of the elastic modulus which results in a lowered capacity to store strain energy, i.e.

$$\overline{E} = E_0(1 - D) \tag{3.14}$$

Where, E = effective modulus

E_o = initial modulus

D = damage factor

This concept of damage and effective modulus can be graphically represented, as shown in Figure 3.11 below (Lee et al. 1997):

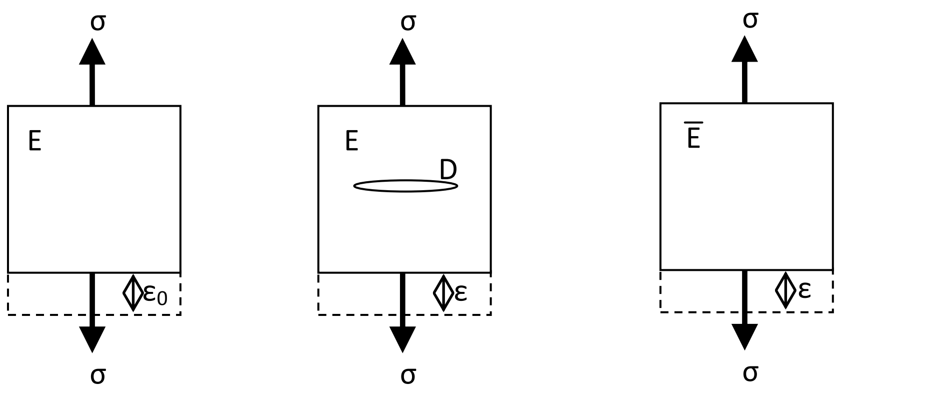


Figure 3.11: Equivalent damage model based on strain equivalence (Lee et al. 1997)

Based on Figure 3.11 above, the stress strain behaviour of a damaged material may therefore be represented as follow,

$$\varepsilon = \frac{\sigma}{\overline{E}} = \frac{\sigma}{E_0(1-D)} \tag{3.15}$$

As shown in equation above, prior to the onset of damage, i.e. D = D_o= 0, the linear elastic postulate is observed. The onset of damage is also termed the linear elastic limit or the endurance limit, again complementing the non-linear behaviour of cement treated basecourse (presented in Section 3.3).

To understand the mechanism of the onset of damage, the fictitious model developed by (Bazant 2002) is referred to. A model of the damage initiation is shown in Figure 3.12 (adapted from (Gdoutas 2005)).

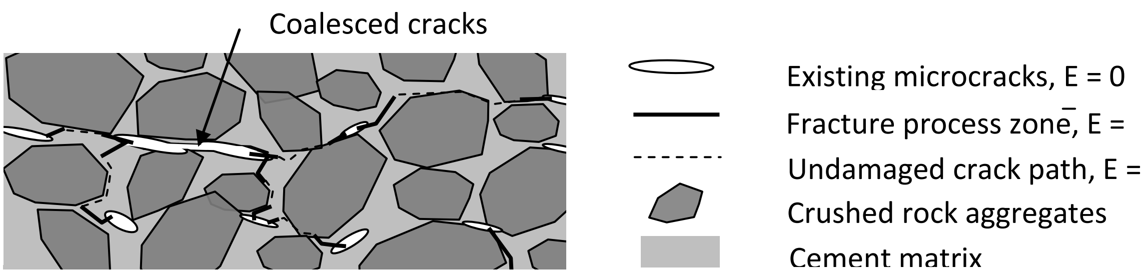


Figure 3.12: Damage Initiation of cement treated basecourse (Gdoutas 2005)

In the body of the cement treated basecourse, existing voids are present within the cement matrix, as shown in Figure 3.12 above. These existing voids occur naturally, due to the hydration process of the material or as a result of deficiencies during construction, e.g. compaction effort, aggregate gradation, water ratio, mixing consistency, etc. These voids coalesce upon continuous traffic loading in excess of the elastic limit, which propagates the crack distance. A typical crack tip within a cement matrix on a micro-scale is shown in Figure 3.13, where the mechanism for damage evolution occurs when the applied stresses exceed the closure stress at the tip of the existing micro-cracks.

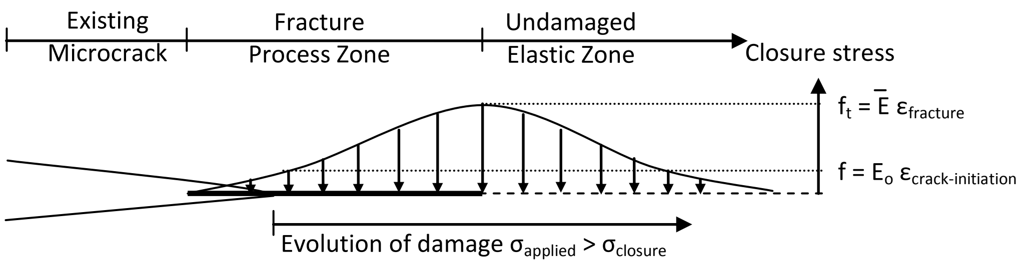


Figure 3.13: Micro-cracking fictitious crack model (Gdoutas 2005)

The coalescence of micro-cracks typically traces the interfacial transition zone (ITZ) between aggregates and the cement matrix as it forms the weakest link in the composite material (Taylor 1997) producing an array of cracks within the composite matrix.

The array of micro-cracks translates to permanent damage being inflicted onto the cement matrix and an increase of fractured area will be observed, resulting in a reduced continuous area for the distribution of stresses, and subsequently a reduction in the stiffness of materials. This explains the cease in linearity of the cement treated basecourse where a certain magnitude of stress or strain load is applied, as discussed earlier in Section 3.3.

The strain measured at the onset of damage, denoted by the termination of elastic linearity, is observed as a material constant (Kaplan 1963; Karihaloo and Fu 1990; Karihaloo et al. 1993; Austroads 2010). Williams (WIlliams 1986) showed that the proportionality of stress strain relationships generally ceases at 30% of the maximum load, a simplified assumption which also conforms to the 0.33f_t estimated for concrete tensile cracking and works by the Department of Transport (1986), as previously shown in Section 3.3. It is however, recognised that the methodology used to measure the cease in proportionality of the stress strain curve requires sound judgement and the precise measurement of strain (Kaplan 1963).

In terms of fatigue, the phenomenology of fatigue generally observes a power law such as that first noted in classic fracture mechanics by the Paris Law:

$$\frac{da}{dN} = C(\Delta K)^m \tag{3.16}$$

where,	da/dN	=	infinitesimal crack length growth per cyclic load
	ΔK	=	stress amplitude factor or difference between maximum and minimum stress intensity
	C	=	material constant
	m	=	material constant

The stress amplitude factor can be characterised based on based on a crack size, a, which gives,

$$K = \sigma Y \sqrt{\pi a} \tag{3.17}$$

where,	σ	=	applied tensile stress
	Y	=	material constant
	a	=	crack length

To find the fatigue life, the equation is rearranged and integrated from 0 to number of cycles at failure, N_f which corresponds to the development of crack length from the onset of cracking to the critical crack length.

$$\int_0^{N_f} dN = \int_{a_i}^{a_c} \frac{da}{C\left(\sigma Y\sqrt{\pi a}\right)^m} \quad (3.18a)$$

$$N_f = \frac{1}{C\left(\sigma Y\sqrt{\pi a}\right)^m}\int_{a_i}^{a_c} a^{-\frac{m}{2}}\, da \quad (3.18b)$$

Where, a_i = initial crack length

a_c = critical crack length

From the classic fatigue law above, similarities can be drawn between the observations concluded from the recent work by Austroads (2010) where fatigue is characterised by a function of the initiation of damage (initial strain) and the critical damage (breaking strain).

Since the effective stress of damaged quasi-brittle materials can be represented by strain, evolution of fatigue damage can thus be explained by a strain dependent function from the onset of crack to failure, i.e.

$$N_f = B\int_{\varepsilon_i}^{\varepsilon_c} f(\varepsilon) \quad (3.19)$$

Where ε_i = minimum damaging strain

ε_b = max tensile strain at break

This relationship has been successfully used to characterise the fatigue life of cemented materials based on a power function of strain. To name a few instances of use: Kaplan (1963), Suaris and Fernando (1990), current Austroads model (2008), Alliche and François (1992), Karihaloo and Fu (1993), and of course the works of the AARB, shown earlier (Austroads 2010).

This is in contrast to the ultimate stresses and strains which are dependent on the type of test undertaken (Karihaloo and Fu 1990), again dismissing the classification based on ultimate stress measurements such as UCS.

In summary, the above literature review has pointed out that, unlike the idealised model, the fatigue mechanism is governed by indefinite crack profiles, formed from a distribution of micro-cracks propagating within the cement matrix in pavement which has been subjected to repeated loads. This phenomenon however, can be explained based on a strain ratio function. Nevertheless, the effective measurement of fatigue life and its associated strain is an arduous task. The subsequent section therefore presents current practices and the limitations of cement treated basecourse fatigue testing.

3.4.3 Testing of Fatigue Life of Cement Treated Basecourse

Because of the immaturity of fatigue characterisation of cement treated basecourse and its theoretical development as discussed in the previous chapter, the testing of the fatigue life of cement treated basecourse is still in its experimental stages. This is due to the complexity of characterising fatigue life which lies not only in the numerical modelling to be considered, but also includes the various testing methods available.

As the distinction with cement treated basecourse material is that it is essentially tensile, the two predominant forms of testing are the indirect tensile configuration and the flexural beam configuration. Neither configuration provides a realistic representation of actual pavements. However, Yeo et al. (Yeo 2008) have undertaken an extensive study to compare the flexural properties of materials to characterise the behaviour of cemented basecourse under repeat loadings. In the report, the flexural beam test and the conventional cylindrical indirect tensile strength test (refer Section 3.3) were compared against readings from the Accelerated Loading Facility (ALF) located in Dandenong, Victoria. The flexural beam test was found to exhibit comparatively closer readings to the in-service conditions shown by ALF (Austroads 2008).

The subsequent section therefore presents a discussion of flexural beam theory and a review of the testing methodology of flexural fatigue testing.

3.4.4 Flexural Beam Theory

Flexural properties are considered characteristic of the structural system of pavements since pavements essentially “flex” under traffic loads. They are generally accepted as the most representative test for assessing the tensile capacity of concrete pavements (Griffith and Thom 2007; Yeo 2008).

Flexural properties of materials are derived specifically from the flexural beam test and explain the stress-strain relationship of beam specimens undergoing bending stresses. Amongst the typically measured parameters are the

- modulus of rupture (also known as the flexural strength), i.e. the maximum allowable force applied onto the beam prior to rupture
- the tensile strains at the bottom fibres
- the flexural modulus, i.e. the rate of change of tensile strain at bottom fibres under stress.

The modulus of rupture of a material is a unique measurement of a specimen’s bending capacity and does not represent the true tensile capacity of a material. It is influenced by the thickness of the material as shown in Figure 3.14 below (Griffith and Thom 2007; Thom 2010).

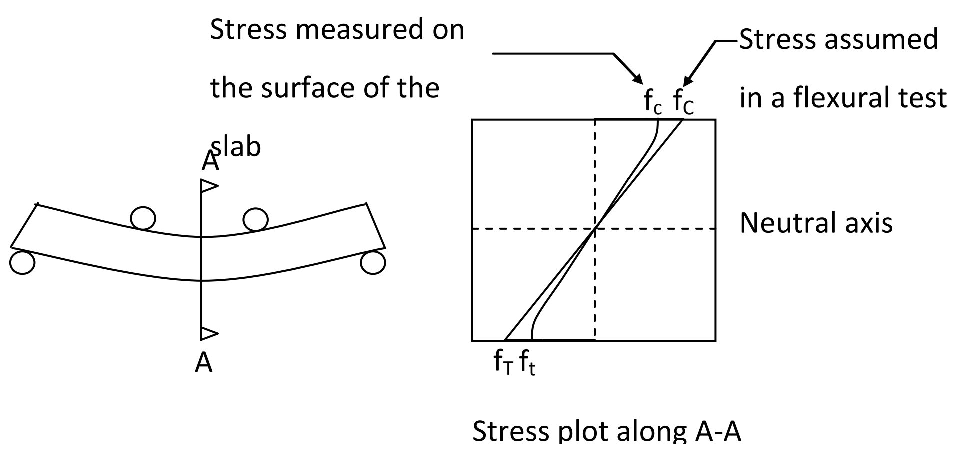


Figure 3.14: Flexural four point bending (Griffith and Thom 2007; Thom 2010)

Based on the assumed flexural stress, the bending moment,

$$M = \frac{\sigma h^2}{6} \tag{3.20}$$

By taking into account the non-linear stress-strain relationship presented in Figure 3.2 and the free body diagram in Figure 3.14, the relationship between flexural strength and tensile strength can be determined, as shown below (Thom 2010):

$$\sigma_f = \sigma_t \frac{4(0.5h - z)^3}{p\left[h^2(0.5h + z)\right]} \tag{3.21}$$

Furthermore, the elastic modulus can be determined based on the shear effect and moment response of the beam in bending, as shown below (Thom 2010):

shear effect,

$$\delta_{shear} = \frac{PL}{6AG} \tag{3.22}$$

where	A	=	beam cross-sectional area
	G	=	shear modulus [E/2(1 + v)]

moment effect,

$$\delta_{moment} = \frac{23PL^3}{1296EI} \tag{3.23}$$

therefore,

$$\delta_{moment} = \frac{PL}{bh\delta}\left[\frac{(1+v)}{3} + \frac{23L^3}{1296EI}\right] \tag{3.24}$$

Beyond that, the fatigue response of a specimen can be derived by applying repeated loads where laboratory results are extrapolated to predict in-service conditions. The degree of fatigue susceptibility is represented by the Load Damage Exponent (LDE), i.e. the exponent of the relationship between the number of repeated loads and the pavement's mechanical properties, as shown in the equation below:

$$N = \left(\frac{K}{b}\right)^{LDE} \qquad (3.25)$$

where k = constant

b = mechanical property

Suffice it to say, the database for fatigue data on specimens is limited, due to the countless challenges of tests. Even when the configuration is selected, the parameters to run the test to provide meaningful results are countless and complex. These considerations are summarised as follow:

i. Three Point vs. Four Point

The execution of flexural beam tests can be undertaken with either a 3-point or a 4-point bending test setup. In Australia, and common to concrete specimens, asphaltic materials typically use the four-point bending test setup in fatigue related testing regimes. The four point bending test has the added advantage of providing a uniform stress distribution between the two loading points (Thom 2010). In contrast, the three point bending setup has the maximum stress concentrated locally, below the loading point, exposing only very limited areas of the specimen to maximum loads. This allows a wider region of controlled strain and stress to be applied to the specimen. The four point bending setups used in this experiment are shown in Figure 3.15 below.

ii. Duration of Test

Austroads (2010) have identified that the effective fatigue life period to be considered in the design of cemented basecourse should start within 1 x 10^6 cycles. In order to physically attain these results, considerable time is required for the test. The duration therefore limits the practical determination of materials. In order to prevent appreciable damage occurring, 1200 cycles has been selected as providing sufficient data to assess the dynamic effects of the fatigue test.

iii. Numerical Model

Road authorities around the world use various fatigue models which take fatigue as a function of stress, strain, stress ratio or strain ratio (Austroads 2010). The model adopted currently in Australia is based on the strain at the bottom of the cement treated basecourse layer (Austroads 2008). This is not directly relevant to the test, but dictates the loading type and magnitude, as discussed below:

iv. Loading Type and Magnitude

Given the different numerical models, the applied load may be a constant strain or constant stress. Realistically, the load varies, based on the thickness of the overlying pavement and the type of vehicles passing over the pavement. The typical strain value, calculated using CIRCLY by Austroads in their previous assessment of pavements with cemented basecourse, is 75με. This typically represents 65% of the strain at the break in the cemented basecourse.

v. Loading Shape

Haversine load patterns are believed to be the most representative of traffic loading and have therefore been adopted.

vi. Rest Periods

Rest periods are the pauses between successive loads, and they are important in the analysis of asphaltic materials, as “healing” can potentially occur between applied loads. However, it is believed by this author that this is not applicable to cemented basecourse materials, as the rehydration of broken bonds takes much longer and requires sufficient moisture to occur. No rest periods have been allowed with loads applied successively. This also assists in shortening the testing duration.

vii. Specimen Size

The size of quasi-brittle materials plays an important role in the propagation of cracks, a mechanism closely related to fatigue development (Bazant 2002). The size of specimens is however dictated by the testing rig used. Specimens measuring 390mm (L) x 63mm (W) x 50mm (D) are prepared

viii. Laboratory Compaction / Specimen Preparation

There is no current standard methodology to fabricate beams used for flexural beam tests. However, in previous work by Austroads (2010) fabricated slabs were created using a BP compactor and the slabs were then cut to the required size. An in-house developed compaction method, utilising the modified Proctor compactor, was used for this test. The target MMDD is 2.35 t/m3.

ix. Rate of Load Application

The rate of load application represents the dynamic effects caused by the different speeds of vehicular traffic on the road. There is very limited literature available which discusses the effects on stiffness by the varying of this parameter.

x. Test Termination

The distinction of failure during beam fatigue testing is not defined as the ultimate fracture, but instead as a reduction in stiffness, typically to 50% of its original value. At termination, the cracking of tensile fibres at the base of the specimens is not distinctively clear. Instead, a distribution of cracks occurs throughout the member.

With reference to Figure 3.6, the distinction of equivalent granular phase i.e. the end of the effective fatigue life of the specimen is typically estimated to be at a point where the modulus of the material reduces to become 50% of its original value.

Figure 3.15: IPC Global four point bending test apparatus

3.5 Summary of Engineering Properties

Summing up the points discussed, this section has presented the different engineering properties relevant to the characterisation of cement treated basecourse material. This includes the unconfined compressive strength and indirect tensile strength. Nevertheless, the flexural fatigue behaviour of cement treated basecourse is the primary characteristic that is relevant to cement treated basecourse. The classical Paris Law of fatigue, derived from fracture and damage mechanics, shows similarities with previous works on fatigue characterisation of cement treated materials which suggest that a strain-based equation provides some reliability in characterising fatigue response. The limitations and complexity of the tests undertaken and the non-linear behaviour of cement treated basecourse are acknowledged here.

3.6 Microstructure of Cement Treated Basecourse

Although the principal investigation of the study aims to identify engineering applications for cement treated materials, a qualitative conceptualisation of the microstructure of cement treated basecourse allows the opportunity to correlate fundamental properties with the behaviour of the material (Diamond 2004).

The existence of a relationship between its microstructure and the macro behaviour of a material has been long recognised. For example, Taylor (1997) established that the strength of cement is not defined by empirical relationships to physical properties alone but that an understanding is required of how the material is held together and what its mechanisms of failure are.

Hence, as part of the characterisation and discovery of the fundamental properties of cemented pavement materials, this section is pivotal in that it investigates the internal constructs of cement, and it includes an investigation into cement at its microscopic level, to derive the fundamental properties of cement treated basecourse. This will allow the identification of correlations between the fundamental properties of cement treated basecourse material and the behaviour of the material, i.e. shrinkage and moisture sensitivity.

This section first provides a brief introduction to cement chemistry in order to illustrate the different constituents of cement, and the hydration process. Subsequently, comprehensive discussions are presented regarding the implications around shrinkage and moisture sensitivity.

3.6.1 Introduction to Portland Cement Microstructure

Microstructural investigation of pavement materials is very limited but not absent. The CSIR International Convention Centre of Pretoria has undertaken extensive observational studies of the microstructure of pavement materials, recognising the pivotal roles of material properties at the micro level through to the loading response at the macro level (Steyn 2007; Mgangira 2008).

Current microstructural studies of cement and its constituents are focused on its application in concrete structures (due to its vast application), or otherwise in its pure paste form, as the presence of aggregates complicates the studies (Taylor 1997). Regardless, (Diamond 2004) has concluded that the microstructural characteristics of HCP within concrete do not vary significantly from those found in pure cement paste.

The predominant focus of studies so far has been on assuming that a relatively significant amount of cement paste is present, which is less often than not in the case of cement treated pavement materials. There is a general lack in the relevant literature, regarding investigations into the microstructural interactions of low cement content and aggregate concoctions, in materials such as cement treated basecourse.

This section discusses the existing literature on the microstructural morphology of cement grains and their relationships with a range of aggregates at different phases of service life with a focus on the effect of the relationship on visible properties.

3.6.2 Portland Cement Microstructure and Its Constituent Phases

Portland cement is made by heating limestone and clay at 1450°C, initiating partial fusion to form nodules of clinker. The clinker is then further mixed with calcium sulphate and milled to form cement powder. It is a formation of four major constituents or phases, viz. alite, belite, aluminate and ferrite.

The following subsections expand further on each of the phases to provide a concise understanding of their chemical construct and main functionality in cement.

i. Alite

Alite, which is chemically known as tricalcium silicate (Ca_3SiO_5 or C_3S), forms the most important constituent of normal Portland cement, constituting 50% - 70% (in mass) of all normal Portland cement. It reaches its dominant strength in the development constituent phase, primarily at the 28 day hydration stage. Pure alites contain 73.7% of CaO and 26.3% of SiO_2 displaying hexagonal crystals in cross-sections at sizes of about 150µm (Taylor 1997).

ii. Belite

Belite or dicalcium silicate (Ca_2SiO_4 or C_2S) is built from Ca^{2+} and SiO^{4-}_4 ions making up 15 – 30% of normal Portland cement mass (Taylor 1997). Belite contributes primarily to strength at later stages of curing (after 28 days) as it reacts significantly more slowly to water. One year after curing, the strength contributed to belite is equal to that of alite. It is generally rounded with crystal sizes ranging from 5µm to 40µm.

iii. Aluminate

Aluminate or tricalcium aluminate (C_3A) comprises 5 – 10% of normal Portland cement mass but contrary to alite and belite, it has a cubic morphology built from Ca^{2+} ions and rings of six AlO_4 tetrahedra. Pure aluminates contains 62.3% CaO and 37.7% Al_2O_3, however substantial proportions of Al can be replaced by other ions, primarily Fe^{3+} and Si^{4+} (Taylor 1997). Aluminate is considered to be an interstitial or matrix phase since it predominantly forms between silicate crystals whilst acting as a binder (Strutzman 2004).

iv. Ferrite

Ferrite or tetracalcium aluminoferrite (Ca_2AlFeO_5 or C_2F) is a cement constituent that can be substantially modified in composition by variation in Al/Fe ratio and ionic substitutions (Taylor 1997). Its reaction in water is inconsistent due to its variability in composition. It represents 5-15% of mass of normal Portland cement and like aluminate is deemed to be an interstitial or matrix phases as it forms between silicate crystals and binds them. (Strutzman 2004).

3.6.3 Summary of Cement Constituent Phases

Each phase has unique characteristics that contribute to the overall development of cement. Figure 3.16 shows a backscattered image at 100µm zoom of a typical unhydrated cement clinker section. The percentage mass distribution of each phase is evident, as shown in Figure 3.16, where alite is seen to be the dominant phase in terms of mass. Alite and belite form solid masses after hydration while aluminate and ferrite form interstitial matrices between the solid phases.

It is inferred from this that the microstructural construct of cement can be translated into its mechanical properties, as suggested by various authors (Taylor 1997).

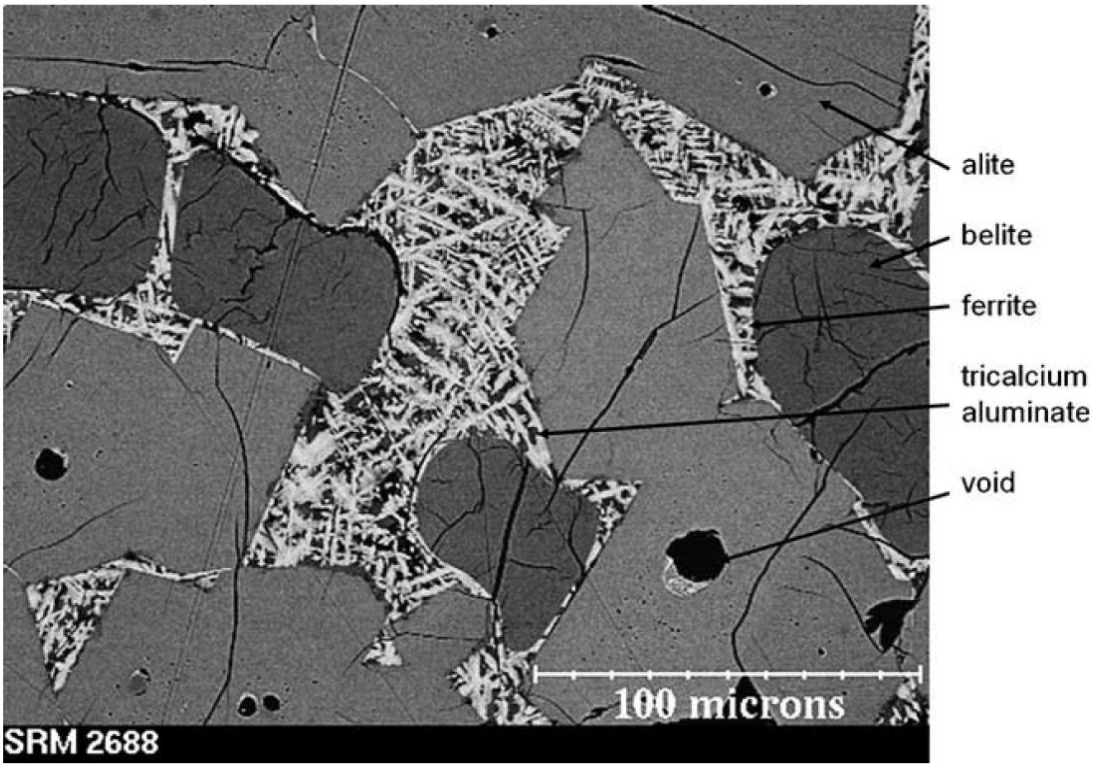


Figure 3.16: SEM backscattered electron image of cement paste (Taylor 1997)

When in contact with water, each of the phases undergoes the hydration process, forming the binding agent used in concrete and, more importantly, used in relation to this study of pavement material stabilisation. The following section discusses in detail the cement hydration of Portland cement.

3.6.4 Portland Cement Hydration

Hydration refers to the whole altering process of anhydrous cement when it is mixed with water. The essence of cement hydration is the reaction of each anhydrous cement phase leading towards the formation of hydrates, predominantly Calcium-Sulphate-Hydrate, C-S-H and Calcium Hydroxide, $Ca(OH)_2$. The hydrates form solid volumes that bridge intermittent voids between cement grains and aggregates, forming a solid mass; the fundamental building blocks which allow the development of the tensile strength achieved from stabilising pavement materials.

These hydrate constituents are also a principle reactant in determining the durability of the cement matrix, as covered further in Section 3.7.

The understanding of the hydration processes of cement is crucial in establishing its fundamental properties and the extent of its role in engineering. The following section therefore presents the current understanding of cement hydrationby providing a detailed account of the chronological development of cement hydration and a discussion of its hydration products.

3.6.5 Chronological Development of Hydrated Cement

The principal investigation of the hydration processes of cement is a chronologically based study, where changes in its microstructure over time are related to its subsequent properties and characteristics. Jennings et al. (1981) discerned three main stages of product formation in C_3S which can be related to cement hydration, i.e. early stage (first 4 hours), middle stage (4 hours to 24 hours) and late stage. The development of the hydration of a cement grain has been graphically illustrated by Scrivener (Taylor 1997), as shown in Figure 3.17. Each of the stages in the hydration process are discussed in the subsections below.

i. Early Stage

Early stage products typically consist of foils, flakes or honeycombs exfoliated from cement grains, where studies by HVTEM (High Voltage Transmission Electron Microscopy) have shown high dilutions of a gel membrane forming over the grains after mixing (Taylor 1997). Within approximately 10 minutes, stubby rods of AFt phase approximately 250nm long and 100nm thick are evident, as shown in Figure 3.17(b). Wet cell studies indicate that the development occurs both on the surface and in the surrounding area (Taylor 1997), nucleating in the solution and outer surface layer of gel. When dried, AFt crystals shrink back to the surface of the cement grains.

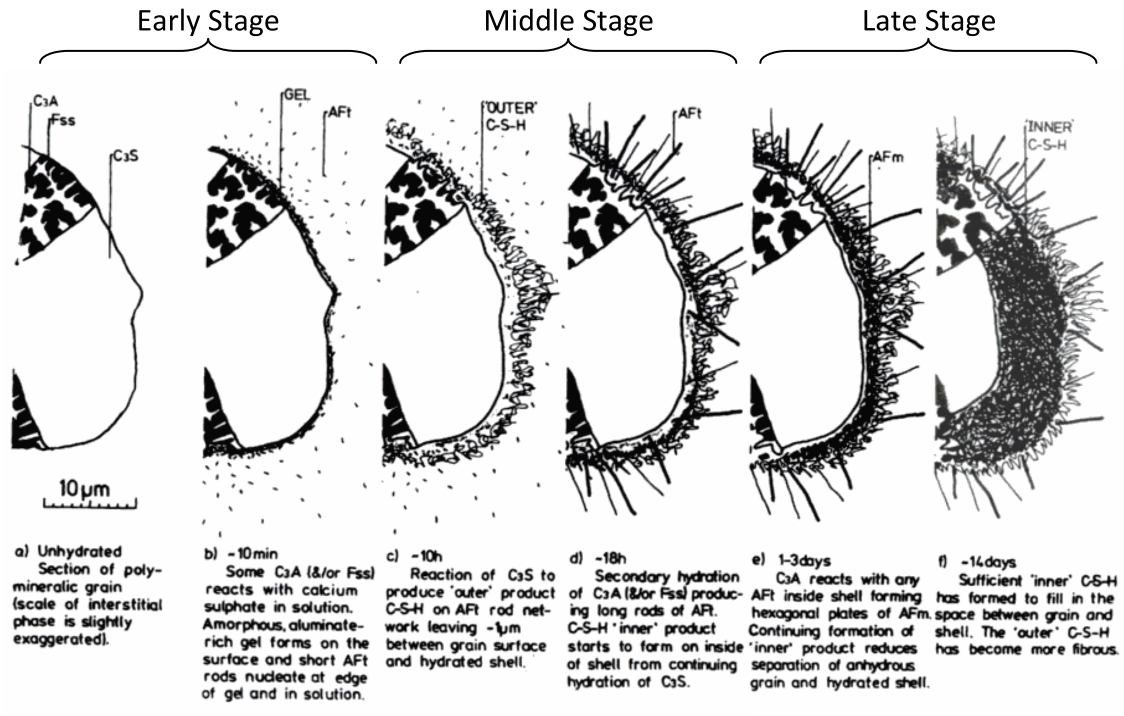


Figure 3.17: Development of microstructure during the hydration of Portland cement (Taylor 1997)

ii. Middle Stage

At the middle stage, 30% of cement has reacted, characterised by the expedited formation of C-S-H and CH. Strong heat evolution is also evident. Studies using wet cells show that the undried C-S-H has a filmy, foil-like morphology (Taylor 1997), which on drying changes to give fibres (Type I C-S-H), where space is freely available, or, honeycombs or reticular networks (Type II C-S-H) where it is more restricted.

The C-S-H forms a thickening layer around the cement grains (Taylor 1997) which engulfs and perhaps nucleates on the AFt rods (Fig 7.3c). A significant amount has formed by 3h, and the grains are completely covered by 4h. The shells grow outwards; by about 12h they are some 0.5-1.0 μm thick, and those surrounding adjacent grains are beginning to coalesce. At this stage, called the cohesion point, fracture through the shells begins to supplant fracture between them. It coincides with the maximum rate of heat revolution and corresponds approximately to the completion of setting.

However, from an engineering point of view, the paramount finding was a correlation between the interconnected shells with particle size distribution, which thus determines the mechanical properties of cement (Taylor 1997).

iii. Late Stage

At late stages of cement hydration, the permeability of shells has diminished, leading towards the development of C-S-H inside cement grains.

Grains smaller than about 5μm appear to react completely before the end of the middle period, and before a great deal of material has been deposited inside the shells; many that originally contained the aluminate phase are empty. The outer product from such grains is often absorbed into the shells surrounding adjacent, larger grains. With larger grains, the spaces between shell and core fill up, and by about 7 days they have disappeared.

At this stage, the shells are typically some 8 μm thick and consist mainly of material that has been deposited on their inner surfaces. The separation between shell and core seen on polished or ion thinned sections rarely exceeds about 1 μm. Hardley grains, i.e. separated shells up to at least 10μm across, and sometimes completely hollow, have been observed on fracture surfaces (Taylor 1997). After the spaces between shells and cores have filled up, reaction is slow and in contrast to that occurring earlier, appearing to concur with a topochemical mechanism (Taylor 1997).

3.6.6 Relationship of Hydration and Strength Gain

Based on Figure 3.17 above, the chemical reaction of cement grains can be related back to strength gain vs. time, a relationship that has been well established by various works. Figure 3.18 below shows the tensile strength gained vs. time for cement content of materials treated with 2%, 3% and 4% binder (Chakrabati and Kodikara 2007).

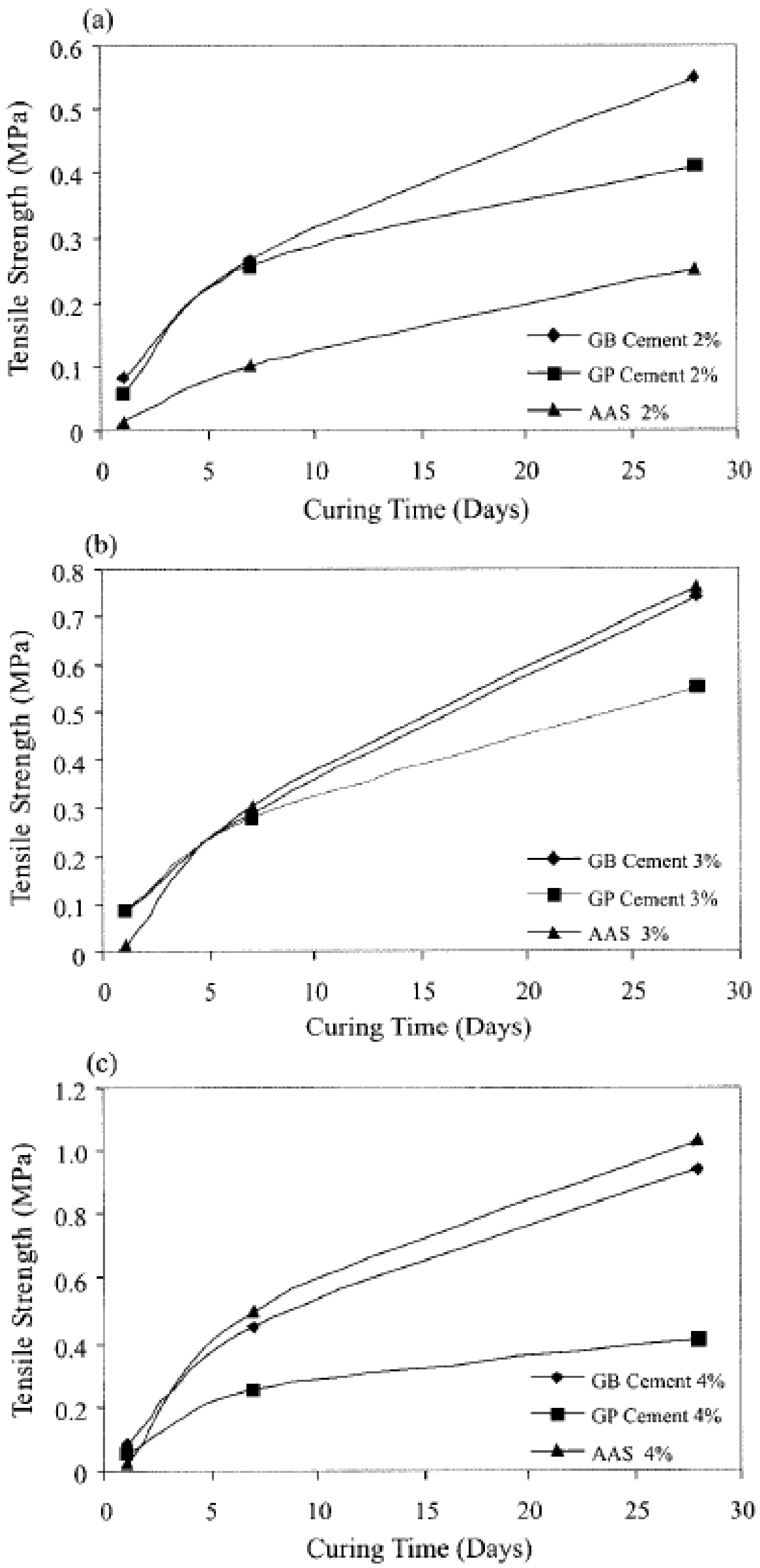


Figure 3.18: Gain in tensile strength with curing time (Chakrabati and Kodikara 2007)

As predicted, after 7 days of hydration, i.e. during the late stage, a significant decrease in the rate of strength gain is observed, corresponding to the chemical behaviour shown above. The hydration process will result in the formation of new constituents, as discussed in the following section.

3.6.7 Hydrated Cement Constituents

As discussed in Section 3.6.4, the predominant hydrate products of cement are calcium sulphate hydrate (C-S-H) and calcium hydroxide ($Ca(OH)_2$). Notwithstanding, other less dominant hydration products of cement include ettringite and monosulphates.

The following subsections discuss in further detail each of the constituents as well as giving general consideration to the microstructure.

Calcium Sulphate Hydrate, C-S-H

C-S-H refers not only to a single substance but to a collective deposits of quasi-amorphous particles and masses made up of the hydration products alite and belite, i.e. calcium silica and water. Chemical and physical consistency is limited within C-S-H due to the different processes of hydration, as is evident by varying grey levels of backscattered electron microscopy images (see Figure 3.16).

C-S-H has a layered structure with pores ranging in size from macroscopic to enlarged interlayered spaces of nanometre dimensions, which means that the definition of water content for cement is related to a specified drying condition (Taylor 1997).

Powers and Brownyard (Taylor 1997) showed that C-S-H in a cement paste develops only if sufficient space is available to permit it to be accompanied by a defined proportion of pore space. This shows that porosity and water cement ratio is paramount to the hydration processes of cement. It is presented that there is a w/c ratio below which complete hydration is impossible.

Calcium Hydroxide, $Ca(OH)_2$

Calcium Hydroxide or CH is yet another dominant hydration product of cement which closely resembles the characteristics of C-S-H. It requires some effort in discerning when observed on a backscattered image, as the grey levels between C-S-H and CH can be almost similar. CH is often brighter and composed of irregular masses of different shapes and sizes contrary to the euhedral crystal forms of C-S-H.

CH is an important product of cement hydration, distinguished on BEM by being slightly brighter than that of CSH, although distinction sometimes requires close examination. (Diamond 2004). CH within cement pastes usually appears as irregular masses of various sizes, rather than as euhedral crystals.

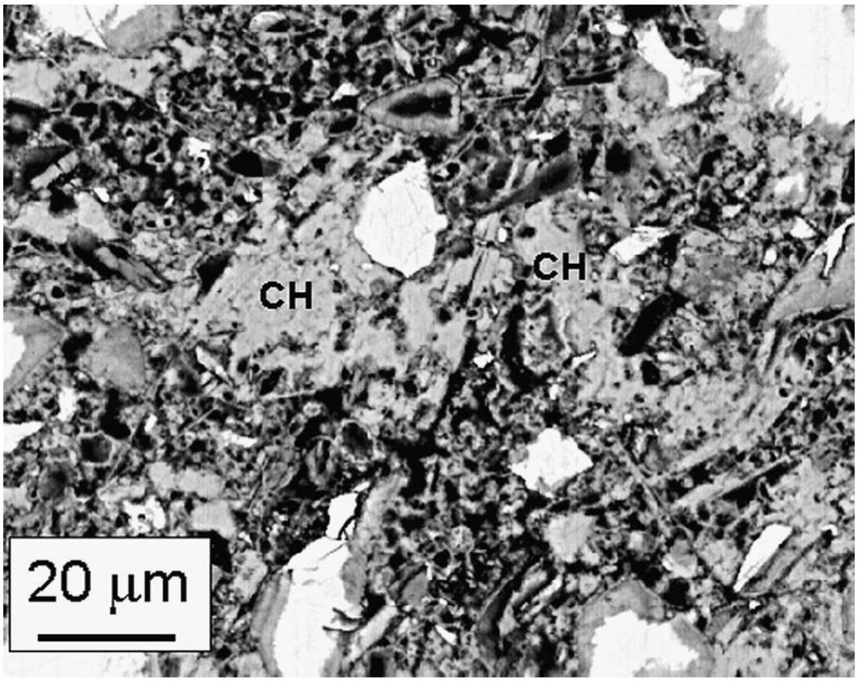


Figure 3.19: Irregular calcium hydroxide deposits in a 7-day old w/c 0.45 paste (Diamond 2004)

Ettringite and Monosulfate

The more minor products of cement are calcium aluminate sulphate hydrates known as ettringite and monosulphate. These products are typically isolated in pockets within water-filled spaces interconnected with C-S-H and CH or within fissures and areas surrounding aggregates.

Under backscattered imaging, ettringite usually shows a characteristic shrinkage-induced pattern of curved cracks, resembling a “tiger stripe” morphology while monosulphate masses display straight “cleavage-like” shrinkage features.

3.7 Durability, Water Damage in Pavements

As roads have a typical service life of 30 years, there is a pivotal role in ensuring that roads are durable, which may seem to be an obvious conclusion. Nevertheless, the task of maintaining the durability of roads is a difficult one, as roads are exposed to the harsh effects of weather which continuously degrades the serviceability and structural integrity of the pavement. Among the elements which affect the durability of pavements is water, despite its importance during construction for compaction and tyning works (Thom 2010).

Water within voids in soils comprises three phases surrounding soil particulates, i.e. tightly bound, loosely bound, and free viscous phases, as shown in Figure 3.20 below (Guthrie et al. 2001). A difference in polarity at the edge of the soil particles surface attracts water molecules. The degree of binding is caused by the electrical capture of the soil particles, thus the phase furthest from the soil forms the viscous phase or free water phase where capillary movement of water occurs. As depicted, the phase closest to the soil particle measuring approximately 0.002 µm is tightly bound to the soil particles, followed by a 0.004 µm loosely bound phase.

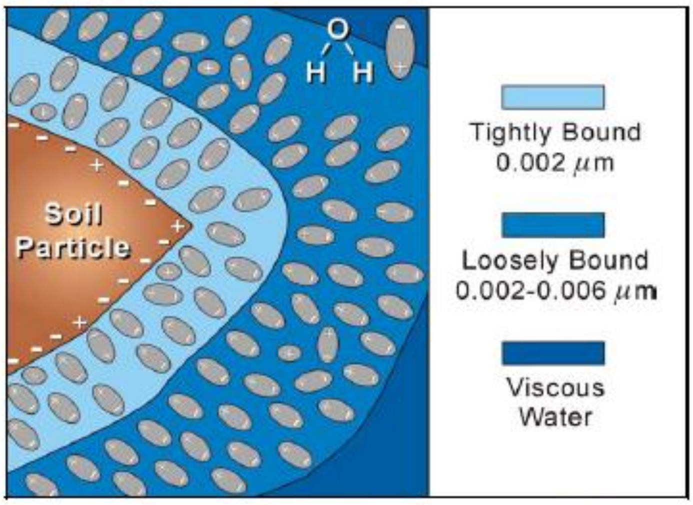


Figure 3.20: Structure of water molecules surrounding soil particles

(Guthrie et al. 2001)

Subsequently, under various degree of saturation, the behaviour of water in pores can be seen in the following Figure 3.21 (Lu and Likos 2004):

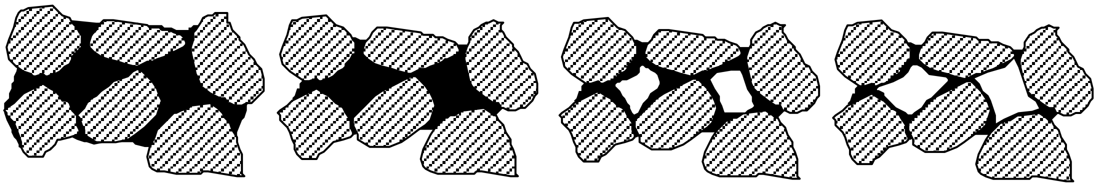


Figure 3.21: Conceptual distribution of pore water and air voids in a cross-sectional area of basecourse (Lu and Likos 2004)

This section will therefore, for completeness, present a more general introduction to water induced damage in typical unbound granular basecourse. It will then elaborate on the effects of water on cement treated basecourse, the theoretical basis for the mechanism of moisture ingress into cement treated basecourse, the Tube Suction Test, and the principles of carbonation.

3.7.1 Water Induced Damage on Basecourse

The major role that water plays in the behaviour of soil has been well established since the mid-1900s, through the works of Casagrande and Terzarghi. Similarly for pavements, which are largely soil based products, the detrimental effects of water are equally, if not more prominent due to the immediate visibility of its effects on pavement surfaces.

As per conventional geotechnical principles, when water saturates a pavement, the pore pressure reduces the effective strength of the pavement material by sustaining portions of the traffic loading. Consequently, when a traffic load is applied onto a pavement structure, a surge in water flows between voids occurs, (Thom 2010) giving rise amongst other events to three major effects:

i. cracking on the sealed surfaces due to the loss of support caused by the formation of larger voids when fines migrate along with the surge of water

ii. the formation of potholes where surges in pressure break through the seal, i.e. “pumping effects”

iii. the potential “washing out” of stabilising agents bound to the aggregate.

Furthermore, as the performance of unbound granular pavements depends heavily on the interlocking of aggregates; the presence of water also has a detrimental effect on the resilient modulus of pavements as seen in Section 2.5.2.1.

In mitigating these effects, cement modification of basecourse, as discussed in Section 2.5.2.1, has typically been used in road construction to increase the durability of pavements (Austroads 2006; Jenkins 2006). The treatment works by amalgamating the fines and aggregate of the basecourse which leads to an improvement in plasticity and reduced moisture sensitivity. These benefits are graphically represented in Figure 3.22 below (McConnell 2009)

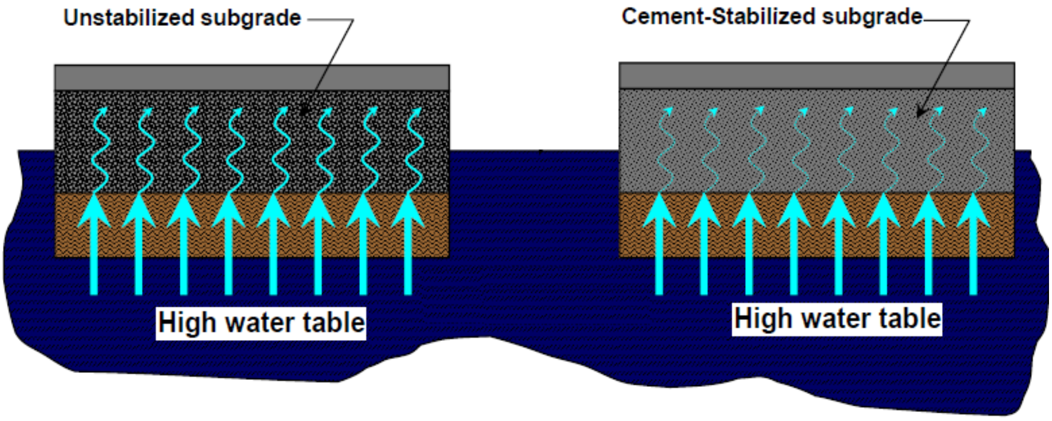


Figure 3.22: Benefits of cement stabilisation for moisture susceptibility (McConnell 2009)

Nevertheless, although cement is perceived to be beneficial to the durability of basecourse materials through the creation of a cement modified basecourse, the durability of the cement mix itself is not fully understood, and this durability is of great concern (as detailed in Section 2.5.5). The following provides further details on the durability of cement treatments.

3.7.2 Durability of Cement Treatments - Carbonation

As presented earlier in Section 2.5.4, the lack of durability of cement treated basecourse is primarily due to a chemical retardation process which affects the cement paste within the material. This issue of the durability of cement treated basecourse pavements is a problem found not only in Western Australia but also around the world. Documented evidence exists in South Africa and the United States where cement treated materials have undergone carbonation (Papadakis et al. 1989; Paige-Green et al. 1990; Guthrie et al. 2001; Harris and Lockwood 2009) and the treatment itself has "disappeared".

Carbonation studies have primarily been focused around concrete materials (Paige-Green et al. 1990). By 1985, there was an increased interest in studies on the carbonation of stabilised pavements, sparked in South Africa by the works of Paige-Green. These works contained observations of at least 44 known cases of stabilised pavements failing much earlier than their anticipated design life (Paige-Green et al. 1990).

The effects of carbonation result in the disappearance of binders and their mechanical benefits, i.e. approximately 40% of their UCS strength is left after certain number of years. Laboratory investigations also show that the strength gained from lime-stabilised samples cured after 7 years, can be reduced by 45% to 70% after 24 hours of applied carbonation (Paige-Green et al. 1990).

The theoretical and observed effects of carbonation are presented in Table 3.5 below (Paige-Green et al. 1990).

Table 3.5: Effects of carbonation on cement stabilised pavement materials.

Theoretical Effects	Observed Effects
1. Destruction of $Ca(OH)_2$ and $Mg(OH)_2$ and the production of $CaCO_3$ and $MgCO_2$ 2. Destruction of CSH and CAH compounds 3. Expansion of lime and shrinkage of hydrated calcium silicates 4. Decrease in ph from about 12.4 ultimately to about 8.3 on completion of carbonation 5. Decreased solubility 6. Reduced relative compaction	1. Destruction of $Ca(OH)_2$ and production of $CaCO_3$ 2. Decrease in UCE and CBR 3. Rutting due to loss of density 4. Microcracking due to loss of stiffness 5. Increase of PI 6. Decrease in ph to 8.3 – 10 7. Decrease in paste electrical conductivity (EC), indicating a decrease in solubility

Carbonation in essence, is a reverse reaction of typical road stabilisation products e.g. lime, cement and other cementitious products, where they revert back to their original constituents, due to the presence of carbon dioxide. Water plays two contrasting roles in the process of carbonation (Papadakis et al. 1989), i.e.

i. it provides a barrier hindering the diffusion of CO_2

ii. it provides a medium for reaction between CO_2 and the stabilising agents

This relationship can be graphically depicted, as shown in Figure 3.23 below, (Papadakis et al. 1989) where water is essentially bound to the cement matrix which acts as both a barrier and a catalyst, a process similar to the different phases of water shown earlier in Section 3.7.3.

As a result of the process, the rate of carbonation also depends on the ambient relative humidity (Papadakis et al. 1989), since in low relative humidity pores are dry and carbonation does not occur; high relatively humidity would clog the pores, limiting the rate of CO2 diffusion. Goodbrake et al (1979) ascertained that the percentage of calcium silicate which reacts with CO2 declines steeply below a relative humidity point of 50% and remains constant in excess of 50% (approximately 75%). This is further supported by Roberts (1981) who states that carbonation is most critical at ordinary temperatures in a relative humidity range of between 50% and 75%.

The carbon dioxide present in the reaction process is typically sourced from the decomposition of vegetation, transpiration of plants and respiration of insects. Although deemed to be insignificant processes, they provide sufficient sources of carbon dioxide to the road pavement structures, especially under relatively impermeable bituminous surfacing, which limits the infiltration of CO2 (Paige-Green et al. 1990).

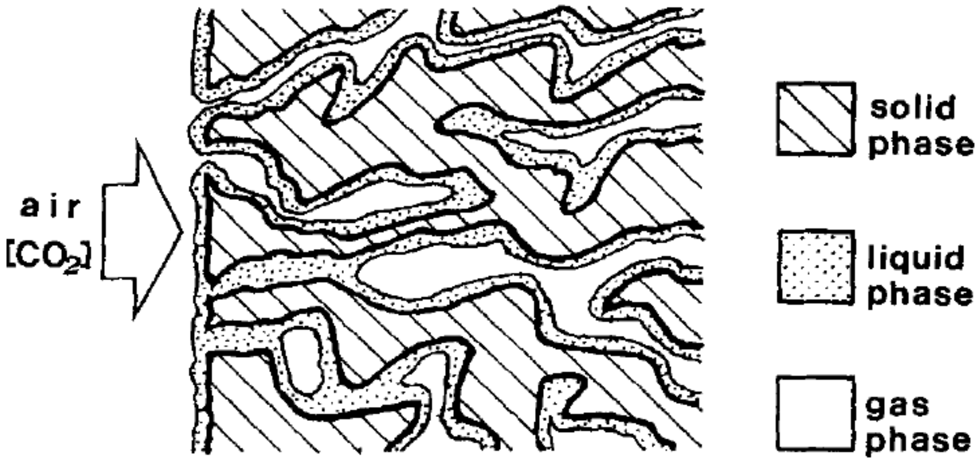


Figure 3.23: CO_2 diffusion in concrete pores (Papadakis et al. 1989; Papadakis 2005)

The two primary constituents, as covered earlier in Section 3.6, i.e. CSH and $Ca(OH)_2$, react with diffused carbon dioxide, as shown below:

Hydrated Cement Constituents

$$CSH + CAH + CO_2 \rightarrow CaCO_3 + SiO_2 + Al_2O_3 + H2O$$

(Hydrated cement constituents → limestone + silica + alumina + water)

Calcium Hydroxide

$$Ca\,(OH)_2\,(s) + CO_2\,(g) \rightarrow CaCO_3 + H_20$$

The overall reaction of Ca(OH)2, completed in the aqueous film between the aggregate wall and free water (see Figure 3.23) and through other elementary steps is shown below (Papadakis et al. 1989)

$$Ca(OH)_2\,(s) \rightarrow Ca^{2+}\,(aq) + 2OH^-\,(aq)$$

$$CO_2\,(g) \rightarrow CO_2\,(aq)$$

$$CO_2\,(aq) + OH^-\,(aq) \rightarrow HCO_3^-\,(aq)$$

$$HCO_3^-\,(aq) + OH^-\,(aq) \rightarrow CO_3^{2-}\,(aq) + H_2O$$

$$Ca^{2+}\,(aq) + CO_3^{2-}\,(aq) \rightarrow CaCO_3\,(s)$$

The carbonation reaction typically requires a minimum pH of about 11.0 to occur, which is generally achieved when as little as 0.005% of $Ca(OH)_2$ is dissolved in the pore water (Paige-Green et al. 1990).

Nevertheless, a certain concentration of $Ca(OH)_2$ is also required to maintain the stability of CSH. It is the opinion of this author that due to the wet-dry cyclic nature of pavements, the leaching of carbonated $Ca(OH)_2$ results in an ongoing and more severe carbonation susceptibility of pavements compared to conventional concrete where the $Ca(OH)_2$ has a higher propensity for stability within concrete.

Aggregates are deemed inert components but play a particularly important role in limiting the diffusivity of carbon dioxide (Papadakis 2005).

It is deemed that it is the impermeable nature of good-quality concrete that contributes to its durability (Paige-Green et al. 1990). It is therefore prudent to assess the rate of infiltration of moisture into cement treated basecourse in order to ascertain its durability. In achieving this, the following subsection discusses the mechanism of moisture ingress in porous media, and the unsaturated flow theory.

3.7.3 Mechanism of Moisture Ingress in Cement Treated Basecourse Materials

In conventional soil mechanics, the permeability of materials is determined based on Darcy's law, which assumes soils are in a fully saturated condition. However, pavements are typically dried back prior to sealing, and they remain mostly unsaturated or partially saturated throughout their service life depending on moisture fluctuations. The flow of moisture through materials under such conditions is known as unsaturated flow.

Unsaturated flow refers to the transient movement of moisture through porous materials where the water content is typically less than unity and inhomogeneous (Hall and Djerbib 2006) and involves external and internal forces, i.e. gravity and capillary/matrix suction. To some degree, osmotic suction of soils due to salt content also contributes to the process (Guthrie et al. 2001; Jayawickrama et al. 2009).

Green and Ampt (1911) first discovered a semi-analytical method to measure the rate of water infiltration through an initially dry and uniform column of soil, explaining their observation as a "sharp wetting front" (Lu and Likos 2004), where a relatively fast transient movement is followed by a reduction in infiltration rate. This is shown in Figure 3.24.

The rate of infiltration on the basis of unsaturated flow theory assumes that the soils are not fully saturated and air voids exist within the pavement. Theoretically, it builds upon the Darcy equation by introducing a dimensionless variable which represents the volumetric water content of the material, which attenuates the permeability factor (Hall and Djerbib 2006).

This is under the assumption that the suction head in the soil beyond the wetting front is constant, and the water content and corresponding hydraulic conductivity of the soil behind the wetting front are constant (Lu and Likos 2004). Therefore, a total infiltration displacement, Q for a unit cross-sectional area at any time, t, can be described as (Lu and Likos 2004):

$$Q = (\theta_o - \theta_i)x \qquad (3.26)$$

Where θ_o = volumetric water content behind the wetting front

Θ_i = volumetric water content beyond the wetting front

This is depicted in Figure 3.24 below (Lu and Likos 2004).

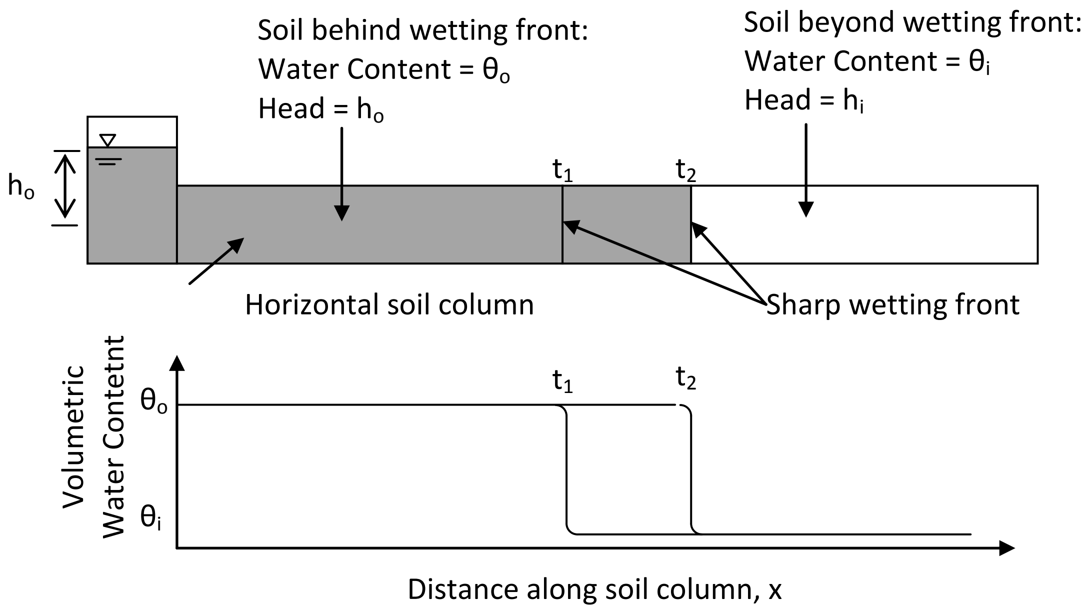


Figure 3.24: Transient infiltration of sharp wetting front in horizontal soil column (Lu and Likos 2004)

By applying Darcy's Law,

$$q = \frac{dQ}{dt} = \frac{(\theta_o - \theta_i)x}{dt} \tag{3.27a}$$

$$q = -k_o \frac{dh}{dx} = -k_o \frac{h_i - h_o}{x} \tag{3.27b}$$

where, h_i = the suction head at the wetting front

h_o = the suction head behind the wetting front

k_o = hydraulic conductivity behind the wetting front (assumed to be equal to saturated hydraulic conductivity)

Imposing an initial condition of x = 0 and t = 0, the integration of space and time yields the following:

$$x = \sqrt{Dt} \tag{3.28}$$

where, D = effective hydraulic diffusivity as the function represented by D has units of length squared over time

therefore, referring back to equation above,

$$Q = (\theta_o - \theta_i)\sqrt{Dt} = s\sqrt{t} \tag{3.29}$$

where, s = sorptivity

In characterising the measurement of unsaturated flow, a term known as Sorptivity is introduced, as shown in equation above. The term Sorptivity (S), introduced to unsaturated flow theory, was first used by Philip in 1957 (Philip 1957) to explain the absorption of water into a porous solid due to capillary suction (Lu and Likos 2004; Hall and Djerbib 2006; Gonen and Yazicioglu 2007).

Sorptivity is the rate of increase in water absorption against the square root of elapsed time. The cumulative volume water per unit inflow surface area, *i* can be represented as:

$$i = \frac{\Delta w}{A} \tag{3.30}$$

where, *i* = inflow volume

Δw = change in weight (g)

A = cross-sectional area of test face (mm2)

ρ = density of water (assumed at 0.988 g ml^{-1})

As numerically implied from the equation 3.27 above, sorptivity, S is defined as the gradient of the slope $i/t^{0.5}$ relationship and its linearity represents the homogeneity of the specimen.

3.7.4 Relationship between Sorptivity and Carbonation

The square root rate of absorption is also evident in other studies of carbonation, where the depth of carbonation of concrete has been determined to be proportional to the square root of time (Hamada 1969; Tuutti 1982; Nagataki et al. 1986; Li and Wu 1987; Richardson 1988) under controlled indoor conditions.

$$x = i\sqrt{t} \tag{3.31}$$

where, x = depth of penetration

i = material constant

Figure 3.25 below shows typical results of the rate of carbonation in concrete, based on the equation above (Papadakis et al. 1989).

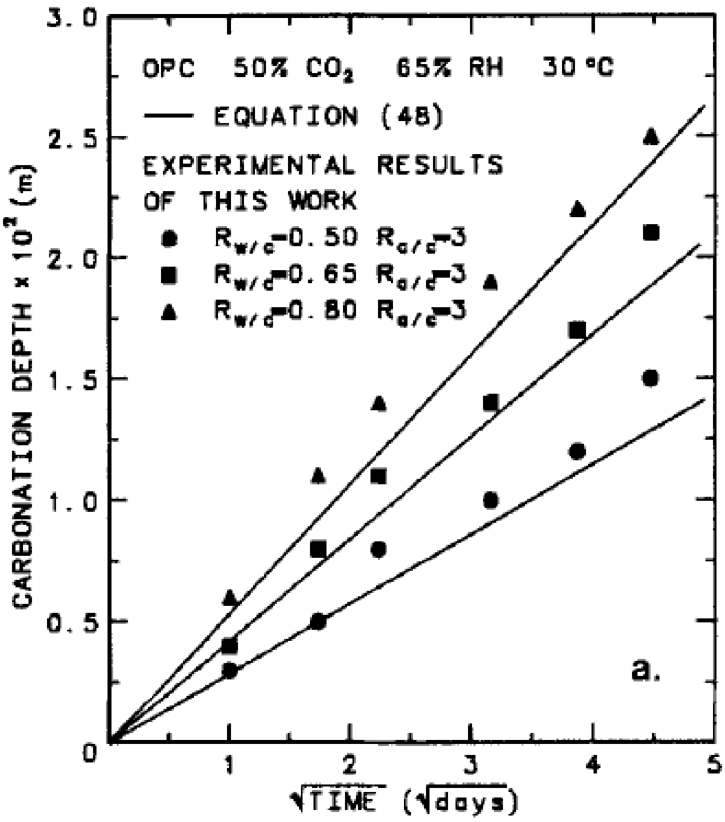


Figure 3.25: Typical carbonation depth vs. $\sqrt{time}$ (Papadakis et al. 1989)

The obvious similarity between the rate of penetration and sorptivity clearly indicates that concrete, or in the context of this dissertation, cement treated basecourse is porous material and its relationship to water infiltration is in effect based on the principles of unsaturated flow theory.

This give rise to an important measurement to assess the impact of carbonation on cement treated basecourse materials. Hence, in order to effectively measure the moisture content in materials, the Tube Suction Test is investigated and presented in the subsequent section.

3.7.5 Dielectric Permittivity and the Tube Suction Test

Typical testing for durability in pavement materials revolves around empirical approaches such as wet/dry cyclic testing and freeze/thaw cyclic testing. However, the Texas Department of Transport (TxDOT) has recently developed and implemented a new testing method known as the Tube Suction Test to assess the durability and moisture susceptibility of pavement materials.

The Tube Suction utilises the Adek Percometer™ to measure the dielectric value of pavement materials. The "percometer" is derived from the words permittivity and conductivity, and is a frequency domain instrument commonly used to measure the soil dielectric constant (ε_r) and conductivity (J) in agricultural studies (Saue et al. 2008). The percometer uses a specially designed metal probe, which acts as an electrical capacitor that is capable of measuring the electrical capacitance of its surrounding media, by means of dielectric permittivity.

Materials are made up of atoms which consist of positive point charges surrounded by a cloud of negative charges, as shown in Figure 3.26. Atoms do not interact with each other as they are separated by certain distance. The presence of an electric field will distort the charge cloud to a simple dipole, using the superposition principle. As a result, a dipole moment is formed, as shown in Figure 3.26 with a blue arrow labelled M pointing in the same direction as the electric field. A dipole moment is a vector quantity, its relationship with the electric field gives rise to the behaviour of dielectric permittivity.

The removal of the electric field returns the atoms to their original state. Usually, solids have a relatively low dielectric value, whereas water has a very high dielectric value. Therefore dielectric value measurement can resolve specimens with low quantities of free water accurately (Vorobieff 2004). The free water in a soil matrix is a better indicator of mechanical performance compared to the traditional gravimetric water content measure (Saarenketo 2000).

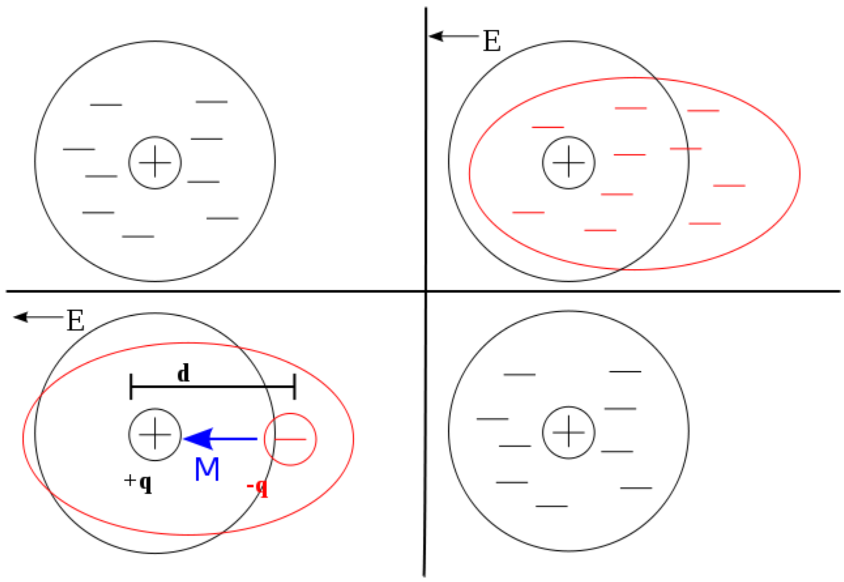


Figure 3.26: Electric field interactions with an atom under a classical dielectric model (Sensortech Systems 2009)

Dielectric permittivity is a complex function consisting of real and imaginary components. The dielectric constant is usually expressed as the real part of dielectric permittivity. It is the ratio of a material's electric field storage capacity to that of free space. The imaginary part of dielectric permittivity is expressed as dielectric loss which indicates attenuation and dispersion.

A distinct relationship exists between the dielectric permittivity of water in soils, and aggregates, as presented by Saarenketo (2000). The electrical permittivity of bulk soil is a function of both soil water content and the permittivity of the pore water. As field extraction of soil samples is often impractical due to instrumental error and variation in soil compositions, the invention of electromagnetic sensors provides an alternative solution to overcoming these problems.

The volumetric moisture and the state of molecular bonding in a material is measured by means of a dielectric value (DV). When the percometer's probe is placed in soil, the measurement provides an indication of the volumetric water content of the soil. By plotting the dielectric value over time, the moisture sensitivity of a material can be assessed (Saarenketo 2000) by observing the DV value attained and the shape of the graph. A similar dependence between dielectric value and moisture in concrete also exists (Bell et al. 1963).

The earlier form of Tube Suction Test was developed by the TxDOT to analyse the behaviour of ground-penetrating radar (GPR) signals of pavement materials (Guthrie and Scullion 2003) to formulate non-destructive methods for assessing in-service roads. From these tests, it was noted that the dielectric permittivity, E_R, of materials was capable of characterising pavement materials.

Through further funded research and a joint investigation between the Finnish National Road Administration and the Texas Transportation Institute (TTI), a standard Tube Suction Test was developed to assess the moisture susceptibility of granular materials (Barbu and Scullion 2006). Further research was then undertaken by Scullion et al. (2005), Guthrie et al. (2001) and George (2001), on the moisture susceptibility of cement stabilised materials using the Tube Suction Test with promising results on ascertaining the durability of the material. A typical test arrangement and its results are shown in Figure 3.27 below.

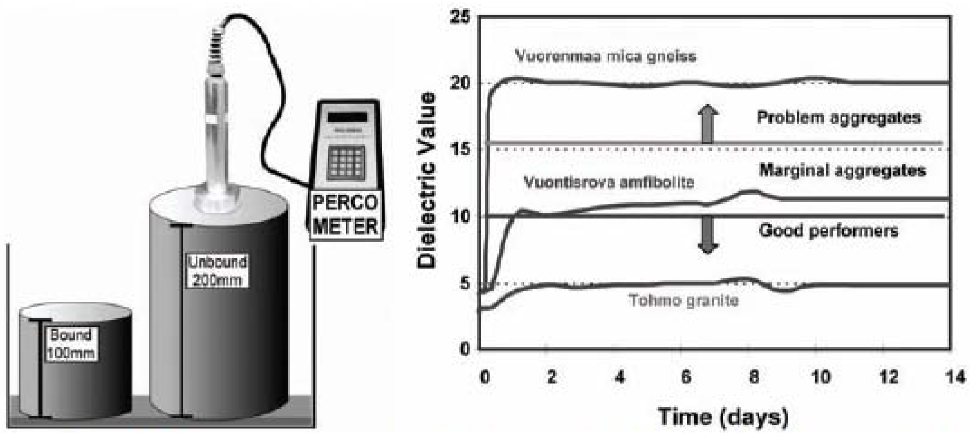


Figure 3.27: Tube Suction Test setup and typical results (Guthrie et al. 2001)

Besides the decrease in durability caused by water, water also plays a significant role in the shrinkage behaviour of cement treated basecourse. This is covered in the subsequent section.

3.8 Shrinkage in Cement Treated Basecourse

Shrinkage of cementitious material is a natural occurrence (George 2002), and is the primary type of distress in flexible pavements constructed with cement treated basecourse (Hanson 2006). This is seen in surveys of Australian local roads, where results revealed that the predominant issue in road pavement with cementitious material is shrinkage induced cracking (Chakrabati and Kodikara 2004). Similarly in South Korea, shrinkage cracking is the predominant deterrent to the application of cement treated basecourse (Cho et al. 2006).

As presented earlier in Section 2.5.4, the shrinkage of basecourse translates into surface cracks, a process known as reflection cracking (George 2002; Adaska and Luhr 2004; Cho et al. 2006). The reflection cracks undermine the serviceability of the pavement and drastically increase the life cycle costs of pavements as a result of increased maintenance.

Reflection cracking is deemed a significant issue both locally (Austroads 2006; Austroads 2008) and internationally (George 2002; Adaska and Luhr 2004; Cho et al. 2006). This was highlighted in a survey on Australian roads where shrinkage cracking was deemed to be the primary issue in the use of stabilised materials. Reflection cracking allows water to infiltrate the pavement structure thus hastening the deterioration of the pavement.

However, the application of cement to pavements affects the shrinkage behaviour of the pavement material, as it consumes a greater amount of water during hydration, thus increasing drying shrinkage as well as increasing the rigidity of the material and increasing its susceptibility to fracture (Guthrie et al. 2001; George 2002; Adaska and Luhr 2004; Scullion et al. 2005). However, it has been identified that cement potentially plays a beneficial role in materials which exhibit high volumetric changes without cement (Adaska and Luhr 2004). This relationship can be seen in Figure 3.28 below:

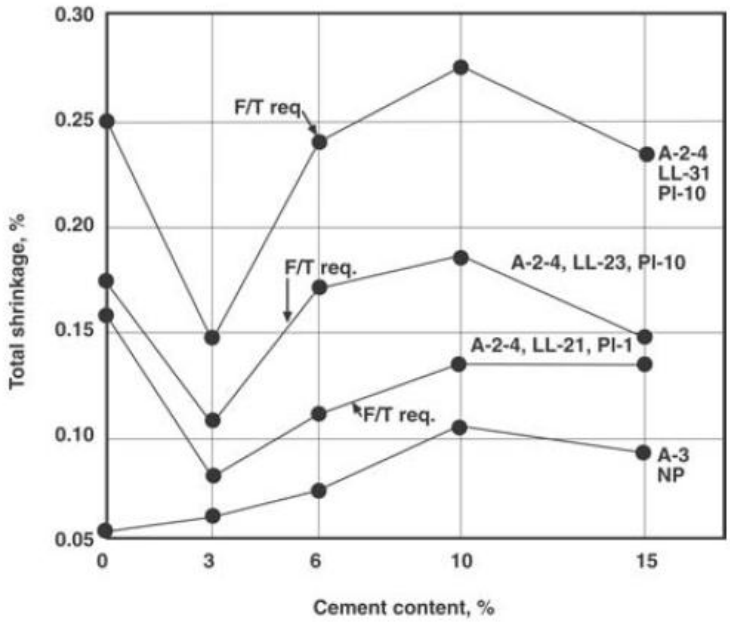


Figure 3.28: Effects of cement content on shrinkage (Adaska and Luhr 2004)

3.8.1 Mechanism of Shrinkage

The shrinkage behaviour of cemented materials has long been understood to be caused by moisture suction force when cement dries (Han and Lytton 1995; Chakrabati and Kodikara 2005; Cho et al. 2006). During the hydration of cement, shrinkage is experienced by the material due to chemical processes which are a form of “self-desiccation” (George 2002). The thermal shrinkage from environmental factors (George 2002), and the loss of moisture from finer pores (Guthrie et al. 2001; Chakrabati and Kodikara 2005; Cho et al. 2006), i.e. dry shrinkage, results in significant stresses on the material. The drying shrinkage or autogenous shrinkage is the predominant factor that effects pavements and is depicted in Figure 3.29, shown below (Cho et al. 2006).

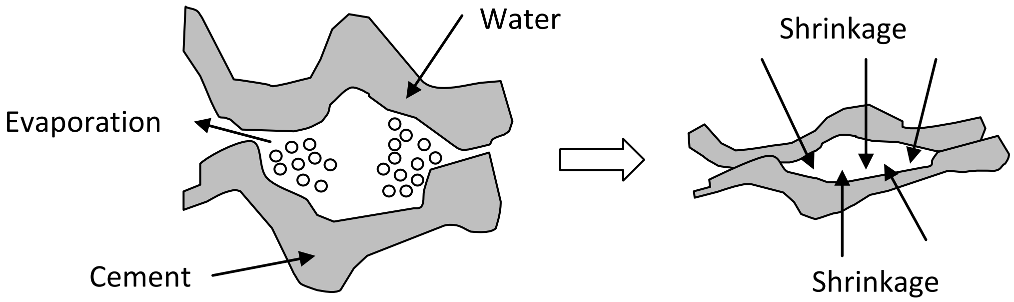


Figure 3.29: Shrinkage mechanism of cement paste (Cho et al. 2006)

When this volumetric change occurs, a restraining tensile stress is applied to the cement matrix, which has the potential to exceed the tensile strength of the material and thus results in the formation of cracks.

Various factors influencing the magnitude of dry shrinkage in CTB are inferred by optimum moisture content, clay content, admixture, moisture loss rate, compaction, and excess use of cement dosage (George 2002; Portland Cement Association 2003; Chakrabati and Kodikara 2005). In particular the authors hypothesised an associative relationship between fines and cement content in shrinkage, asserting the detrimental effects of high fines or clay content.

As shrinkage is inevitable for cementitious material, its mitigation is aimed primarily at limiting the size of cracks to a level which is not detrimental (George 2002). It is suggested that asphalt cracking that is less than 3mm will not cause premature failure of pavements (Portland Cement Association 2003).

The minimisation of cracking has therefore been an area of great research interest. In comprehensive studies of mitigation methodologies, the categories of mitigation measure that have been practiced are (George 2002; Adaska and Luhr 2004; Scullion et al. 2005; Cho et al. 2006):

i. Material proportioning and specification

ii. Additives and special cement

iii. Construction and curing

iv. Physical alterations

This paper in dissertation focuses on the material proportioning method, i.e. prescribing adequate dosages of cement content while establishing the characterising factors of each level of cement. Australia has made significant progress in specifying base material specifications to ensure minimal shrinkage (Caltabiano and Rawlings 1992), while defining shrinkage limits. Further discussion of laboratory investigations to this end are discussed in 3.8.3.

3.8.2 Measuring Shrinkage

Based on Figure 3.29, it can be seen that the pore size distribution of a cement matrix plays a role in determining the propensity of the material to undergo shrinkage, as it dictates the total suction force generated by free water contained within pores. The pore sizes of a cement matrix, as shown in Figure 3.16, can be within the nanometre scale, and can be modelled as a sphere. The matrix suction, Ψ, of a perfectly spherical meniscus in equilibrium conditions can thus be defined as (Chakrabati and Kodikara 2007):

$$\Psi = \frac{2t_n}{r}\cos\theta \tag{3.32}$$

where, t_n = surface tension of water

θ = contact angle between water and the pore wall

r = radius of the meniscus

As the equation suggests, an inverse relationship exists between pore size and the matrix suction and thus the propensity to shrink, i.e. smaller pore sizes, will result in higher matrix suction and a higher propensity for shrinkage. Furthermore, the suction of water during drying shrinkage can be related to relative humidity (RH) of the pores, through the Kelvin-Laplace equation.

$$\Psi = \frac{R\mathrm{T}}{W_v}\ln RH \tag{3.33}$$

where, R = universal gas constant

T = temperature

W_v = molecular weight of the water

RH = relative humidity of the pores

Works by Chakrabati and Kodikara (2007) have shown that the pore sizes of cement and cement treated basecourse are generally the same. A relationship between RH, pore size and suction can therefore be established, as shown in Figure 3.30 below (Chakrabati and Kodikara 2007).

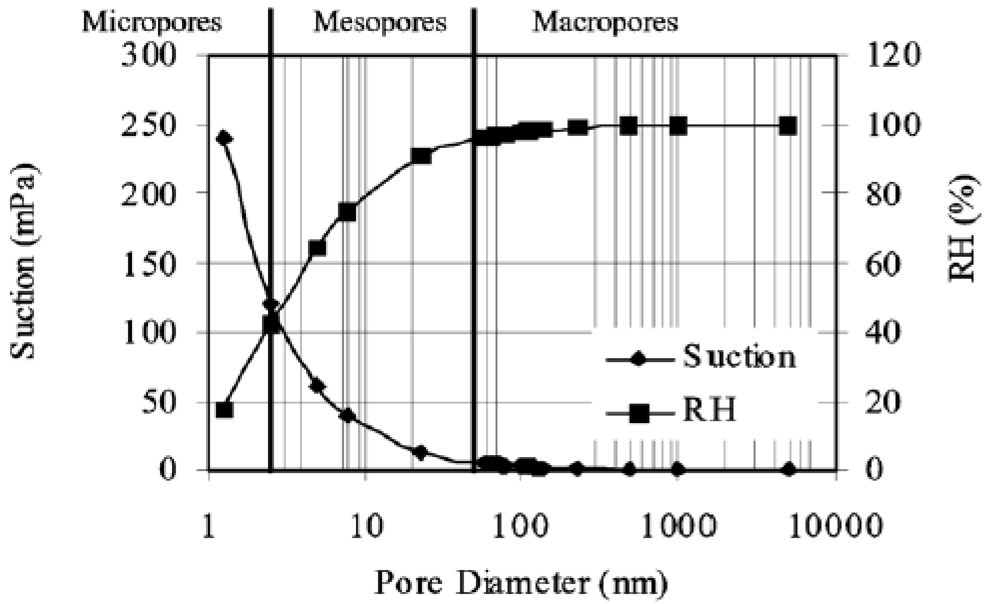


Figure 3.30: Relationship between shrinkage, pore diameter and relative humidity for cement treated materials

On the basis of IUPAC (International Union of Pure and Applied Chemistry) classifications, micropores are less than 25nm, mesopores range between 25 and 500nm and macropores range between 500nm and 50µm (Chakrabati and Kodikara 2007). Figure 3.30 above shows that shrinkage of cement treated basecourse is caused by pores within the range of micropores and mesopores, i.e. pore sizes between 2.5 nm and 30 nm.

However, in order to assess the propensity of shrinkage for cement treated basecourse materials, based on the known range of cement, nitrogen adsorption can be applied, as discussed in the subsequent section.

3.8.3 Nitrogen Adsorption

Based on Figure 3.30 above, the critical pore size that will dictate the shrinkage potential of cement treated basecourse is within the size of a mesopore and a macropore. N_2 adsorption techniques have been used for pore size investigation into Portland cement (Chakrabati and Kodikara 2007).

Gas adsorption isotherm makes a significant contribution to the characterisation of a wide range of porous and nonporous materials alike. One widely utilised gas is nitrogen, which is an abundant adsorptive gas. Nitrogen adsorption isotherm has been used since the late 1940s for the analysis of pore size and surface area (Sing 2001).

One general method to determine the adsorption isotherms of nitrogen at the temperature of liquid nitrogen is gas adsorption manometry. This type of approach involves measuring the change in gas pressure before and after absorption. In this case, the adsorptive is introduced into the system at a slow rate to achieve a state of “quasi-equilibrium” in order to obtain a continuous isotherm (Bhambhani et al. 1972).

Langmuir’s theory of monolayer adsorption paved the way for the interpretation of nitrogen adsorption isotherm. Later, Brunauer, Emmett and Teller (BET) extended Langmuir’s theory to the BET method which is now a standard tool for surface area and pore size estimation. Assumptions made in BET are such that the adsorbent surface is pictured as an array of equivalent sites on which molecules are adsorbed in a random manner, and the molecules in the first layer act as sites for molecules in the second layer, and so forth. However, the BET method faces limitations such as (1) the monolayer structure is not the same on all surfaces; (2) strong adsorption at very low p/p_0 may involve localised monolayer coverage (Bhambhani et al. 1972; Sing 2001).

3.9 Erodibility Index and the Wheel Tracking Test

De Beer (1989) undertook a comprehensive review of testing methods available at the time, to assess pavement erodibility and durability (Scullion et al. 2005). In his study, the South African Wheel Tracking Test (SAWTET) was deemed as a more representative testing method for lightly cemented basecourse materials, due to its ability to model the in-situ distress mechanisms experienced by thin sealed pavements (Guthrie et al. 2001).

The test is also deemed appropriate to model the distress mechanisms of the unsealed pavements in this section. It was proposed that an erodibility index be used as an empirical quantification of the propensity of the particulates of a surface to erode, and this is expressed as a depth of erosion caused by the SAWTET apparatus (see Figure 3.31) after 5000 passes (De Beer 1989).

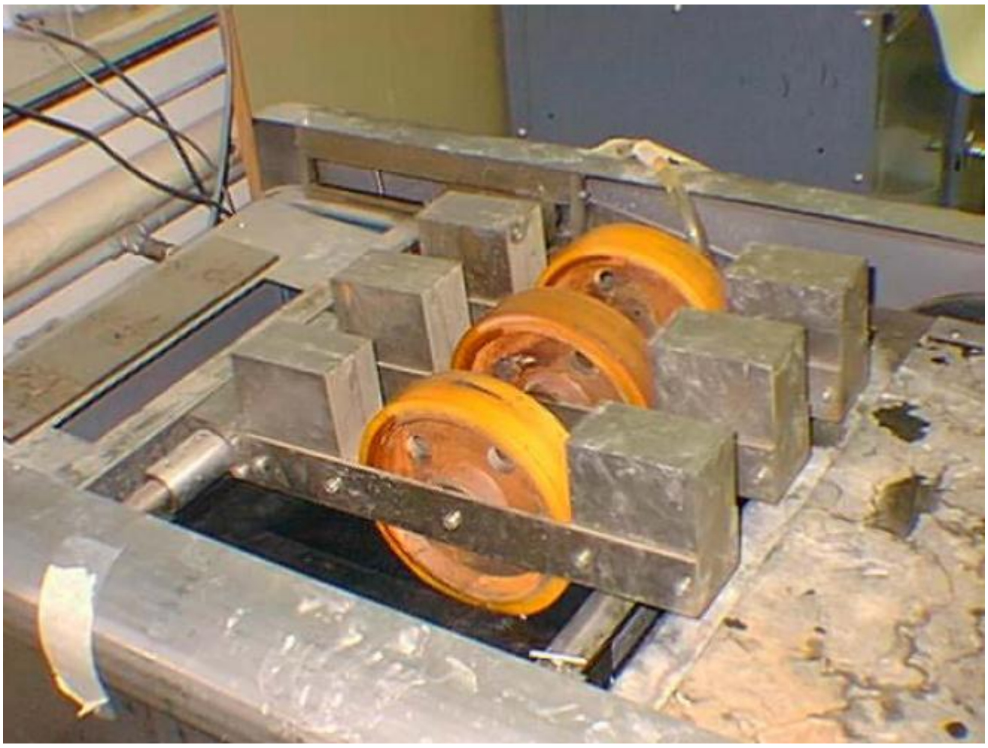


Figure 3.31: South African Wheel Tracking Test (SAWTET)

In Australia the erodibility index is used typically for concrete pavements and asphalt wearing courses. The only known test method in practice is the Road and Transport Authority of New South Wales Test Method T186, as shown in Figure 3.32 below (Austroads 2006):

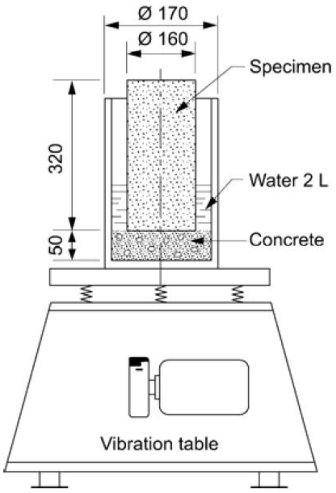


Figure 6.6: RTA NSW erodibility test

Figure 3.32: Road and Transport Authority New South Wales Erodibility Test

Similar test setups emulating the concept of SAWTET also exist for asphalt testing, to measure the rutting resistance of asphaltic seals. The Cooper Wheel Tracking Device is the most widely accepted asphalt tester in Australia (Austroads 2006) and is part of the repertoire of testing apparatus available at Curtin University's Pavement Research Group. The Wheel Tracker Test uses a reciprocating table which travels 230mm on linear bearings at a specified speed. The test specimen is then placed onto the bed, with a rubber tyre wheel connected to a transducer resting on the specimen. A typical setup is shown in Figure 3.33 below.

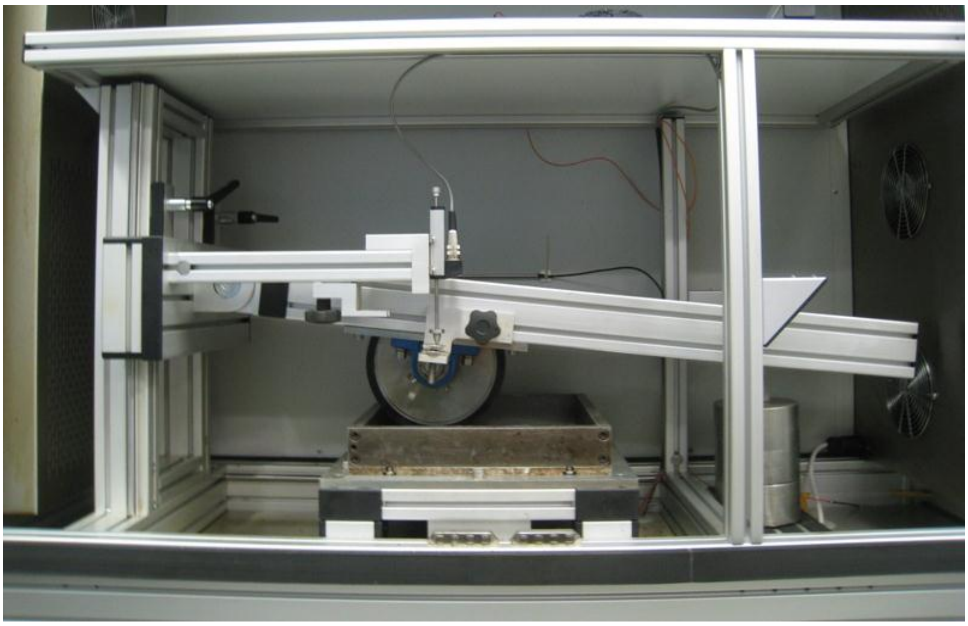


Figure 3.33: Cooper Wheel Tracking Test

3.10 Summary of Background

This chapter has presented a thorough theoretical discussion on the material properties relevant to achieving the objectives of this dissertation.

It is concluded that a wide range of academic and industrial research has been undertaken to assess the material properties of cement treated basecourse. However, the transfer of knowledge to the Australian context, and more specifically to the Western Australian context, has not been addressed with similar rigour.

Based on the literature review, the following tests shown in Table 3.6 have been adopted to assess relevant material properties.

Table 3.6: Summary of testing of characteristics of cement treated basecourse

Material Property	Test Method
Strength	Unconfined Compressive Strength and Indirect Tensile Strength
Fatigue Susceptibility	Four Point Flexural Fatigue Bending
Shrinkage Susceptibility	Nitrogen Adsorption
Moisture Sensitivity	Tube Suction Test
Erodibility	Wheel Tracking Test

As presented earlier in Section 3.3, the strength properties of cement treated basecourse can be calculated based on UCS and ITS. These values can then be further combined to assess the shear parameters of the material.

As for the fatigue characteristics of cement treated basecourse, studies by the Australian Road Research Board (ARRB) have given light to the need for revised fatigue models. A strain ratio based relationship is deemed necessary to better explain the phenomenon. The four point flexural bending test as used by ARRB is thus deemed suitable for the scope of this dissertation.

Since durability has been a significant issue for cement treated basecourse in Western Australia, as covered earlier in 2.5.4, the Tube Suction Test is adopted in this dissertation to assess the durability of materials. An obvious synergy exists between carbonation, infiltration, and the unsaturated flow theory, measureable with the Tube Suction Test.

The wheel tracking test typically used for asphalt is also adopted by this dissertation for the assessment of erodibility.

In summary, the discussions presented in this chapter have allowed the formulation of non-standard tests, as shown in Table 3.6, that will provide new insights into cement treated basecourse, to assist in assessing its overall performance. The materials and the details of the testing procedures are presented in the next chapter.

4 Materials and Methodology of Research

With Chapter 3 having discussed the theoretical principles of cement treated basecourse, this chapter provides key details on the materials and methodology used to complete the works in this dissertation.

The chapter comprises two main sections, i.e.

i. Materials, specifications and details

ii. Testing methodology

The chapter concludes with a comprehensive depiction of the laboratory program undertaken for this dissertation.

4.1 Materials

The materials used in this research include general purpose cement and crushed rock basecourse, typically utilised in road construction in Western Australia. Details of these materials are presented below:

4.1.1 General Purpose Cement

Cement used in cement treatment must comply with the requirements of Australian Standards AS3972 *General Purpose and blended cements*, Type General Purpose (GP) Cement. GP Cement was procured from Cockburn Cement Ltd. who are accredited with AS/NZS ISO 9001:2000. Figure 4.1 below shows the typical GP cement used in this dissertation.

Figure 4.1: GP Cement

The material specifications of Cockburn Cement's Type GP Cement are as shown in Table 4.1 below

Table 4.1: GP Cement material general specification

Parameter	Method	Units	Typical	Range	AS3972 Limits
Chemical Analysis					
SiO_2	XRF	%	20.3	19.8 – 20.6	
Al_2O_3	XRF	%	4.9	4.6 – 5.2	
Fe_2O_3	XRF	%	2.8	2.6 – 3.0	
CaO	XRF	%	63.9	63.1 – 64.7	
MgO	XRF	%	2.0	1.5 – 2.5	
SO_3	XRF	%	2.4	2.1 – 2.7	3.5% max
LOI	AS2350.2	%	2.5	2.1 – 2.9	
Chloride	ASTM C114	%	0.015	0.005 – 0.025	
Na_2O equiv	XRF	%	0.5	0.4 – 0.6	
Fineness Index	AS2350.8	m^2/kg	400	375 – 425	
Normal Consistency	AS2350.3	%	29.5	28.5 – 30.5	
Setting Times					
Initial	AS2350.4	mins	135	105 – 150	45 mins min
Final	AS2350.4	mins	195	165 - 225	6 hours max
Soundness	AS 2350.5	mm	1.0	0.0 – 2.0	5mm max
Comp. Strength					
3 Day	AS2350.11	MPa	38	35 – 41	
7 Day	AS2350.11	MPa	48	44 – 52	35 MPa
28 Day	AS2350.11	MPa	60	56 – 64	45 MPa

4.1.2 Crushed Rock Basecourse

Crushed rock basecourse was sourced from Holcim (Australia) Quarries Pty Ltd in Gosnells, Western Australia. Holcim is the primary supplier of crushed rocks in Western Australia, with materials complying with Main Roads Western Australia

Specification 501 Pavements (Main Roads Western Australia 2011). Figure 4.2 below shows the typical crushed rock basecourse used in this dissertation.

Figure 4.2: Crushed rock basecourse from Holcim Quarries Pty Ltd

The crushed rock basecourse material used for this research work conforms to the particle size distribution as shown in Table 4.2 below

Table 4.2: Particle size distribution for crushed rock basecourse (Main Roads Western Australia 2011).

Sieve Size	Target	Min	Max
26.5			100
19.0	100	95	100
13.2	82	70	90
9.5	70	60	80
4.75	50	40	60
2.36	38	30	45
1.18	25	20	35
0.600	19	13	27
0.425	17	11	23
0.300	13	8	20
0.150	10	5	14
0.075	8	5	11

Furthermore, the crushed rock basecourse used meets other material specifications as listed in

Table 4.3.

Table 4.3: Material Specification for Crushed Rock Basecourse (Main Roads Western Australia 2011).

Test	Limits
Liquid Limit (Cone Penetrometer)	25.0%
Linear Shrinkage	2.0% Max / 0.4% Min
Flakiness Index	30% Max
Los Angeles Abrasion Value	35% Max
Maximum Dry Compressive Strength	1.7 MPa Min
California Bearing Ratio at 99% MMDD	100% Min
Wet/Dry Strength Variation	35% Max
Secondary mineral content in basic igneous rock	25% Max
Accelerated soundness index by reflux	94% Min

4.1.3 Specimens Mixes

In order to identify the influence of cement content on the behaviour of cement treated basecourse, and to meet the objectives of this dissertation, six different mixes of specimen were prepared, i.e. specimens with 1% to 6% cement content by dry mass. The required mass of cement content was calculated based on the equation below:

$$cement(g) = \frac{M_w}{1+mc} \times C \qquad (3.31)$$

where, M_w = wet mass of specimen (g)

mc = moisture content (%)

C = cement content (%)

Cement and aggregates were dry mixed before adding water, (typical to the standard process) and subjected to the various mechanical and chemical tests as discussed in the following section.

4.2 Testing Methodology

This section presents the testing methodology used in the characterisation of cement treated basecourse, as described in Table 3.6 in the previous chapter, along with the modified compaction test. The following subsections provide detailed information on the tests undertaken which include:

i. Modified Proctor Compaction Test
ii. Unconfined Compressive Strength Test (UCS)
iii. Indirect Tensile Strength Test (ITS)
iv. Flexural Bending Test (FBT)
v. Flexural Fatigue Test (FFT)
vi. Tube Suction Test (TST)
vii. Nitrogen Adsorption
viii. Linear Shrinkage Test
ix. Wheel Tracking Test

In addition to the tests for assessment of material properties, a modified proctor compaction test was also undertaken.

4.2.1 Modified Proctor Compaction Test

The modified Proctor compaction test allows the determination of the modified dry density (MMDD) and optimum moisture content (OMC) of different mixes. The modified Proctor compaction test was carried out in accordance with AS1028.5.2.1 – 2003 *Methods of testing soils for engineering purposes – Method 5.2.1: Soil compaction and density tests – determination of the dry density/moisture content relation of a soil using modified compaction effort*. A cylindrical cell measuring 105 mm (diameter) x 115 mm (height) was used and a minimum of three specimens for every water content measure was fabricated to determine the MMDD and OMC.

4.2.2 Unconfined Compressive Strength Test

The Unconfined Compressive Strength Test (UCS) is undertaken to assess the unconfined compressive strength of materials. The test used in this research was carried out in accordance with AS5101.4 – 2008 *Methods for preparation and testing of stabilised materials – Method 4: Unconfined compressive strength of compacted materials*.

A cylindrical cell measuring 105 mm (diameter) x 115 mm (height) was used. The specimens were compacted to 100% MMDD at OMC. Two sets of triplicate specimens were fabricated and subjected to 7 days curing in a moisture chamber and 28 days wrapped curing respectively.

All specimens were soaked for 4 hours prior to testing. The GCTS Hydraulic Soil Triaxial Machine, as shown in Figure 4.3 and Figure 4.4 below, was used for the loading of the specimens at a displacement rate of 1.0 mm/minute. Results of the tests were generated by the connected computer which generated the transient stress strain responses.

Figure 4.3: Controls MCC 8 Computerised Control Console for UCS tests

Figure 4.4: GCTS STX-3000 for UCS and ITS tests.

4.2.3 Indirect Tensile Strength Test

The Indirect Tensile Strength Test (ITS) is undertaken to assess the tensile strength of materials. The ITS used in this research was developed with reference to AS 1012.10 – 2000 *Methods of testing concrete – Method 10: Determination of indirect tensile strength of concrete cylinders (‘Brazil’ or splitting test)*.

Figure 4.5: ITS rig with indicative specimen

A cylindrical cell measuring 105 mm (diameter) x 75 mm (height) was used. The specimens were compacted to 100% MMDD at OMC. Two sets of triplicate specimens were fabricated and subjected to 7 days curing in a moisture chamber and 28 days wrapped curing respectively. Specimens are then inserted into an ITS rig as shown in Figure 4.5 above.

All specimens were soaked for 4 hours prior to testing. The GCTS STX-3000, as shown in Figure 4.4 above was used for the loading of the specimens at a displacement rate of 1 mm/minute. Results of the tests were generated by the connected computer which generated the transient stress strain responses.

4.2.4 Flexural Bending Test

The Flexural Bending Test (FBT) is undertaken to assess the flexural strength and flexural breaking strain of materials. The FBT was developed with reference to AS1012.11 – 2000 *Methods of testing concrete – Method 11: Determination of modulus of rupture*.

A uniquely designed mould, as shown in Figure 4.6 below, was used to fabricate specimens measuring 64 mm (width) x 50 mm (height) 400 mm (length). Triplicate specimens were compacted to 100% MMDD at OMC and allowed to set in the mould for 24 hours before being removed. Specimens were then subjected to 28 days curing in a moisture chamber.

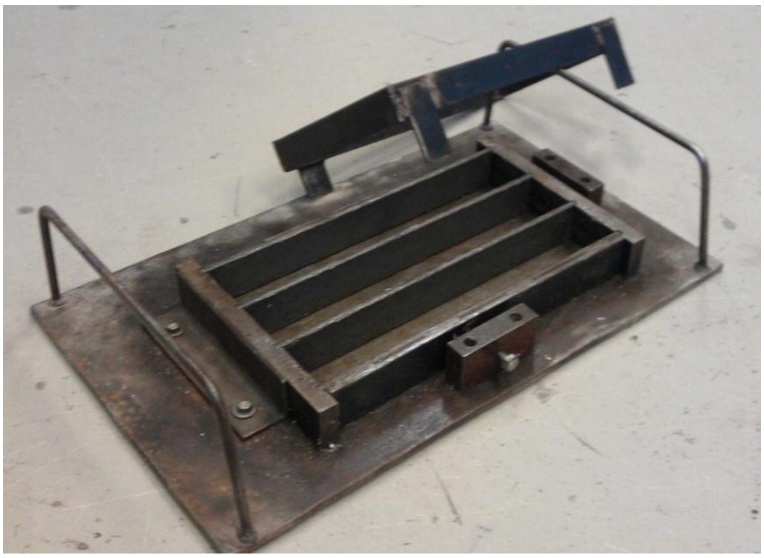


Figure 4.6: Flexural beam mould and collar

The Instron 5500R Beam Testing Machine, as shown in Figure 4.7 below, was used for the loading of the specimens at a displacement rate of 1 mm/minute. Results of the tests are generated by the connected computer which generated the transient stress strain responses.

Figure 4.7: FBT loading machine

4.2.5 Flexural Fatigue Test

The flexural fatigue bending test is a non-standard test based on the methodology typical for asphalt pavements, and with reference to the methodologies developed by Australian Road Research Board (ARRB), and the user manuals provided by IPC Global (IPC Global). A similar specimen preparation methodology as per the FBT was used but specimens were instead cured for 7 days in a moisture chamber.

Two tests were executed as part of the fatigue tests, which included an elastic limit test and a repeated flexural test, in order to determine the elastic limit of the material and to determine the modulus vs. cycle (S-N) curve. A minimum of three samples at various cement content was fabricated for each test.

Specimens were encased in clear wrap to minimise the drying out of the specimens. They were then loaded onto a four point bending jig, developed by IPC Global, as shown in Figure 4.8, and fitted into the IPC Global Pneumatic Loader (shown in Figure 4.9).

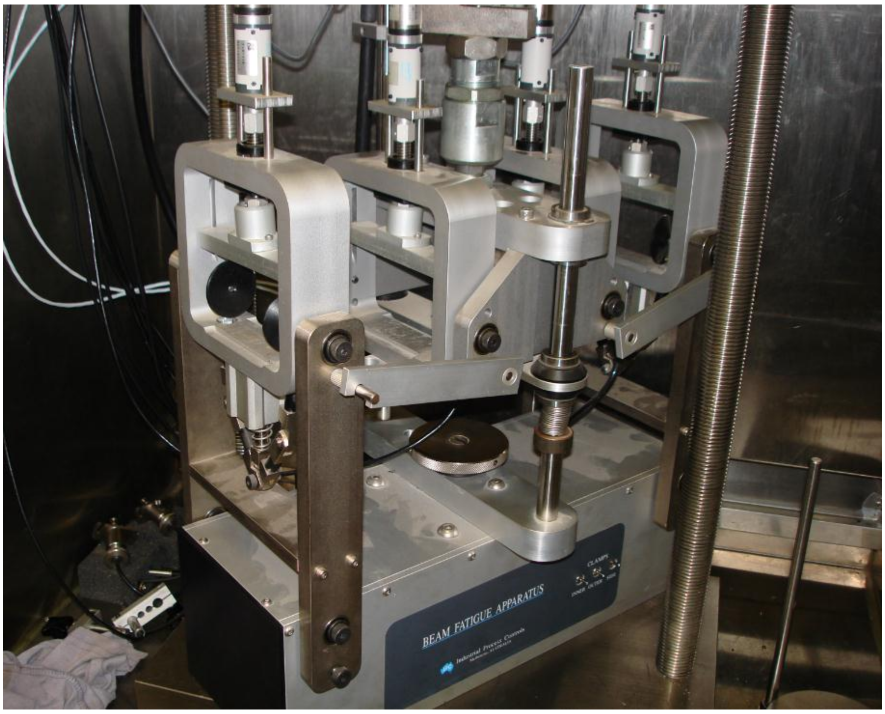


Figure 4.8: IPC Global beam fatigue apparatus

Figure 4.9: IPC Global universal testing machine and environment chamber

The elastic limit tests were undertaken by applying 200 cycles of haversine shaped strain loads at 10 microstrain increments of 200 Hertz. This allowed a plot of a stress – strain curve and the assessment of the point at which initial damage occurs.

The fatigue test on the other hand, applied a repeated cyclic haversine shaped 75 $\mu\varepsilon$, 200 $\mu\varepsilon$ and 400 $\mu\varepsilon$ constant strain force, up to a maximum of 368,000 cycles at 100 Hertz to assess the S-N response.

After removing the specimens from the moulds, they were weighed both before and after the test to ensure consistency of measurement of moisture levels.

Where specimens were still intact after the fatigue tests, further testing was undertaken to assess the sensitivity of the tests to other parameters. These included increasing the applied strain, and varying the frequency of loading.

4.2.6 Tube Suction Test

The Tube Suction Test was carried out in accordance with the works undertaken by Scullion et al. (2002), and the guidelines produced by the Texas Department of Transport (TxDoT 2002).

A cylindrical cell, similar to that used in the UCS test, measuring 105 mm (diameter) x 115 mm (height) was used. Quadruple specimens were compacted to 100% MMDD at OMC and allowed to cure in a moisture chamber for 7 days.

Upon curing, the specimens were oven dried for 24 hours at 60^{o}C, to remove all water content. Three of the four specimens were then submerged in approximately 13 mm water bath (6.5 mm exposed, and 6.5 mm covered by a rubber membrane) in an enclosed case for a maximum of 10 days, as shown in Figure 4.10 below, while the fourth specimen was encased in clear wrapping to act as a control specimen.

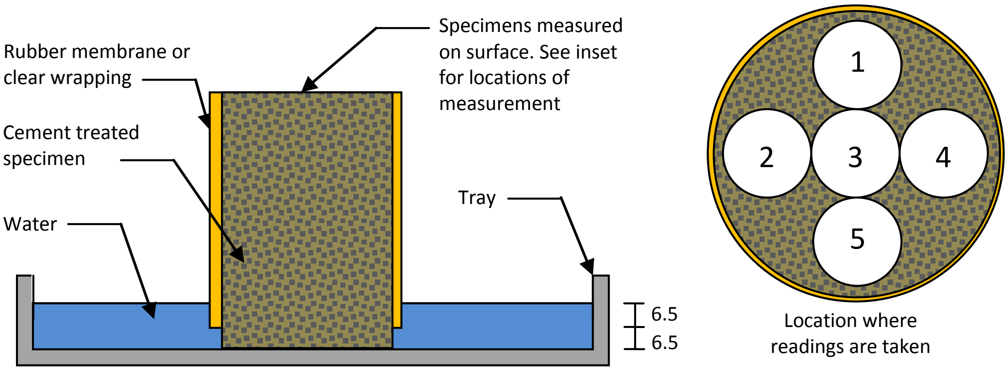


Figure 4.10: TST setup and surface measurement profile

Dielectric values and the wet mass of the specimens were taken with the Adek Percometer, and a weighing scale, prior to being submerged. Values and mass measurements were then taken consistently every day, with a maximum of 10 readings taken (shown in Figure 4.11).

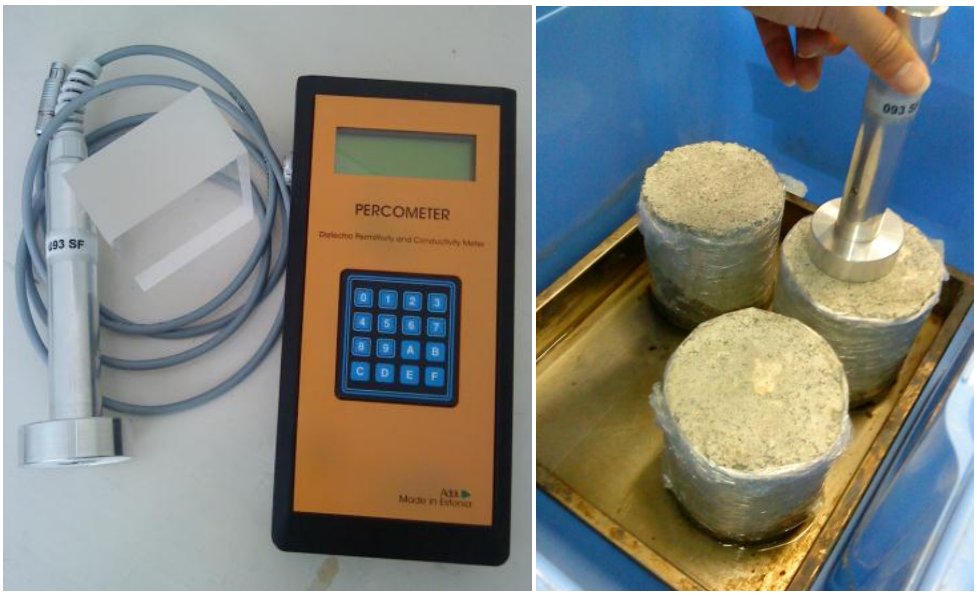


Figure 4.11: Dielectric values, DV are read using the Adek Percometer

After taking 10 readings, the specimens were then weighed to ascertain their (individual) final mass and water content, before being subjected to a UCS test, in accordance with the methodology (as discussed in section 4.2.2).

4.2.7 Nitrogen Adsorption

Nitrogen Adsorption tests were undertaken to assess the pore size distribution of cement treated crushed rock basecourse material. The target mass fraction of materials passing the 0.425 mm sieve size, as shown in Table 4.2, was mixed at OMC with varying cement contents, to produce specimens as shown in Figure 4.12 below. Specimens were then immediately encased in clear wrap and placed in an enclosed container to maintain temperature and humidity.

Figure 4.12 Sample preparation for nitrogen adsoprtion

At 24 hours, 7 days and 28 days, samples weighing approximately 2 g were scraped from the specimens and microwave-oven-dried for 10 minutes. This drying was undertaken to remove the traces of water in the samples in order to hinder the hydration process. Samples were further degassed to remove all traces of water molecules prior to N2 testing by the operators. Reports showing the pore size distribution and surface area were then obtained from the tests. Tests were then carried out using the Micrometrics Tristar II 3020 machine, as seen in Figure 4.13 below.

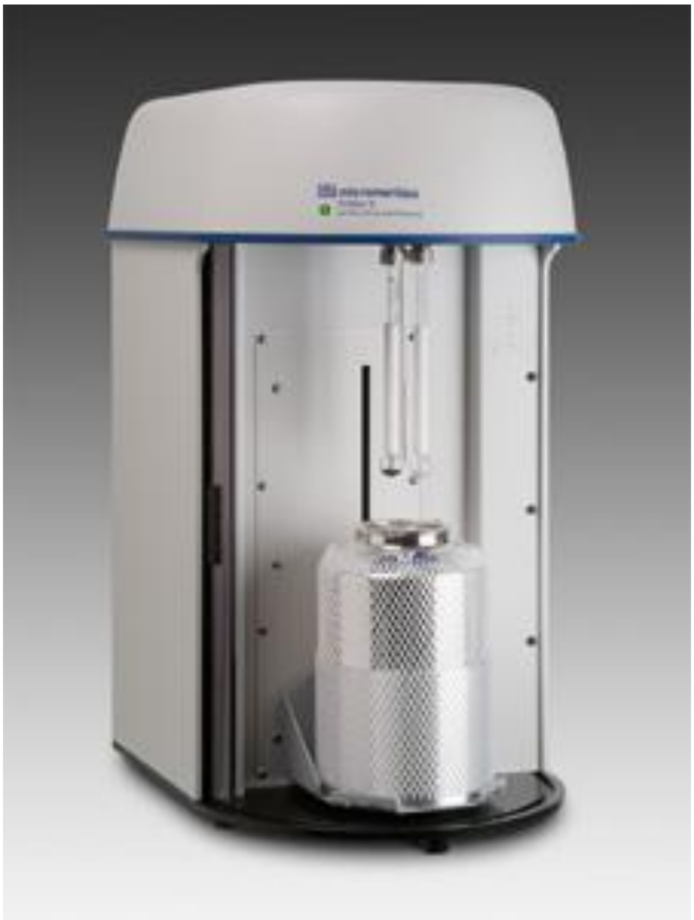


Figure 4.13: Tristar II 3020 for nitrogen adsorption

4.2.8 Linear Shrinkage Test

The linear shrinkage test is used to assess an empirical measurement of the shrinkage propensity of cement treated basecourse. The test used in this research was developed based on AS1289.3.4.1 – 2008 *Methods of testing soil for engineering purposes – Method 3.4.1: Soil classification tests – determination of the linear shrinkage of a soil – standard method* (Australian Standards 2008).

The target mass fraction of materials passing the 0.425 mm sieve size, as shown in Table 4.2 was mixed at OMC with different cement contents. Upon mixing of the material components at OMC, the material was placed within a linear shrinkage mould, as shown in, and allowed to cure in a moisture chamber. After the specimens were removed from the moulds, the strain was then measured using a scaled ruler.

Figure 4.14: Linear shrinkage mould

4.2.9 Wheel Tracking Test

The Wheel Tracking Test was carried out in accordance with the recommendations of Austroads' *Deformation resistance of asphalt mixtures by the wheel tracking test* (Austroads 2006) along with supplier specifications and guidelines.

Cement treated crushed rocks were first prepared to 100% MMDD and poured into steel moulds measuring 50 mm (height) x 400 mm (width) x 400 mm (length). The mixtures were then loaded into a Cooper Compactor and applied with runs of compaction force as shown in Table 4.4 below.

Table 4.4: Compactive effort applied to slab specimens

Stage	1	2	3	4
Number of Runs	7	14	14	14
Pressure (kPa)	5	10	10	10

Figure 4.15: Cooper Compactor

Specimens were cured in sealed bags in a moisture bath for 7 days. The specimens were then soaked for 4 hours before being loaded onto the Cooper Wheel Tracker machine as shown in Figure 4.16.

Figure 4.16: Cooper Wheel Tracking Test Machine

The specimens were subjected to 5000 cyclic loadings where the total deformation of the surface was measured by an LVDT built into the wheel load. Readings were taken by the testing machine and recorded by a linked computer.

4.3 Curing Method

As the curing method plays a significant role in the development of the material characteristics of cement treated materials, this section discusses in some greater detail the curing regime applied in this dissertation.

As can be seen in the previous section, two distinct curing methods were applied, i.e. a curing chamber method and a wrapped cured method. The curing chamber method involved the placement of specimens in a curing chamber (developed by the author), which allowed the relative humidity and temperature to be monitored and kept consistent. Specimens were stored in a sealed and insulated containment unit and supplied with water vapour consistently throughout the curing process as shown in Figure 4.17 below.

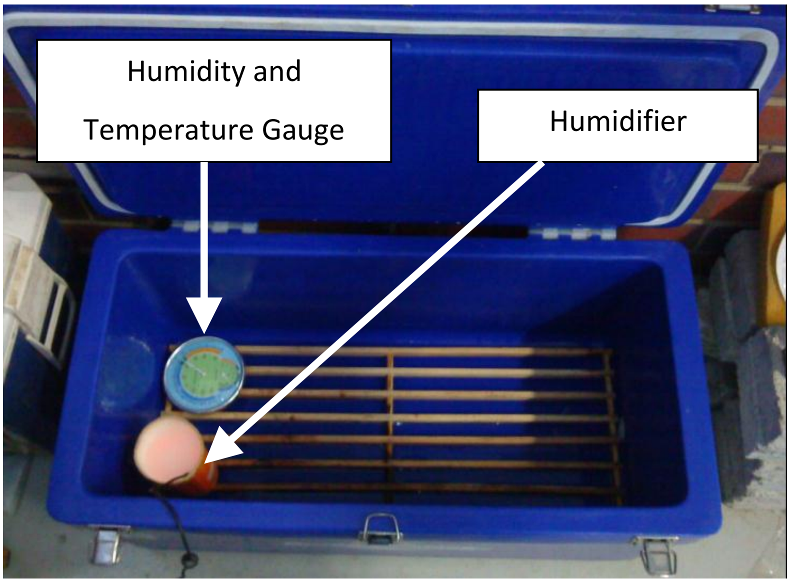


Figure 4.17: Curing chamber

The wrapped curing method is a methodology used typically by MRWA and involves specimens being encased in clear wrap or a sealing material, and then being stored in an environment kept at a constant temperature. This prevents water from evaporating from the specimens and is a simple but effective method to control the curing process.

The summary of the curing procedures adopted for each test is presented in Table 4.5.

Table 4.5: Curing method for tests

Test of characteristics	Duration of curing	Curing method
Unconfined Compressive Strength	28 days	wrapped cured
Indirect Tensile Strength	7 days	moisture chamber curing
	28 days	wrapped cured
Flexural Bending	7 days	moisture chamber curing
Flexural Fatigue	7 days	moisture chamber curing
Tube Suction	7 days	moisture chamber curing
Nitrogen Adsorption	1 day	wrapped curing
	7 days	wrapped curing
Linear Shrinkage	1 day	moisture chamber curing
Wheel Tracking	7 days	wrapped curing

The curing regime was selected based on both practicality and the limitations of the laboratory facilities at Curtin University (see Section 7.2 for further details).

4.4 Summary of research methodology displayed graphically

This dissertation involves an extensive and elaborate testing regime in order to classify cement treated basecourse in Western Australia. In order to facilitate the understanding of the reader, this section summarises each of the tests graphically as shown in Figure 4.18 below.

This chapter has provided the materials and methodology involved in the completion of this research. The data collected from these tests and its interpretation is presented in the subsequent chapter.

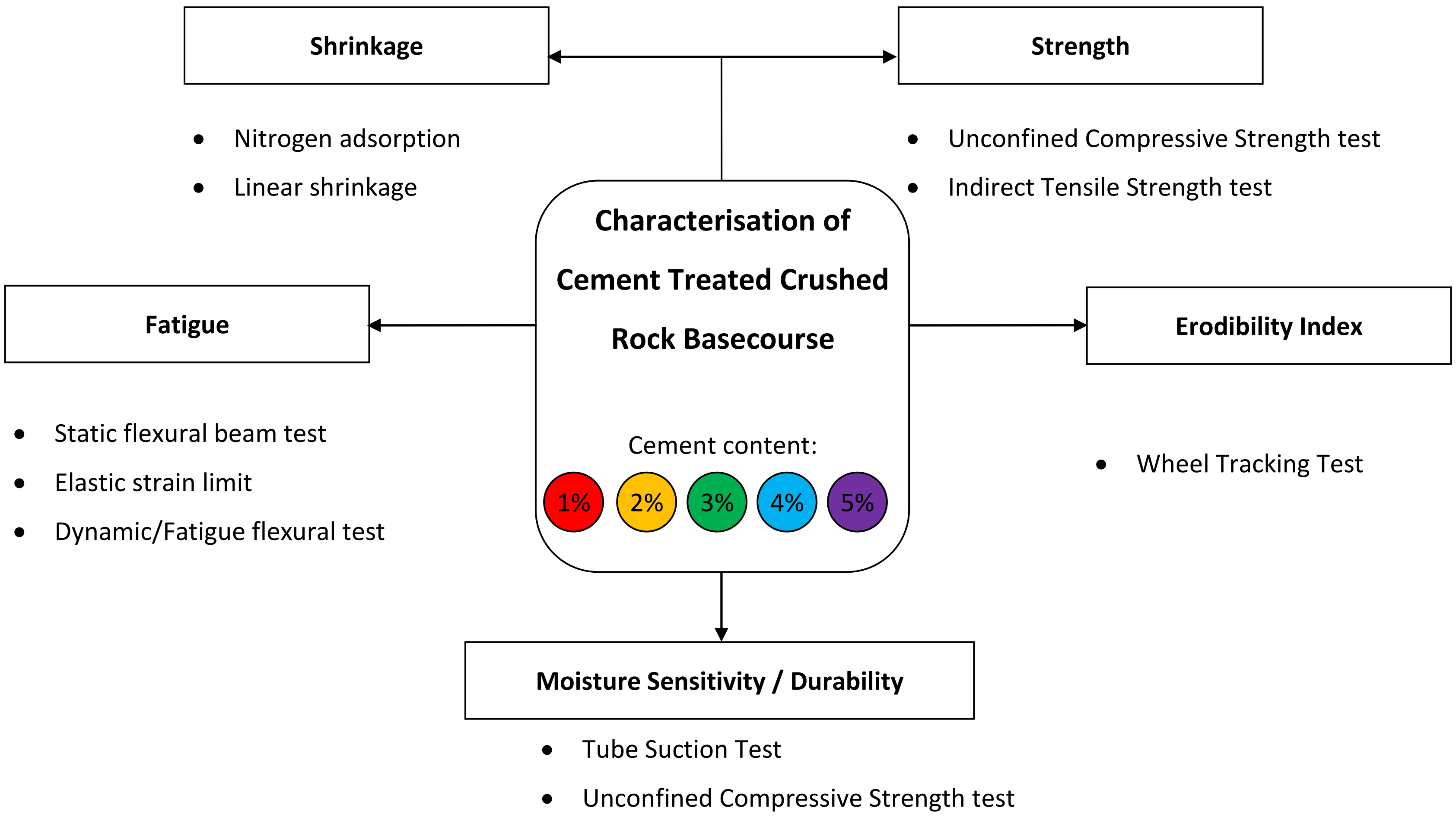


Figure 4.18: Summary of research methodology

5 Experimental Results and Analysis

This chapter presents the experimental data collected from the laboratory investigations discussed in the previous chapter, and the interpretation thereof. The chapter is made up of sections displaying normalised laboratory results, description of the data presented, and an analytical discussion of the data on changes in cement content, along with other observations of the data presented. The raw data of each test is not presented in this dissertation and is available in the compendium.

In addition to the laboratory tests highlighted in the previous chapter, this chapter also presents the results of the compaction test.

5.1 Modified Proctor Compaction Test

The Maximum Modified Dried Density (MMDD) and Optimum Moisture Content (OMC) for cement treated crushed rock is determined in this test. The test results showing the modified dry density vs. water content for triplicate specimens of materials treated with 2%, 4% and 6% cement content are shown in Figure 5.1. The MMDD and corresponding OMC are determined based on the best fitting curve. The water-cement (w/c) ratio is also determined from the experimental results.

Cement treated crushed rock treated with 2%, 4% and 6% cement content by mass is assessed, while 1%, 3% and 5% are interpolated from these results. The results of the compaction tests are summarised in Table 5.1 below.

Table 5.1: MMDD, OMC and w/c ratio for various cement content

Cement Content	1%	2%	3%	4%	5%	6%
MMDD (g/cm^3)	2.348	2.347	2.346	2.345	2.344	2.343
OMC (%)	5.75	6.00	6.25	6.50	6.75	7.00
w/c ratio	5.75	3.00	2.08	1.63	1.35	1.17

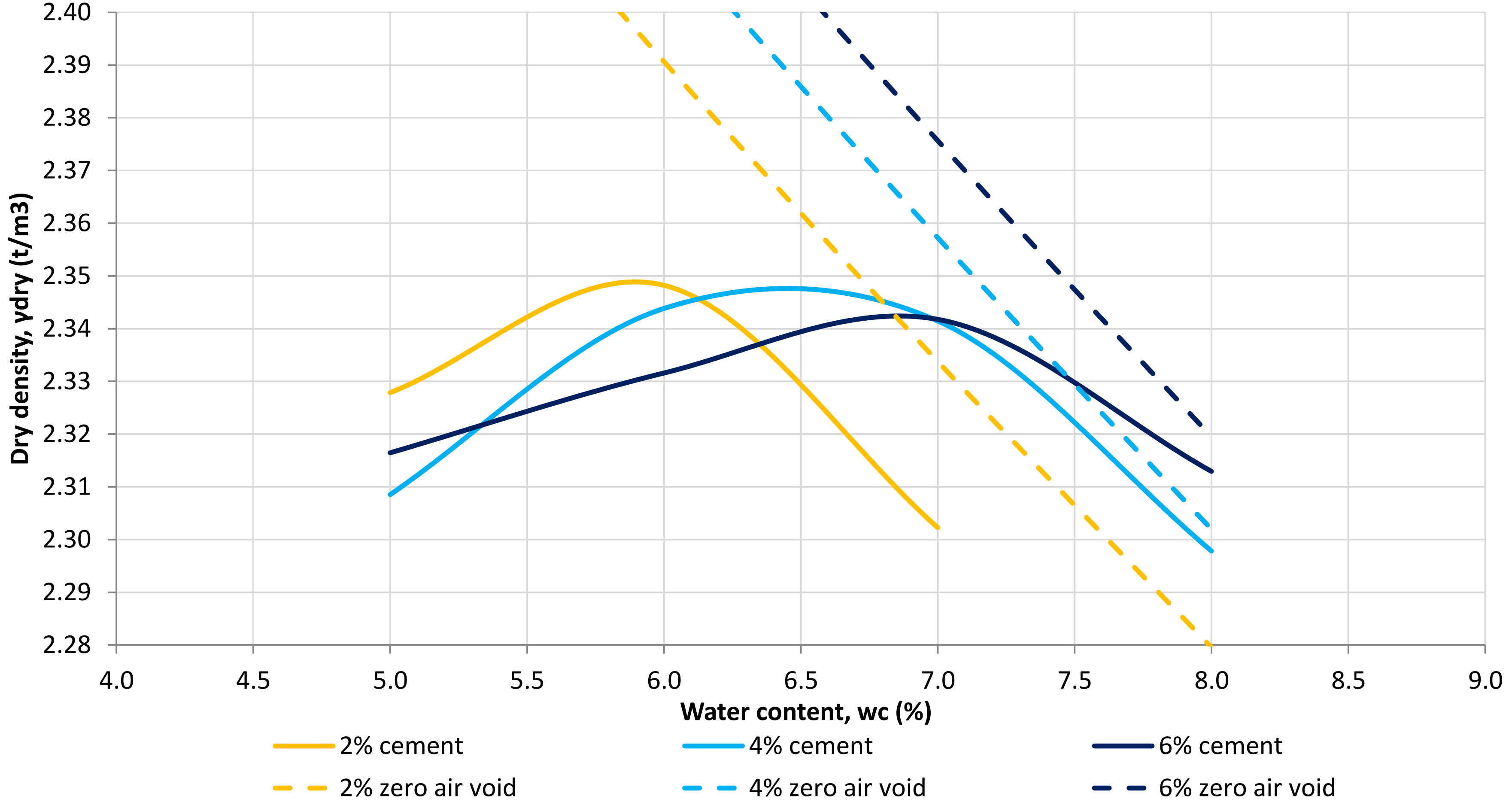


Figure 5.1: Modified Dry Density (t/m^3) vs. Water Content (%)

It can be seen that the modified dry density of the materials remains near constant, but the optimum moisture content increases by 0.25% for every 1% increase in cement. There are two possibilities for this:

i. the water is absorbed by the constituents of cement for the hydration process. The 0.25% increase for every 1% of cement also corresponds well with the water requirements for cement content, as discussed in Section 2.4, where 25% water is required for the hydration process of cement.

ii. fines contents are increased, and thus the specific area of the material increases and subsequently more water is retained by the specimen. Although cement promotes the coagulation of fines into a continuous matrix, the compaction is carried out immediately after mixing and therefore does not

The water required for the cement reaction is selected independent of its requirements for cement reaction but instead aims to promote density. This inherently means that the materials are primarily dependent on roller compaction to promote material interlocking, as opposed to conventional concrete which depends more on the development of a cement matrix to bind aggregates and sand. This interrelationship is discussed further in the subsequent chapter.

5.2 Strength Test Results

The strength test results focused on the experimental data acquired from Unconfined Compressive Strength (UCS) and Indirect Tensile Strength (ITS) tests. These results are further used to assess the stress envelope of the material to determine the indicative shear parameters of the material.

5.2.1 Unconfined Compressive Strength (UCS)

The normalised unconfined compressive stress vs. vertical strain curves for cement treated crushed rock with 2% to 5% cement content by mass is shown in Figure 5.2 below. The data represents the average values of triplicate samples.

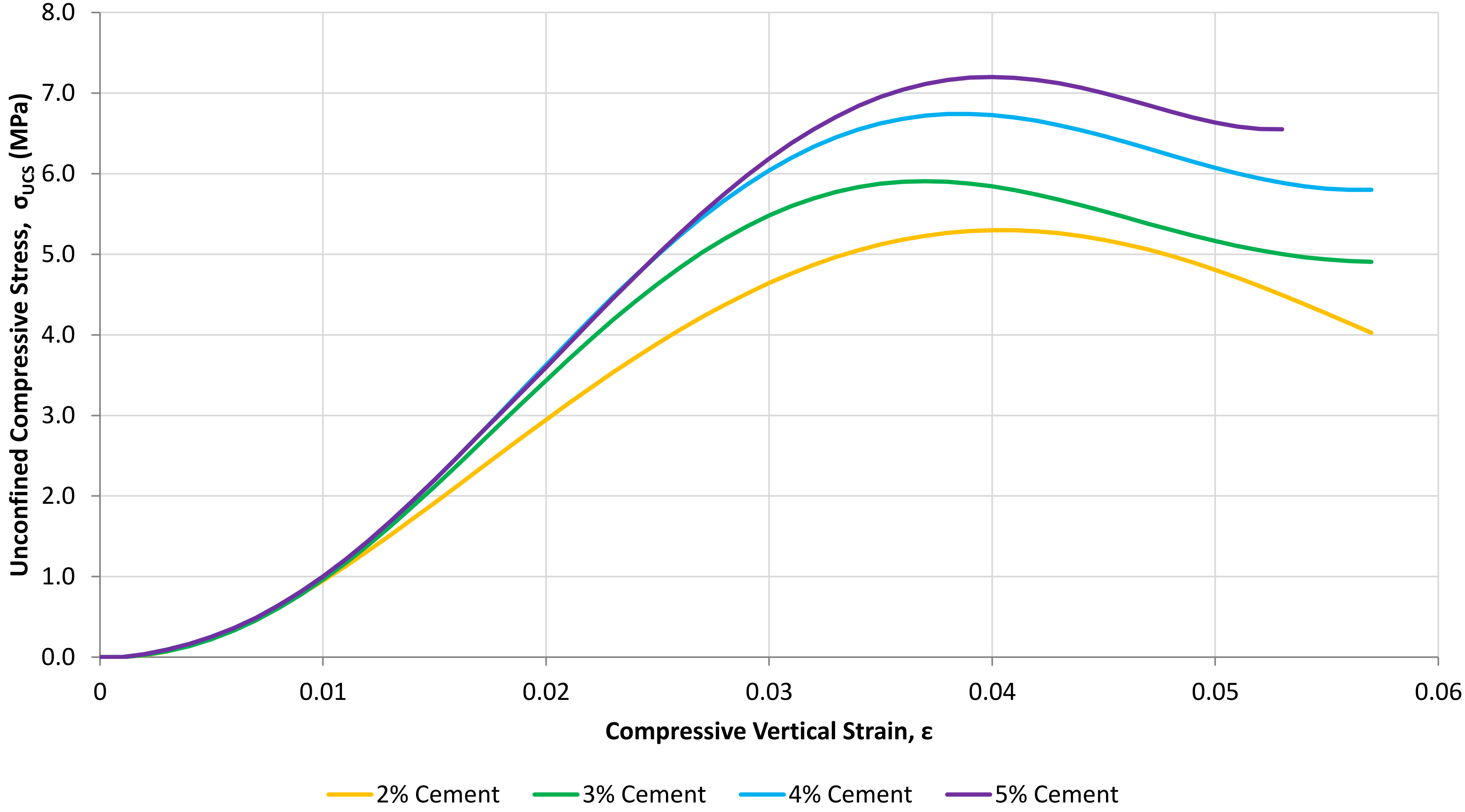


Figure 5.2: Unconfined Compressive Stress, σ_{UCS} (MPa) vs. vertical strain, ε

The applied stress, σ_{UCS} vs. strain, ε_{UCS} curves are typically characterised by a linear section followed by a plateau prior to failure. Besides obtaining the UCS magnitude, which is measured as the maximum stress value from the test, the unconfined compressive modulus, E_{UCS} of the specimen is also determined. The compressive modulus is taken as the gradient of the linear section of the stress strain graph, as shown in Figure 5.2. The UCS tests results are summarised in Table 5.2.

Table 5.2: UCS for various cement content

Cement Content	1%	2%	3%	4%	5%
UCS (MPa)	4.65*	5.30	5.91	6.74	7.20
E_{UCS} (MPa)	-	202.1	256.3	279.0	277.0

*result extrapolated from data set – sample not available

The 1% cement content mix has not been presented due to ambiguous readings which were inconsistent with the data collected. This is further covered in a subsequent chapter of this dissertation which focuses on the limitations of the laboratory work undertaken. The result for the 1% cement content mix has instead been extrapolated from the data set of 2% to 5% cement content by mass. By plotting the UCS vs. cement content as presented in Figure 5.3 below, a linear relationship can be seen between the two parameters.

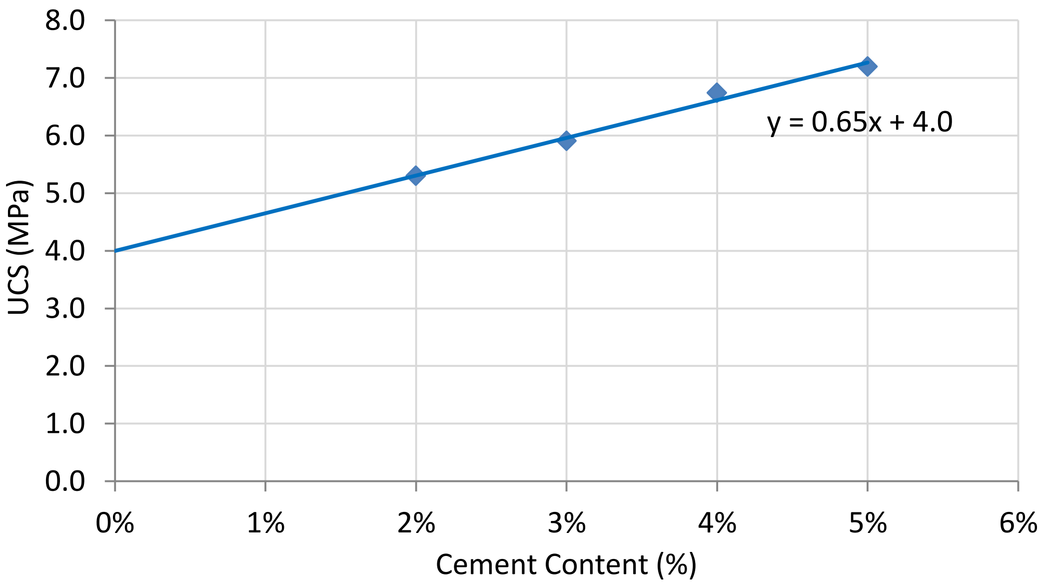


Figure 5.3: Unconfined Compressive Strength, UCS (MPa) vs. cement content (%)

It is clearly noted from Figure 5.3 above that an increase in cement content increases the UCS of the material within the tested cement content range of this dissertation. UCS increases by 0.65 MPa with every 1% in cement content. This linear relationship supports the contention that UCS does have a close relationship with cement content, and therefore it can be used as a quality control measure to determine consistency and benchmarking regarding cement content. However the ranges are grossly in excess of the ranges proposed by Austroads, as discussed in Section 2.2.

By extrapolating the data to an untreated state (0% cement), i.e. the y-intersect, the unbound crushed rock base course is shown to possess an inherent magnitude of 4.0 MPa UCS. This is an inherently high value that is questionable and is more than the 0.7 MPa "modified" limit and the 1.5 MPa "bound" limit as discussed in Section 2.2. This suggests that the material is heavily bound even without treatment. This is of course not the case, thus again substantiating the lack of reliability of UCS as a pure measurement of behaviour.

On a similar note, the UCS results taken from the tests also suggest that even with a cement treatment as low as 1% cement content, the material behaves as a "bound" material, i.e. the UCS is more than 1.5 MPa. This highlights the injudicious application of UCS as a primary classification criterion for cement treated crushed rock, and it also explains the decision of MRWA to apply cement treatment with as low as 0.5% cement content to roads in order to avoid using "bound" materials, as covered in Section 2.5. The discrepancy within the ranges recommended by Austroads further suggests that UCS varies from material to material, as it depends on Poisson's ratio, elastic limit and specimen treatment and the preparation methods as discussed earlier in Section 3.3. This means that a national blanket rule, based on UCS, is perhaps not appropriate, given these inaccuracies.

Given that the reason for classification is principally intended for the determination of failure criterion, and subsequently to select an appropriate design methodology, there is no distinct trait in cement content vs. UCS which may indicate a specific delineation point. The UCS merely provides a benchmark for a certain mix design, which makes it a useful measure for quality control purposes during construction, but its relationship to specific failure modes is not apparent. Having a strong compressive strength as suggested by the results, does not immediately imply that the material will undergo fatigue duress and shrinkage. Section 5.3 discusses these parameters in greater detail.

Figure 5.4 shows that the relationship between cement content and compressive modulus is polynomial in nature.

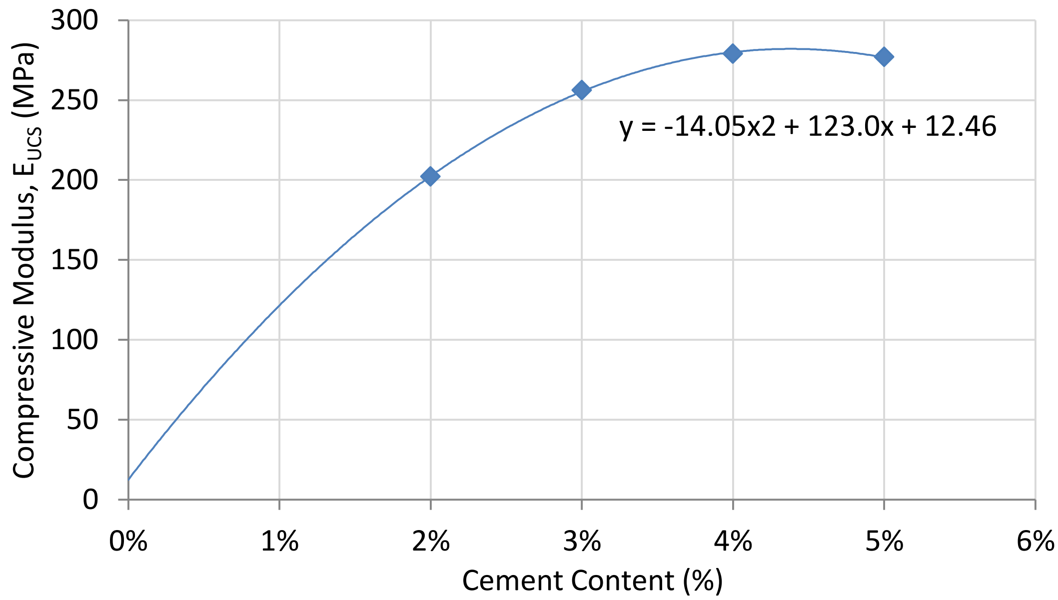


Figure 5.4: Compressive modulus, E_{UCS} (MPa) vs. cement content (%)

The change in gradient suggests that when crushed rock base course is treated with more than 3% cement content, the appreciation of modulus reduces and the material will undergo similar deformation rates when compressed, despite increasing its ultimate strength. The compressive modulus provides a better indication of the brittleness of the material and is an indication of the material's susceptibility to cracking.

Furthermore, the extrapolated results show that the compressive modulus for untreated materials and 1% cement content mixes is very low and susceptible to excessive deformation under loading. The elastic modulus of the untreated and 1% cement content mixes are approximately 12.5 MPa and 121.4 MPa respectively.

5.2.2 Indirect Tensile Strength Test (ITS)

The Indirect Tensile Strength (ITS) of cement treated crushed rock with 1% to 5% cement content by mass cured for 7 days and 28 days is shown in Table 5.3 below. The data represents the average values of triplicate samples.

Table 5.3: ITS results for various cement content

Cement Content	1%	2%	3%	4%	5%
$ITS_{7\text{-day}}$ (MPa)	0.047	0.153*	0.259	0.324*	0.389
$ITS_{28\text{-day}}$ (MPa)	0.241*	0.466	0.692	0.806	1.111

*result interpolated/extrapolated from data set

Similarly to UCS, when plotting ITS vs. cement content as shown in Figure 5.5 below, a linear relationship can be seen between the two parameters within the tested cement content range of this dissertation. The ITS for specimens cured for 28 days increased by 0.218 MPa with every 1% in cement content and was approximately one third of UCS while the ITS for specimens cured for 7 days increased by 0.08 MPa with every increase of 1% in cement content constituting approximately one third of the ITS gained at 28 days.

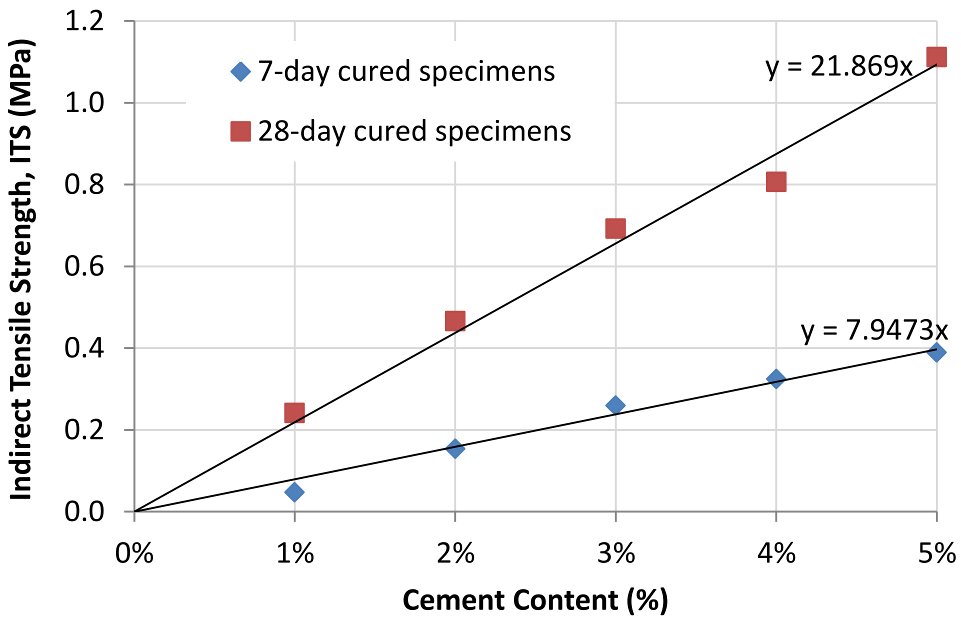


Figure 5.5: Indirect Tensile Strength, ITS (MPa) vs. cement content (%)

By extrapolating the results for both the 7-day and 28-day data set, the y-intersect of the graphs, which represent the untreated unbound granular condition, shows that the material does not possess tensile strength, agreeing with the phenomenology of the material. This supports the view that the ITS provides a good quantification methodology for the degree binding achieved from the cement treatment. Nonetheless, the valuable difference between ITS and UCS is that it is a parameter that can be used for the characterisation of road flexural response as per the structural model presented in Section 3.2.

Furthermore, based on guidelines used in South Africa, as presented previously in Section 2.4, the appreciable tensile strength defining bound behaviour is 0.12 MPa for specimens after 28 days of curing, which in the context of the above results for $ITS_{28\text{-day}}$ suggests that materials with a minimum of 0.55% cement behave as a bound material. Nevertheless, by comparing that limit against the 7-day cured data set, the delineating point for bound material will be 1.5% cement content and more.

Similarly to UCS, despite potentially quantifying the degree of binding of a material, the limit applied in South Africa may not necessarily serve the purpose of dictating the failure mechanism of the material – which, as per Austroads design guidelines, is based on fatigue and shrinkage. These results do however provide information on the behavioural characteristics of the materials and thus they may be used as design parameters in determining pavement responses through numerical modelling works, as presented in the next chapter.

5.2.3 UCS, IDT and the Stress Envelope

Based on the UCS and ITS results above, the internal angle of friction and cohesion can be determined by plotting a Mohr circle envelope. The Mohr circle envelopes for 1% through to 5 % cement content derived from the UCS and ITS results are presented in Figure 5.6 below, where the compressive stress is plotted on the positive scale of the x-axis and the tensile stress on the negative scale of the x-axis.

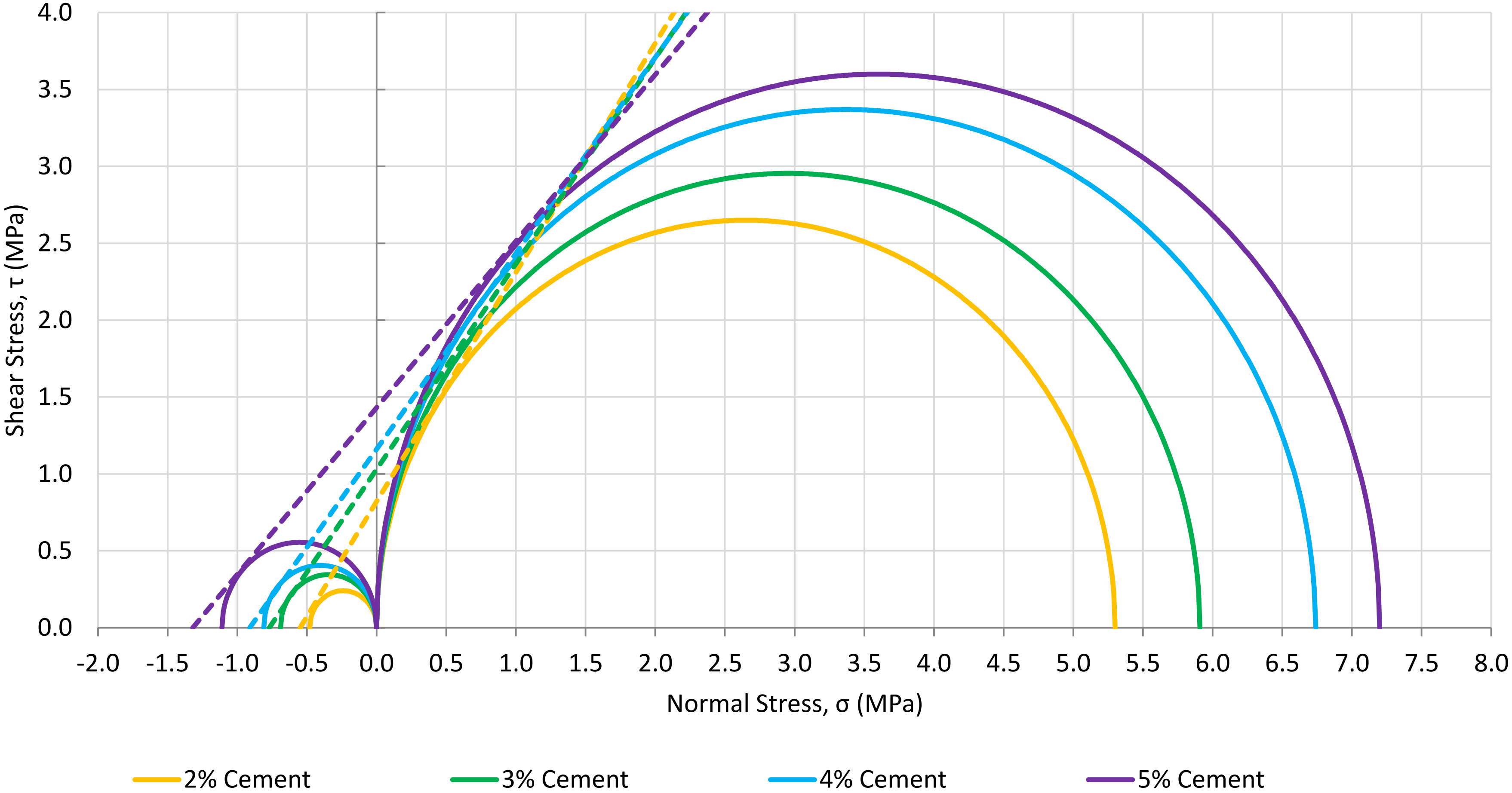


Figure 5.6: Mohr circle for cement treated crushed rocks with 2% to 5% cement content by mass

From the Mohr circles shown, the approximated cohesion, c_{approx}, denoted as the y intersect of the tangent intersecting both the UCS and ITS envelopes and the approximate internal angle of friction, $Ø_{approx}$, denoted as the angle of the tangent line to the x-axis are summarised in Table 5.4 below

Table 5.4: Shear parameters of cement treated crushed rock basecourse

Cement Content	1%	2%	3%	4%	5%
Ø (^{o})	58.9*	56.1	53.2	51.9	47.3
c (MPa)	0.62*	0.82	1.03	1.16	1.43
σ_c tan Ø	7.7	7.9	7.9	8.6	7.8
τ	8.3	8.7	8.9	9.8	9.2
%c	7%	9%	12%	12%	15%

*result extrapolated from data set – sample not available

The approximated internal angle of friction decreases linearly with the increase in cement content by 2.74^o for every 1% increase in cement content, as shown in Figure 5.7 below.

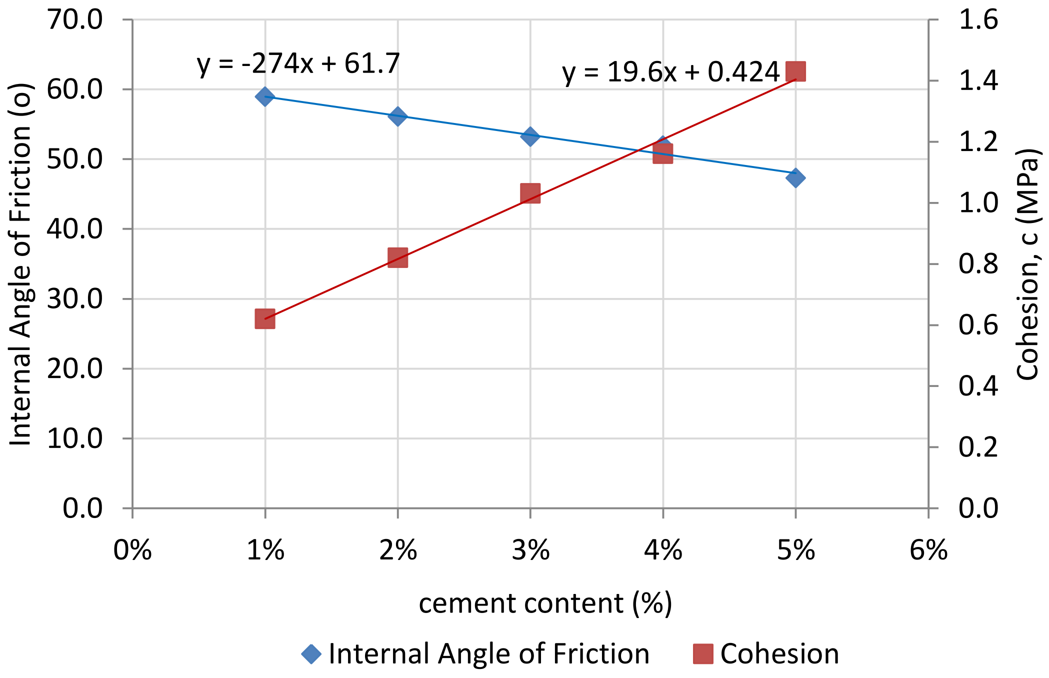


Figure 5.7: Shear parameters vs. cement content

This reduction suggests that dependency on the shear interlock of aggregates is reduced, due to the volumetric increase of a cement matrix between aggregate contact interfaces. The cement matrix instead provides a cohesive strength, increasing by 0.196 MPa as the volumetric content of the cement increases by 1%, as shown in Figure 5.7.

A limitation of this methodology is evident when the data set is extrapolated to include untreated samples – when ITS = 0 MPa and thus c_{approx} = 0, the internal angle of friction would be 90^{o} should the premise be that the confining pressure = 0 MPa (unconfined). This backs up the contention that the values ascertained from the stress envelopes are simply approximates, and therefore shows only an indicative behavioural response of cement treated crushed rock materials. It also demonstrates that the UCS does not have any mechanistic significance.

Furthermore, by calculating the shear strength components of the material as shown in Table 5.4 above, the interlock nature of the material remains generally constant except for 4% cement content which increased by 0.7 MPa. This again supports that the primary strength gained from the addition of cement content is predominantly through the binding nature of the cement matrix. This is mathematically indicated by the % of the shear strength dependent on the cohesion component shown in Table 5.4.

5.2.4 Summary of Strength Parameters

The UCS and ITS results show a good relationship to cement content by mass, but do not distinctly outline the in-service behaviour of the material. The UCS shows some level of ambiguity with magnitudes grossly in excess of expected values, and it does not provide any evidence of mechanical substance for pavement structural design, accentuating its unreliability as a mechanism for classification. Nonetheless, a trend of the compressive modulus shows that the material reaches a plateau in its modulus, at 3%, signifying a change in behaviour. The Mohr circle envelope results also show that the dependency on inter-particle interlocking decreases and is substituted by a cohesive force generated from the volumetric increase in cement when cement content is increased.

5.3 Flexural Behaviour

5.3.1 Flexural Bending Test (FBT)

The flexural bending stress vs. strain relationship for cement treated crushed rock with 1% to 5% cement content is shown in Figure 5.8 below. The data represents the average values of triplicate samples.

The flexural stress, σ_f vs. strain, ε_f curves are typically characterised by a largely linear increase in ultimate Flexural Bending Strength, FBS followed by a sharp decline. The FBS is taken as the maximum stress of the test, while the flexural modulus, E_f of the specimen can also be determined by obtaining the gradient of the linear section of the curves. Furthermore, the chart provides the assessment of the strain at 95% of loading which is a parameter in the characterisation of fatigue distress, as discussed in Section 3.4. The FBT tests results are summarised in Table 5.5.

By considering the equation 3.19 as presented in Section 3.4, the mathematical approximation of the equivalent elastic strain ratio at failure stress, *p-value,* can be estimated.

$$\sigma_f = \sigma_t \frac{4(0.5h - z)^3}{p\left[h^2(0.5h + z)\right]} \qquad (3.19)$$

where, h = 50 mm

z = 0mm

Table 5.5: FBT results for various cement content

Cement Content	1%	2%	3%	4%	5%
FBS (MPa)	0.498	0.805	1.383	2.048	2.195
$\mu\varepsilon_{f\text{-breaking}}$	2346.0	1493.4	1262.8	1480.7	1098.5
p-value	48.4%	57.9%	50.0%	39.4%	50.6%
$E_{f.static}$ (MPa)	438.6	931.0	2190.3	3510.5	3949.0

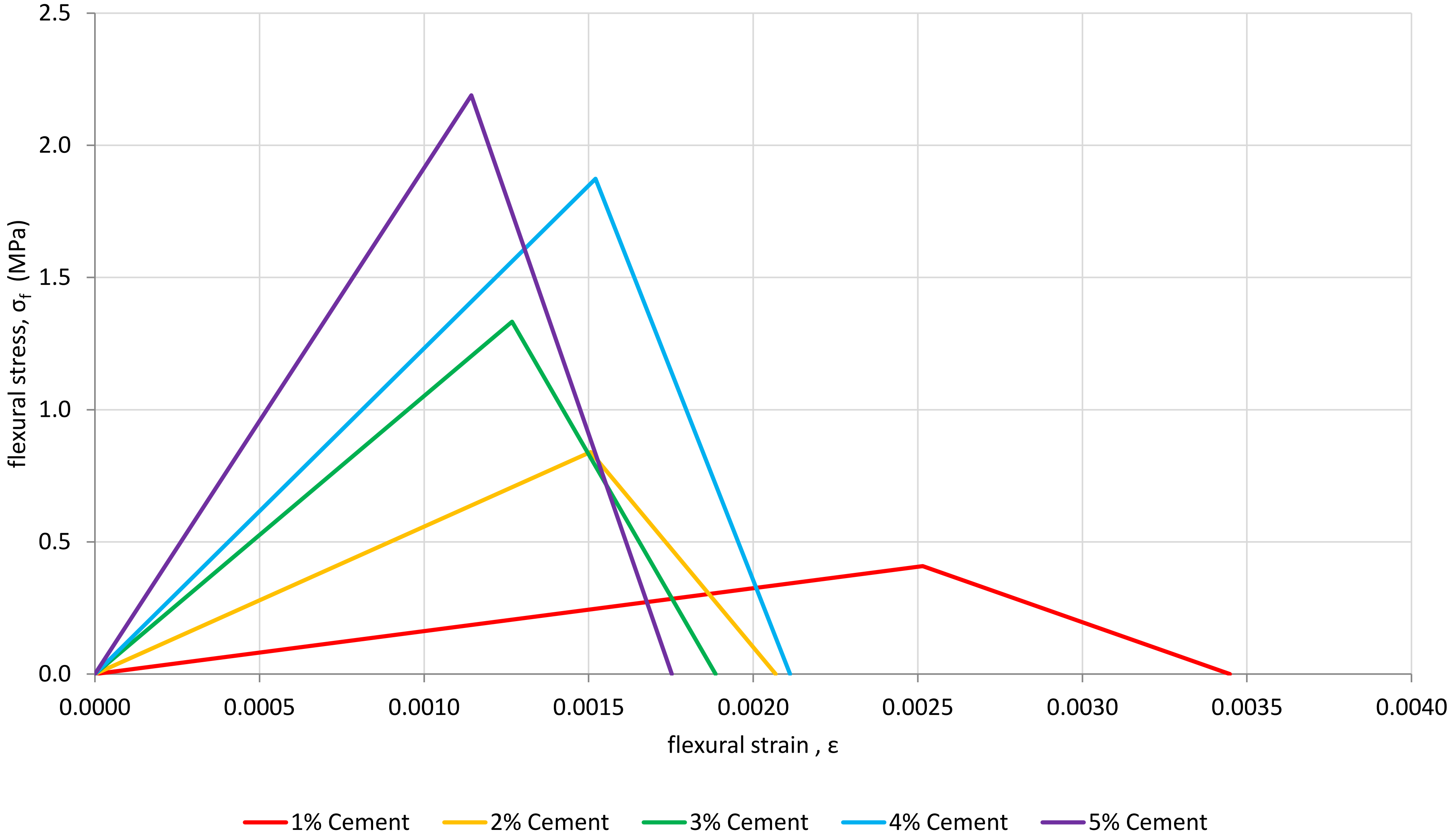


Figure 5.8: Flexural stress, (MPa) vs. strain

The FBS increases linearly with cement content as shown in Figure 5.9 below. This implies that the strength of the pavement to withstand higher bending or flexural forces increases with cement content as anticipated. The flexural strength increases by 43.4 MPa for every 1% increase in cement content. Similar to Indirect Tensile Strength, the y-intersect of Figure 5.9 shows that for untreated specimens, the materials exhibit no flexural strength.

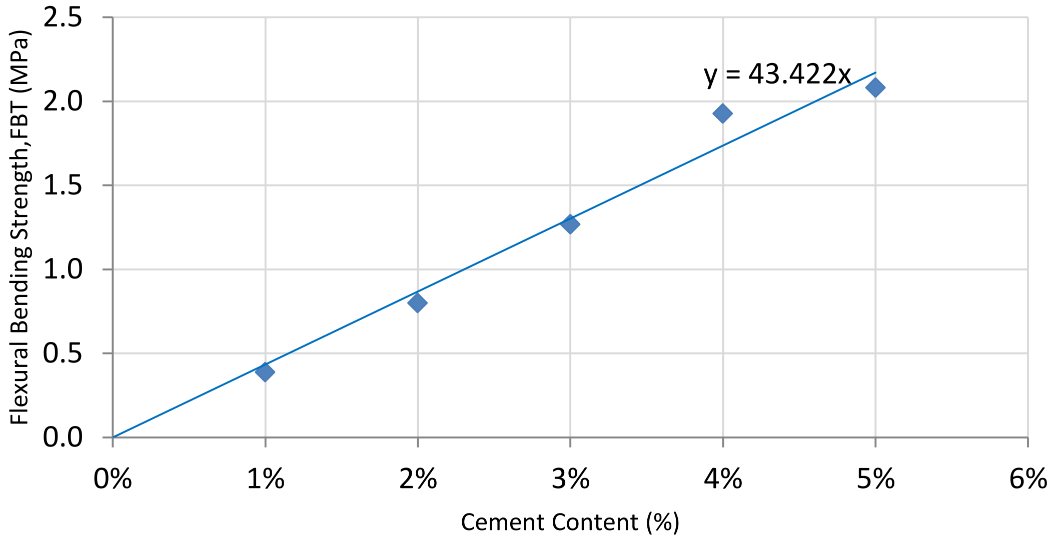


Figure 5.9: flexural beam strength, (MPa) vs. cement content (%)

In comparing the above results to the indirect tensile strength, it can be seen that the flexural bending is twice the value of the indirect tensile strength. This is primarily due to the influence of the elastic limit and the geometry of the specimen. The elastic limit of the specimen

Furthermore, the tests show that the breaking strain decreased exponentially with the increase in cement content, as shown in Figure 5.10 below. A sharp decrease in breaking strain can be observed between 1% and 2% cement content, followed by a relatively small reduction between 2% through to 5% cement content. A rebound was noted in the 4% triplicate samples, which can be attributed to variability in the testing results.

The p-values calculated from the test also conform with the observations by ARRB, where appreciable fatigue deterioration is noticed when a load of more than 50% of the maximum load is applied. The calculated p above, also suggests that the 4%

cement content mix provides the least resistance to fatigue damage, due to its susceptibility to accumulate damage even at lower strain ranges.

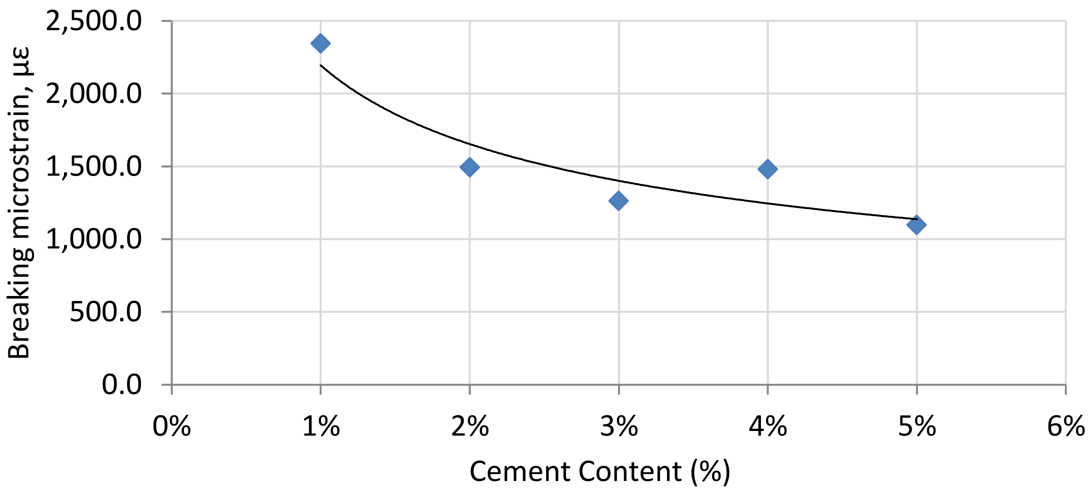


Figure 5.10: Breaking strain, ε vs. cement content(%)

The empirical relationship of cement content and breaking strain is exponential and shows a significant increase in brittleness of cement treated crushed rock beyond the 2% cement content mark. However, since the flexural strength and modulus increase linearly, the strength of the pavement to undergo flexural strain decreases disproportionately with increase in cement content. A tipping point can therefore be assessed, where the flexural modulus of the material can minimise the accumulation of fatigue, based on the mathematical model presented in the following chapter.

Moreover, since the onset of damage, which when accumulated becomes fatigue, is a function of the breaking strain as suggested in Section 3.4, the breaking strain results are further analysed in the subsequent Section.

5.3.2 Dynamic Testing – ELT and FFT

As discussed in the previous section, the dynamic flexural testing regime comprised two sets of laboratory investigations, which included the elastic limit test to assess the onset of damage under flexural loads, and the repeated flexural tests to assess fatigue behaviour. The subsections below presents the results from the two tests.

5.3.2.1 Elastic Strain Limit Test

Figure 5.11 below shows the flexural stiffness, S_f (MPa) vs. number of cycles, N curve for incremental strain stages (magnitude of applied microstrain denoted at the top of each 600 cycle stage) for crushed rock treated with 1% to 5% cement content by mass.

The results show an exponential reduction in stiffness with the increase in strain. However, for cemented materials that are susceptible to fatigue, damage in the specimens within the tested strain range can be observed. This damage is characterised by a reduction in stiffness within a tested applied strain stage. No appreciable strain limit within the testing range of up to 100 microstrains has been observed for specimens with less than 3% cement content. The onset of damage to crushed rock with cement content of 3%, 4% and 5% occurs when 85 με, 55 με, 75 με are applied respectively, as summarised in Table 5.6. The increase in stiffness from cement treatment, results in the tendency of the material to undergo damage, for materials treated with 3% cement content or more. However, at 5% cement content, the material has gained sufficient stiffness to resist bending before undergoing damage.

The results also show a significant dispersion of data for low strain stages. This dispersion is due to the fluctuations in the pressure of the testing rig, and the sensitivity of the specimen at lower strain stages. Furthermore, since the intent of the test was to ascertain the onset of damage, the test had in certain instances, been allowed to run in excess of 600 cycles to 1200 cycles to confirm the observed reduction in damage. This resulted in a cease in continuity of stiffness between each strain stage, as noted in Figure 5.11.

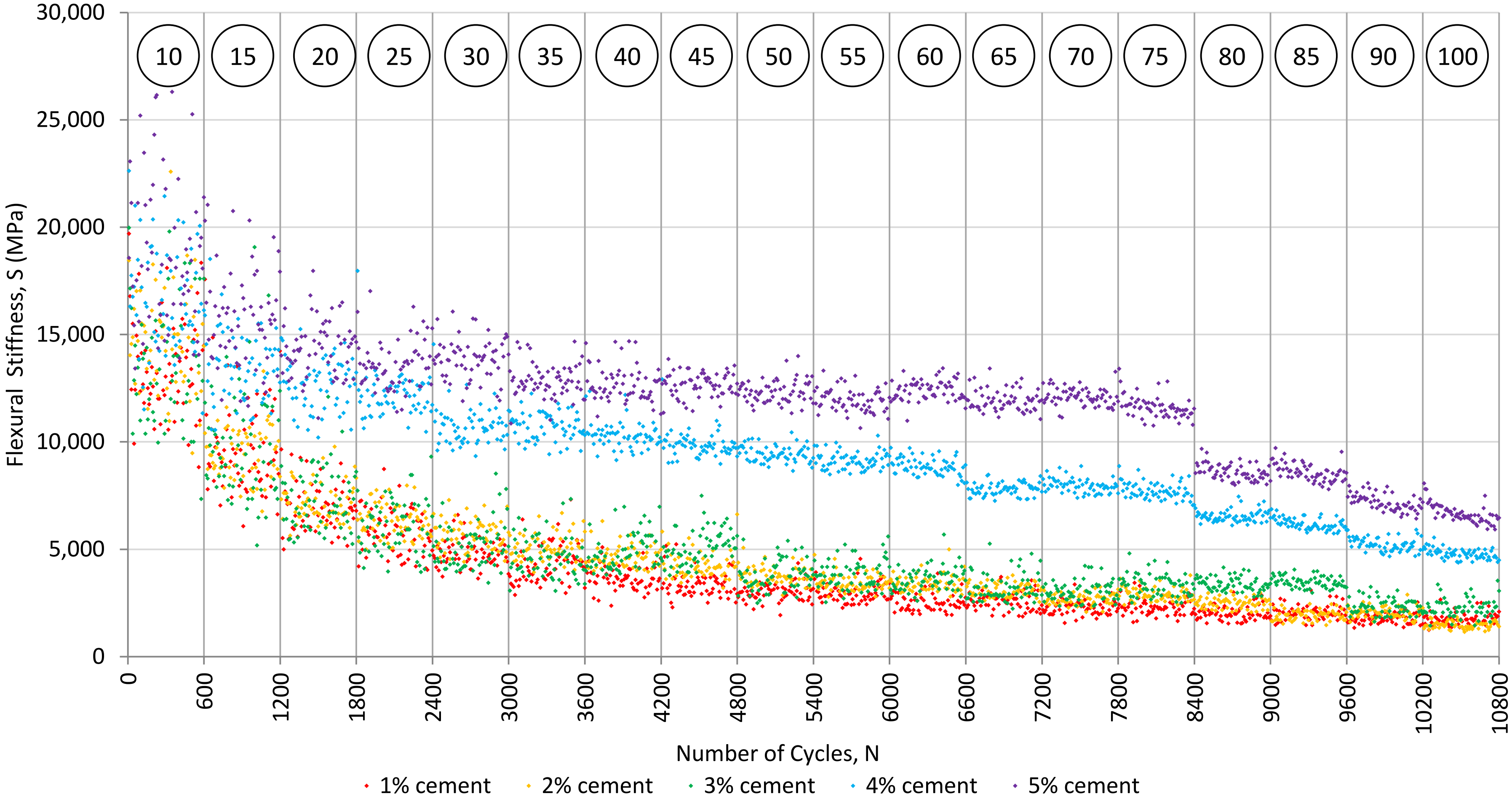


Figure 5.11: Incremental strain stages vs. flexural stiffness

Table 5.6: Elastic Strain Limit test result summary

Cement Content	1%	2%	3%	4%	5%
$\varepsilon_{crack\text{-}initiation}$	N/A	N/A	85	55	75
% breaking strain	N/A	N/A	6.7%	3.7%	3.8%
Applied stress at crack initiation	N/A	N/A	0.294	0.494	0.856
p value	N/A	N/A	21%	24%	39%

Furthermore, by taking the percentage of the applied stress at the onset of damage to the ultimate stress, as presented in Table 5.5, the *p* value is assessed, as shown in Table 5.6. The *p* values measured are much less than the values calculated in the previous section, but they are within the range proposed by Department of Transport (Deparment of Transport 1986) (see Section 3.3). This explains that the cease in linearity of materials is more strain dependent than stress dependent.

With the known onset of fatigue damage, a fatigue test was thus run for 75 με, 200 με and 400 με to assess the response of the material over a wider spectrum of applied strain.

5.3.2.2 Repeated Fatigue Load Test

The repeated fatigue load tests were undertaken for three different constants of applied strain, i.e. 75 με, 200 με and 400 με as discussed previously. Figure 5.12, Figure 5.13 and Figure 5.14 below show plots of these three different applied strain values based on the averages of samples undertaken. Subsequent subsections present discussions of these plots as well as the calculated Load Damage Exponent and the fatigue life (determined as the number of cycles required to reach 50% of the initial modulus).

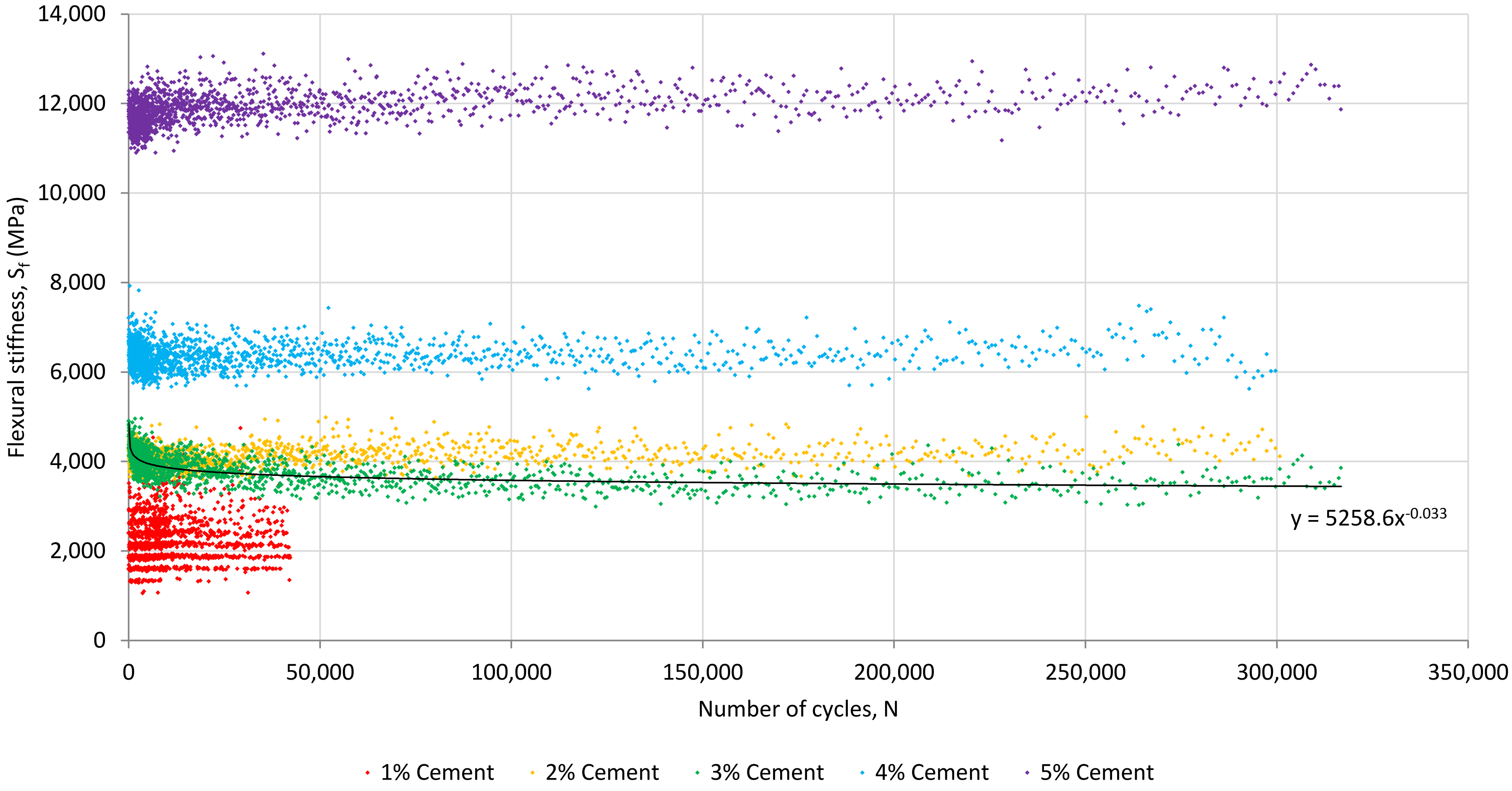


Figure 5.12: Flexural stiffness, S_f vs. cycle, N (S-N) curves for cement treated specimens under 75 με constant applied strain

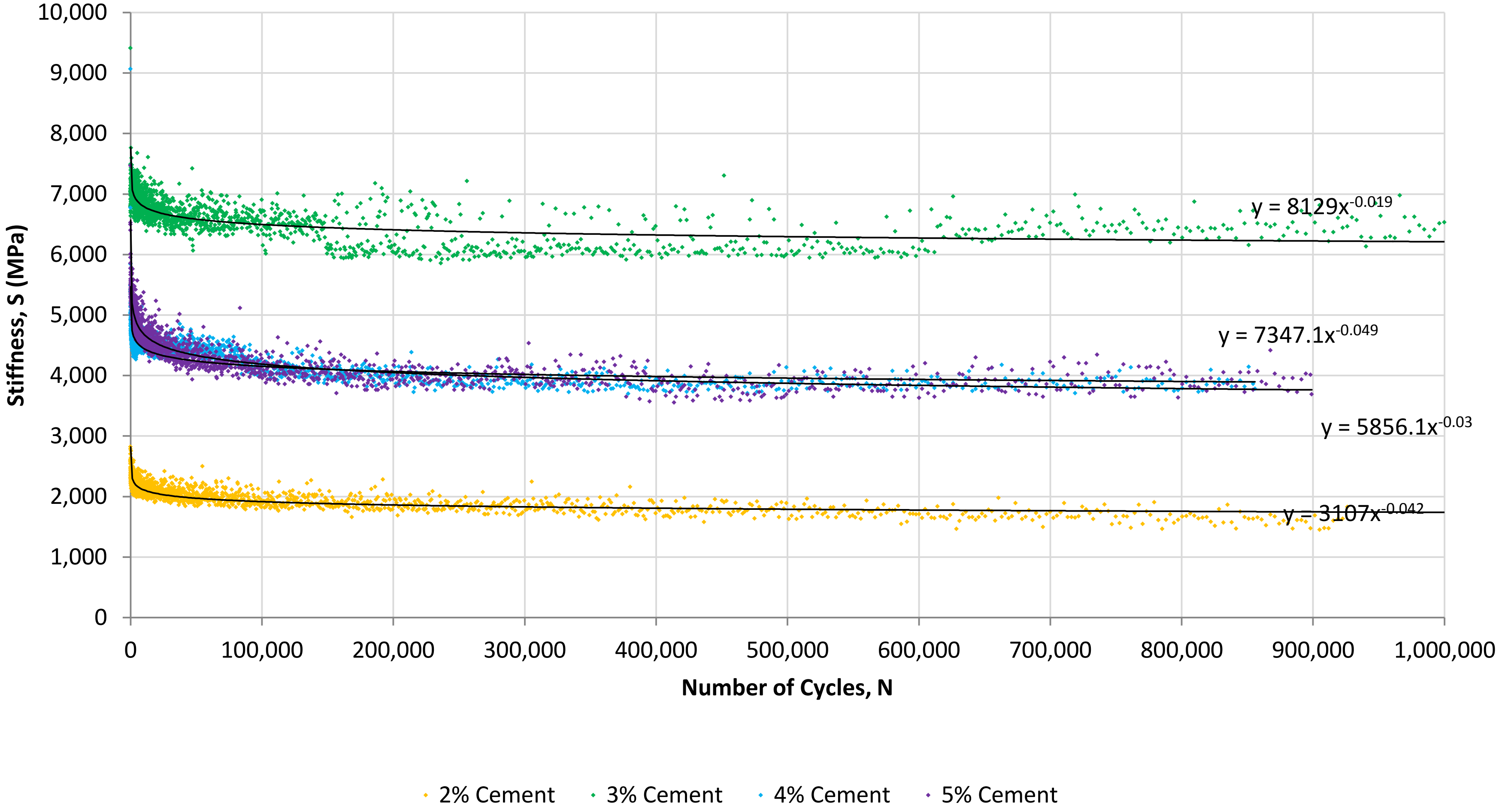


Figure 5.13: Flexural stiffness, S_f vs. cycles, N (S-N) curves for cement treated specimens under 200 με constant applied strain

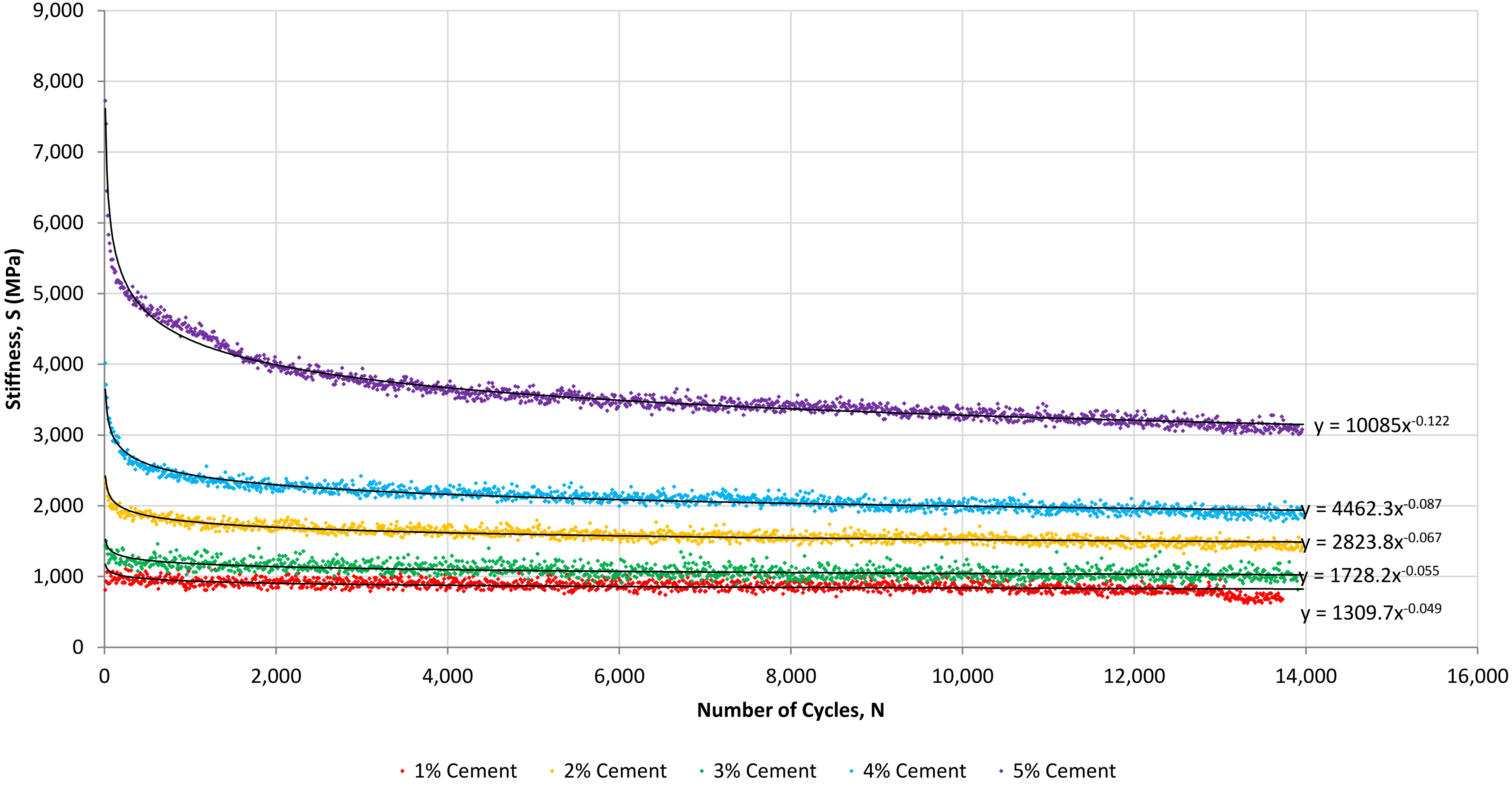


Figure 5.14: Flexural stiffness, S_f vs. cycle, N (S-N) curves for cement treated specimens under 400 με constant applied strain

75 με Applied Strain

The 75 με results presented in Figure 5.12 show that no appreciable fatigue behaviour was evident for the duration of the test for most of the cement content mixes. This is based on the observation that the stiffness of each cement content mix remained constant for the duration of the test. The flexural stiffness for 1%, 2%, 4% and 5% cement content mixes were 2092 MPa, 3506 MPa, 6341 MPa and 12077 MPa respectively.

An exception exists for the 3% cement content mix which showed a slight decline in stiffness over the duration of the test. This outcome agrees with the observations presented in the Elastic Strain Limit Test, as presented in Table 5.6, where the initiation of damage occurred at 55 με. The reduction in stiffness was characterised by an exponential drop in stiffness where the exponent of best fit curve was -0.03, translating to a Load Damage Exponent (LDE) of 30.3. This LDE is comparatively higher than the Australian accepted numerical model of 12 by Austroads (2008), which in turn provides a fatigue life of 1.32 x 10^9. The 3% cement content mix is ascertained to be susceptible to fatigue due to its low strength but brittle nature.

Despite having observed that 75 με is the elastic strain limit for 5% cement treatment, as presented in Table 5.6, the averaged specimen results did not show any distinct behaviour of an accumulation of damage. Nevertheless, some damage in several specimens was observed for 5% at 75 με, which suggests that because the tested strain is the borderline for the onset of fatigue behaviour, variability in specimen preparation affects the fatigue response of the specimens. The 1% cement material did not last for the full duration of the test, due to the fragility of material, a limitation further discussed in Chapter 7.

200 με Applied Strain

For the 200 με test results, as shown in Figure 5.13, traits of fatigue become more apparent for all mixes tested which include 2%, to 5% cement content mixes. The 1% cement specimen was too fragile to undertake the test and specimens were lost during testing, a point further discussed in Chapter 7. The fatigue behaviour of specimens under fatigue loading is exponential in nature and is characterised by an initial drop in modulus before entering into a mode characterised by continuous degradation of the material.

At 200 με, the 2% cement content mix was susceptible to fatigue damage, with an exponent of -0.04 or LDE of 23.8, as shown in Table 5.7. The rate of damage was again higher than that expected of Austroads, and corresponds to a fatigue life of $1.46x\ 10^{7}$.

In contrast to previous data, the initial stiffness of the 3% cement content was relatively high at the start of the test. This measurement is attributed to variability in specimen preparation. A portion of the data was also normalised due to erroneous reading of the laboratory equipment. The tested specimens provided an unacceptably high initial modulus of 8,129 MPa, and an LDE of 52.6 which constitutes a fatigue life of 6.83×10^{15}. The results suggest that errors occurred during the tests, and these can be linked to malfunctions in the test rig or human error during specimen preparation. Due to the time consuming nature of the tests, a retest was not carried out as part of the scope of this dissertation, a limitation discussed further in Chapter 7. Nevertheless, based on a pro-rata assessment of the behaviour of the material under 75 με and 400 με, the expected behaviour of the material would be an LDE within the range of 20 – 25, constituting an average fatigue life of 5.93×10^{6}.

The 4% cement content specimen showed almost similar characteristics to that of the 5% cement content mix design, displaying an initial modulus of 5856 MPa and undergoing a reduction in stiffness by an LDE of 33.3. This constitutes a fatigue life of 1.06×10^{10}.

Of the specimens tested at 200 με, crushed rock stabilised with 5% cement content by mass shows the most distinct trait of fatigue. By the end of 1,000,000 cycles, the specimens showed a significant reduction in initial stiffness from 7,300 MPa to an approximate 3800 i.e. 52% of the initial stiffness, which is just short of the accepted failure criterion for pavement materials of <50% of initial stiffness. The LDE of the specimens are calculated to be 20.4.

400 με Applied Strain

400 με is a relatively high applied strain used for base course and it was selected to exaggerate the fatigue phenomenon of the specimens and identify distinctive traits of materials. As shown in Figure 5.14, the 400 με test results show a distinct fatigue deterioration in cement treated crushed rock for all material mixes where two pronounced traits can be noted from the curves, i.e. an initial drop in modulus and a subsequent reduction in stiffness.

The 1% cement content mix showed very little initial reduction but suffered from continuous damage from the loading at an LDE of 20.6, giving the mix design a fatigue life of 1.55 x 10^6. At the other end of the spectrum, the 5% cement content mix showed an initially high stiffness of 10085 MPa, dropping to approximately 4000 MPa by 2000 cycles; a significantly large drop. The calculated fatigue life for the 5% cement content mix was 296 cycles, implying that the material had effectively failed to perform.

As for specimens with 2% cement content by mass, the initial stiffness was higher than that of 3% and the LDE calculated was 14.9 compared to the 18.2 calculated for 3% cement content. This corresponds to a fatigue life of 3.06 x 10^4 and 3.01 x 10^5 for 2% and 3% cement content mixes respectively. However the initial concavity of the graph is not as pronounced, as suggested mathematically by the LDE value.

Finally, the 4% cement content mix design shows a reasonable initial drop in stiffness of 4462 MPa to approximately 2400 by 2000 cycles, nearly 50% of its initial stiffness. The calculated LDE and fatigue life are 11.5 and 2896 cycles respectively.

Combined Analysis of Fatigue Results

The results of the tests are summarised in the table below.

Table 5.7: Summary of flexural dynamic test results

Applied Strain	Cement Content	1%	2%	3%	4%	5%
75 με	S-N	2092	3506	$5259N^{-0.033}$	6341	12077
	LDE	constant	constant	30.3	constant	Constant
	N_{50}	∞	∞	1.32×10^{9}	∞	∞
200 με	S-N	-	$3107N^{-0.042}$	$8129N^{-0.019}$	$5856N^{-0.03}$	$7347N^{-0.049}$
	LDE	-	23.8	52.6	33.3	20.4
	N_{50}	-	$1.46x10^{7}$	$6.83x10^{15}$	$1.06x10^{10}$	$1.38x10^{6}$
400 με	S-N	$10085N^{-012}$	$4462N^{-0.08}$	$2823N^{-0.06}$	$1728N^{-0.05}$	$1309N^{-0.04}$
	LDE	20.6	14.9	18.2	11.5	8.2
	N_{50}	$1.55x10^{6}$	$3.06x10^{4}$	$3.01x10^{5}$	2896	294

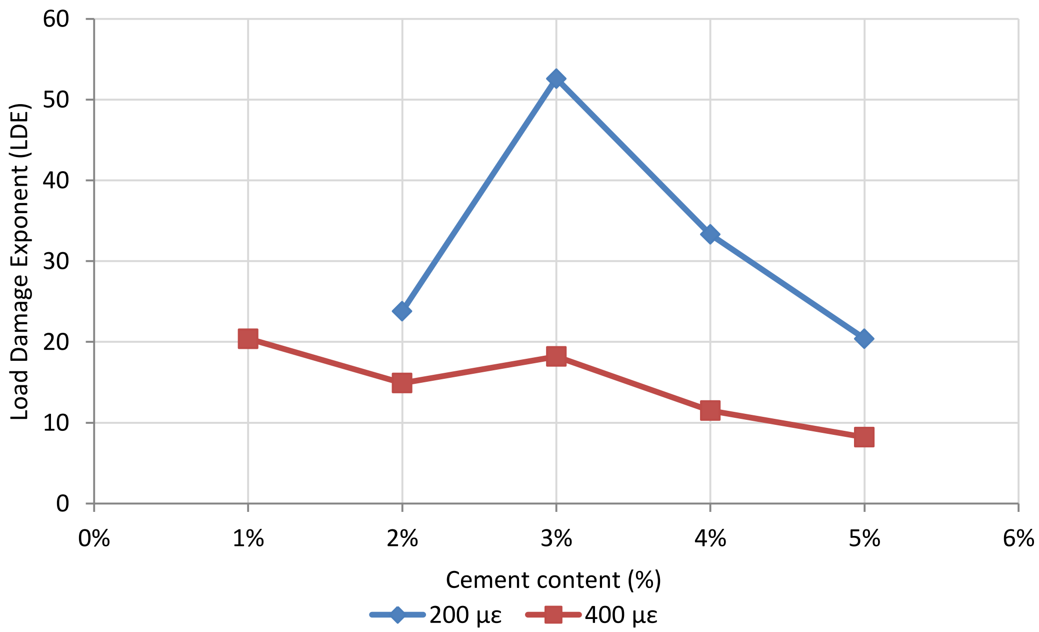


Figure 5.15: Load Damage Exponent vs. cement content (%)

Based on Figure 5.15 below and the data presented in Table 5.7, although the LDE generally decreased with an increase in cement content, a rise in LDE was evident for the tested strain ranges. This is potentially explained by the balancing point - when the specimens become sufficiently stiff to resist excessive strain. The curves show that the rise in LDE shifted from 4% to 3% with increasingly applied strain, showing that an increase in strain indicates

Furthermore, the failure plane of specimens, after undergoing dynamic testing, was typically located at 150 mm - 250 mm along the length of the specimen, as shown in Figure 5.16. The failure plane is determined by applying manual force to the specimen upon completion of fatigue loading. The location mentioned above corresponds to the constant bending moment area of the beam, as discussed in Section 3.4. This confirms that the failure mechanism is predominantly caused by flexing/bending and not by shear.

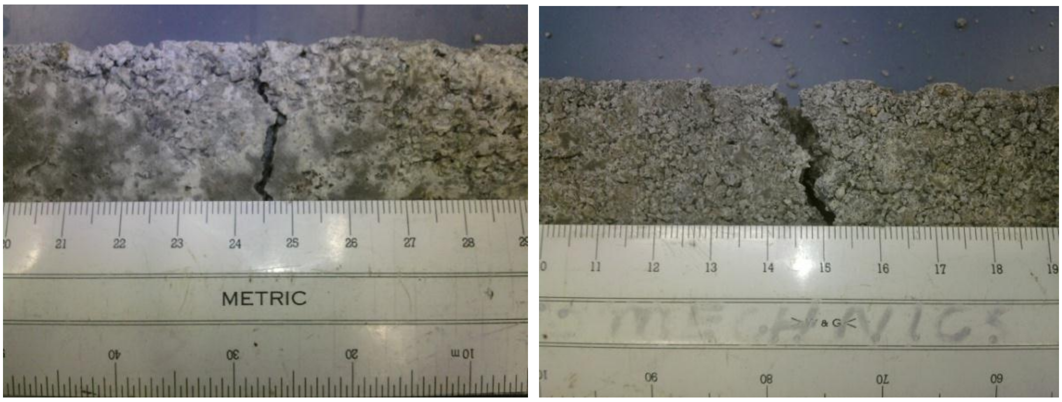


Figure 5.16: Location of failure plane along length of specimen

Figure 5.17 shows the failure surface of specimens after the completion of fatigue testing. The failure section was characterised by a coalescence of cracks through the cement matrix and there were undulations of aggregates therein. Most of the failure plane consisted of large extruding aggregate which suggests that this may be a weak link in cement stabilised materials. Large aggregates were found at the location where the highest concentration of interfacial transition zones existed, this thus became the weakest structural component in the material. Furthermore, the strain incompatibility between aggregate and cement matrix was most prominent in these areas, leading to further and severe developments in cracking.

The cement matrix did not seem to have disintegrated in other locations as no other traces of cracking were evident. This suggests that the redistribution of moment is minimal upon damage to the specimen and that damage typically concentrates in a concentric manner.

Figure 5.17: Failure section of flexural beam specimen after fatigue loading

5.4 Tube Suction Test (TST)

This section provides the experimental results and analysis of the Tube Suction Test. Figure 5.18 below shows the average dielectric value, DV, of triplicate samples vs. time, t, for untreated crushed rock and crushed rock treated with 1% through to 6% cement content.

As shown in the results, the typical relationship between DV and time is characterised by an initial sharp rise in DV followed by a plateau. However, this was only distinctly pronounced for specimens with a cement content of less than 3%.

For specimens with equal to or more than 4% cement content, a steady rise in DV was instead displayed and a plateau was not apparent for the duration of the test. This observed plateau was, in effect, an end to the increase of water infiltration, as the water column had reached the top of the specimen and achieved equilibrium.

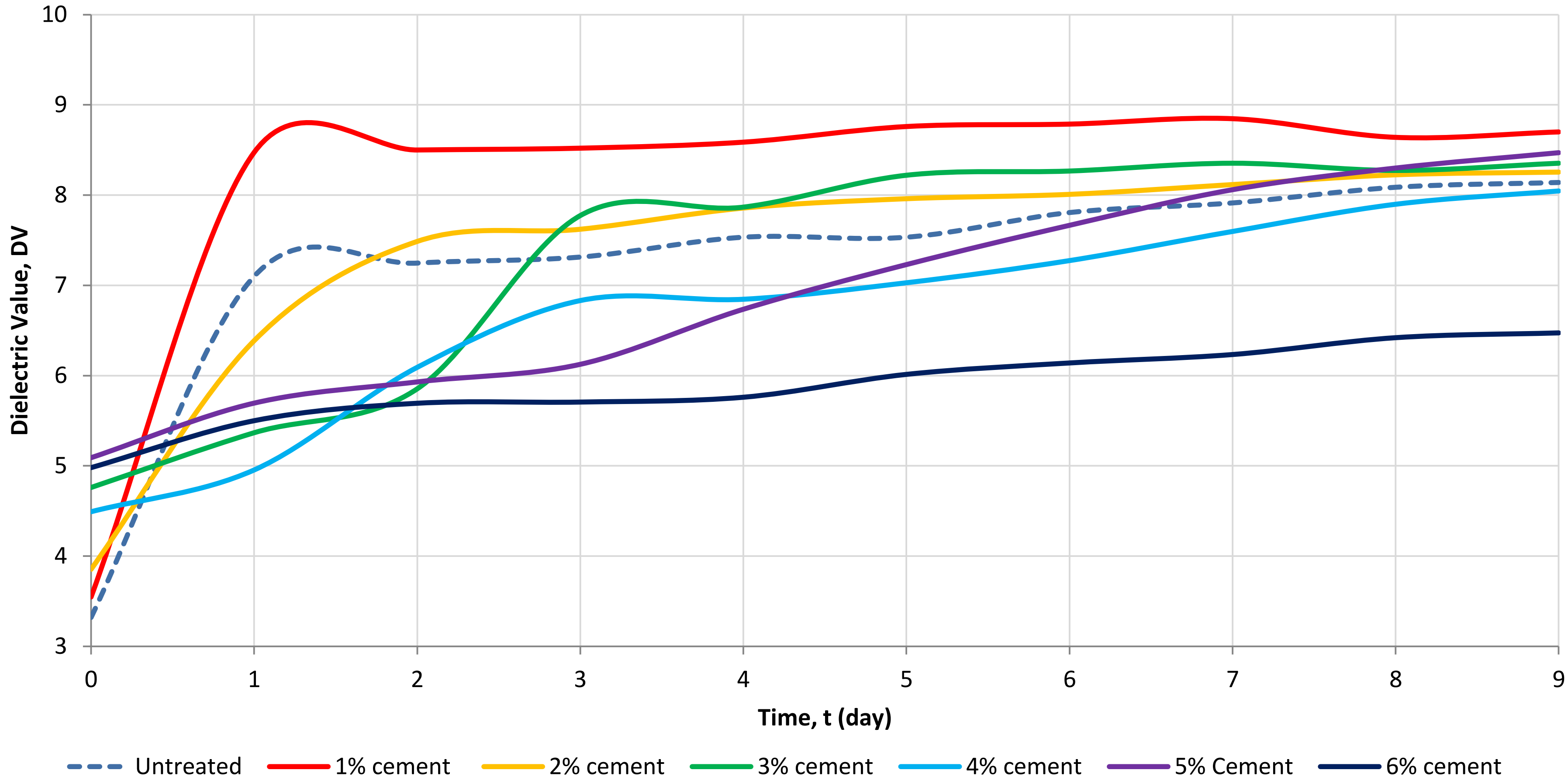


Figure 5.18: Tube Suction Test results - Dielectic Value, DV vs. time, t (day)

The DV_{max} measured for the duration of the tests, in the most part was independent of cement content and no distinct trend can be noted between cement content and DV_{max}. The DV_{max} at the end of the duration of the test, for specimens with 1% through to 5% cement content, was similar, and lay between 8 and 8.5, comparable to that of untreated crushed rock base course (shown in the dotted line). Nevertheless, specimens treated with 6% cement showed a marked reduction in DV_{max}, simply because they had not yet reached their saturation point.

The maximum dielectric value, DV_{max}, measured using the Tube Suction test, was meant to provide an indication of the material's durability. Based on this principle, it is implied that cement treatment of crushed rock base course does not improve the material's durability. Furthermore, as DV is also a measure of the water content of the specimens, this suggests that an increase in cement content does not reduce the amplitude of moisture fluctuations or the total volumetric content adsorbed by specimens.

In supporting the relationship between DV and water content, the DV readings were plotted against the water content of the specimens in

Figure 5.19. As can be seen, the DV measurement produced from the Tube Suction Test shows a statistically moderate relationship to water content, with an error of $\pm$0.5% and a least square regression, R^2 of 0.654. This means that through proper calibration, the methodology may be potentially used for non-destructive testing of in-situ pavements for quality control and material characteristics purposes. A concentration of data scatter is noted for DV of 5 to 6 which corresponds to 2.3% to 2.6% of water content, which is typically an initial boundary condition of specimens.

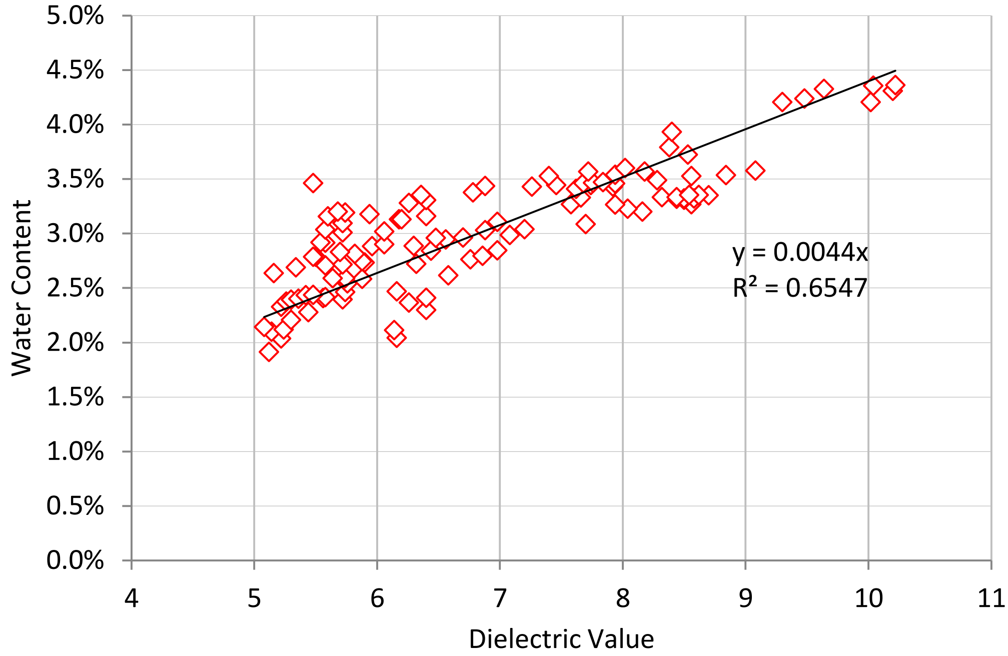


Figure 5.19: Dielectric Value. DV vs. water content (%)

An evident linear relationship exists between infiltration and the square root of time. The sorptivity of specimens with 3% to 6% cement content was derived based on the gradient of each linear measurement, as shown in the legend in Figure 5.20 below. The y-axis intersect of the linear equations, which average approximately 4.53 cm, an empirical constant noticeable to cement treated crushed rock basecourse.

As discussed earlier, the high least square regression value, R^2, of the linear trend lines indicates that the materials are highly homogeneous. The 1% and 2% cement specimens however did not provide any direct results, as the equilibrium point had been achieved prior to the first measurement at day 1, but by assuming an initial boundary condition of 4.53cm (average of other specimens), the sorptivity of the two specimens can be determined.

The results show that an increase in cement content decreases sorptivity. This means that the increase in cement content improves the resistance of the material to carbonation. A marked decrease in sorptivity exists for treatments of 3% or more of cement content.

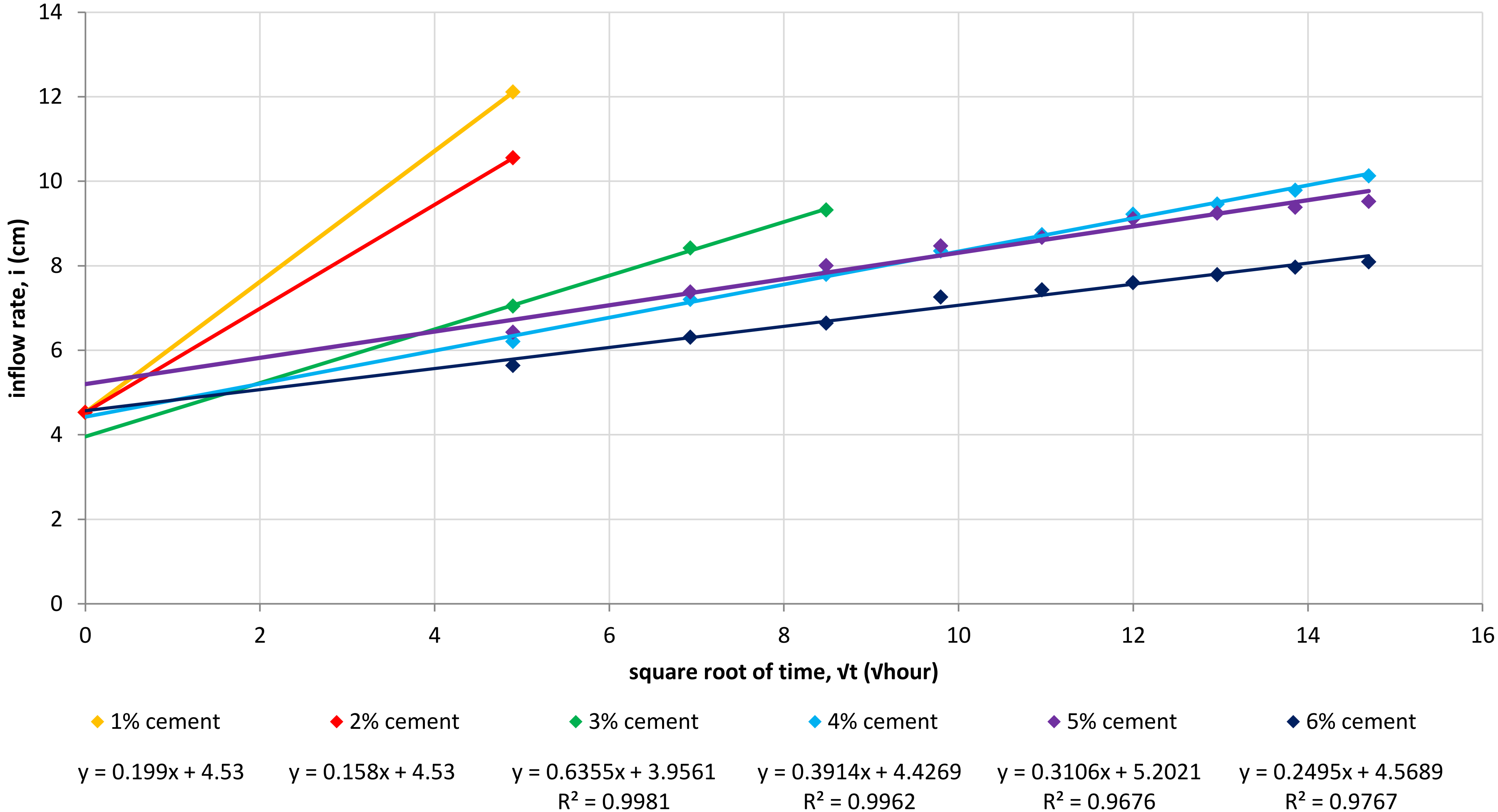


Figure 5.20: inflow rate, i (mm) vs. square root of time, √t (√hour)

The Unconfined Compressive Strength of the specimens was measured at the end of each test to assess the residual strength of the specimens. The UCS values of all specimens at the end of each test are shown in *Figure 13* below. They were compared against the UCS of a controlled specimen that was maintained in a dry state throughout the Tube Suction Test, and UCS values were measured by the author, based on AS5101.4 (Australian Standards 2008).

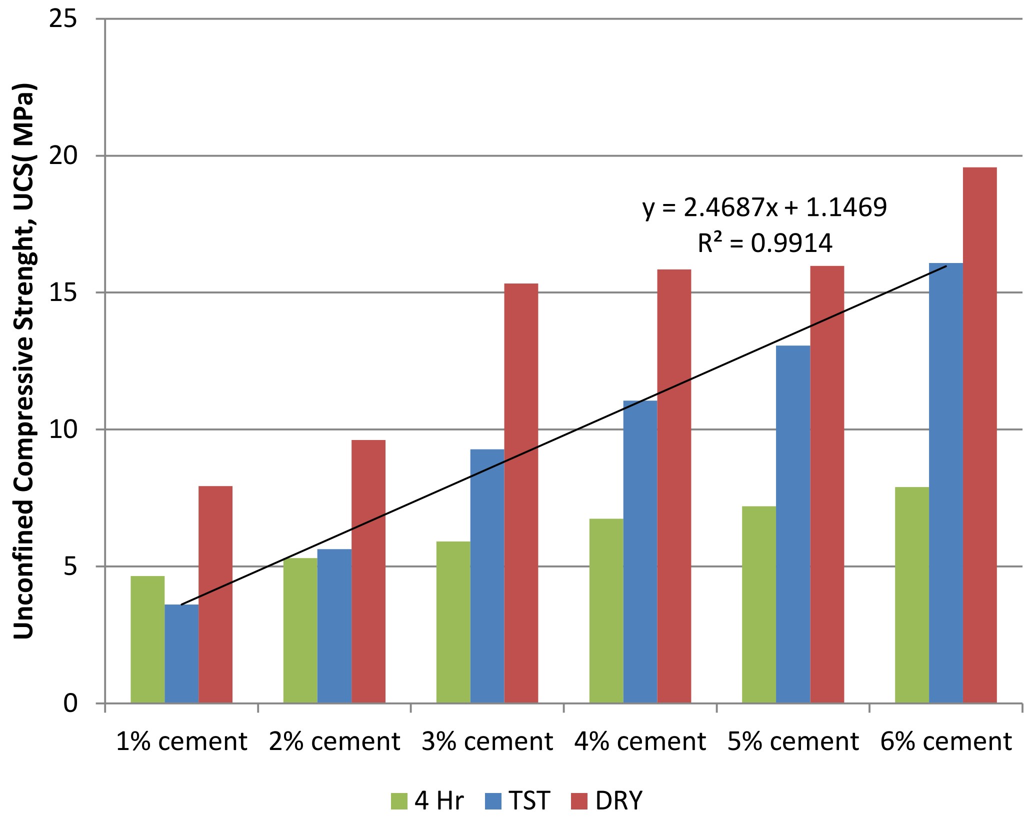


Figure 5.21: UCS vs. cement content under various soaked conditions

Figure 5.21 above shows that a distinct linear relationship exists between cement content and the average UCS values of specimens at the end of the Tube Suction Test. After 10 days soaking, a more distinct margin between the UCS values of fully saturated specimens and the end of TST (10-day soak) was apparent. The results complement the observations made earlier in that the 1% and 2% specimens reached saturation, and the margin between the 4 hour soak and TST was similar. 5% and 6% cement content specimens were the best performing specimens in terms of residual strength while 3% and 4% specimens showed moderate performances.

It is recognised that the UCS test results show that the specimens, when treated with as little as 1% cement, behave as "bound" materials as opposed to "modified" materials based on the classification discussed earlier in this paper. However, in order to achieve sufficient resistance against carbonation, it was decided to further investigate by examining the 3% (or more) treatment.

The summary of Tube Suction Test is shown in Table 5.8 below.

Table 5.8: Tube Suction Test results summary

Cement Content	1%	2%	3%	4%	5%	6%
DV_{max}	8.1	8.8	8.3	8.4	8.0	8.5
i (mm/$t^{0.5}$)	0.199	0.158	0.082	0.0505	0.0401	0.0322
R^2	-	-	0.9997	0.9982	0.9847	0.989
$UCS_{10\text{-day soak}}$ (MPa)	3.61	5.63	9.28	11.06	13.07	16.08
UCS_{dry} (MPa)	7.94	9.62	15.33	15.84	15.98	19.57

5.5 Nitrogen Adsorption

This section presents the results of the nitrogen adsorption test, undertaken as part of this dissertation, which include pore size distribution curves for cement treated base course with 1% to 5% cement content. Figure 5.22 and Figure 5.23 below show the pore size distribution of the cement mixes for 1 day's and 7 days curing respectively.

As seen from Figure 5.22, the pore size distribution curves for cement treated crushed rock materials after 1 day of curing are characterised by a concentration of pores, with the diameter of mesopores being 2.5 nm – 30 nm, and a gradual decline is evident in the pore volume of macropores, ranging from pores with a diameter of 30 nm to 100 nm.

This is followed by a constant pore volume of pore diameters of 100 nm – 215 nm, before a sharp decline in pore volume for diameters of more than 215 nm. The total volume of mesopores increased with cement content, with 1% cement exhibiting very low volumes of cement content, increasing steadily to 4% cement content before a significant increase to 5% cement content. On the other hand, the macropores increased steadily from 1% to 5%, with little difference between 1% and 2%. The pore volumes of the mesopores are summarised in Table 5.9 below.

Table 5.9: Pore volume (cm^3/g) and cement content

Cement Content	1%	2%	3%	4%	5%
$V_{meso\text{-}1day}$ (cm^3/g)	0.0003	0.0026	0.0044	0.0024	0.0101
$V_{meso\text{-}7day}$ (cm^3/g)	0.0007	0.0043	0.0022	0.0005	0.0028
%	+150	+100	-50	-79	-73
$V_{macro\text{-}1day}$ (cm^3/g)	0.0086	0.0113	0.0200	0.0343	0.0469
$V_{macro\text{-}7day}$ (cm^3/g)	0.0281	0.1002	0.0486	0.0483	0.0652

In distinct contrast, Figure 5.23 shows that at 7 days curing, a concentration of macropores is more prominent in the pore size distribution curve. The pore size distribution is characterised by a constant rise in pore volume, with an increase in pore diameter, along with a distinct peak in pores with a diameter of 215nm for all cement content. The total pore volume did not seem to show any direct relationship with the cement content of 2%, which showed the highest pore volume. This observation can be potentially explained by the inhomogeneity of samples used for the test.

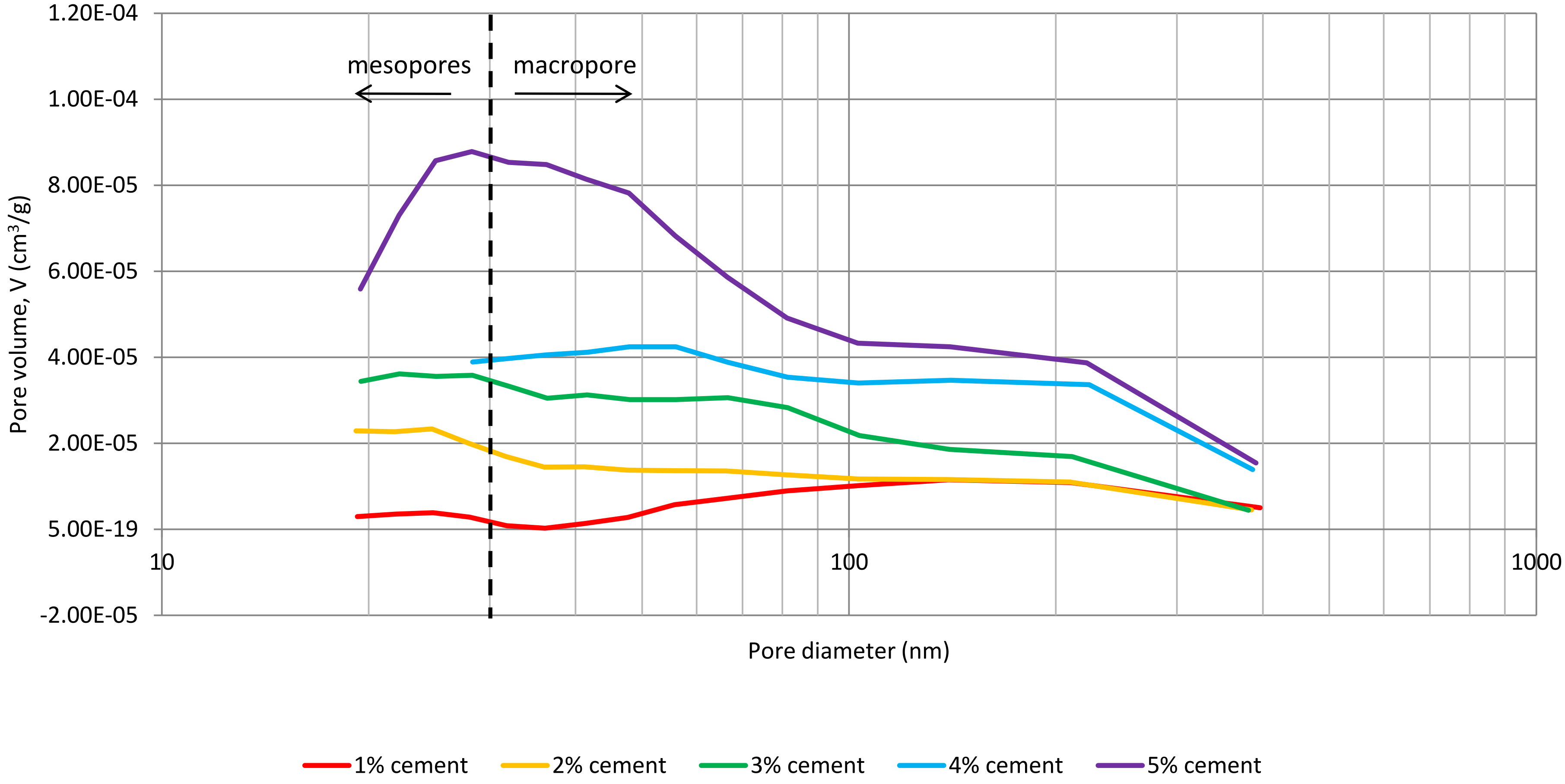


Figure 5.22: Pore size distribution of cement treated basecourse after 1 day curing

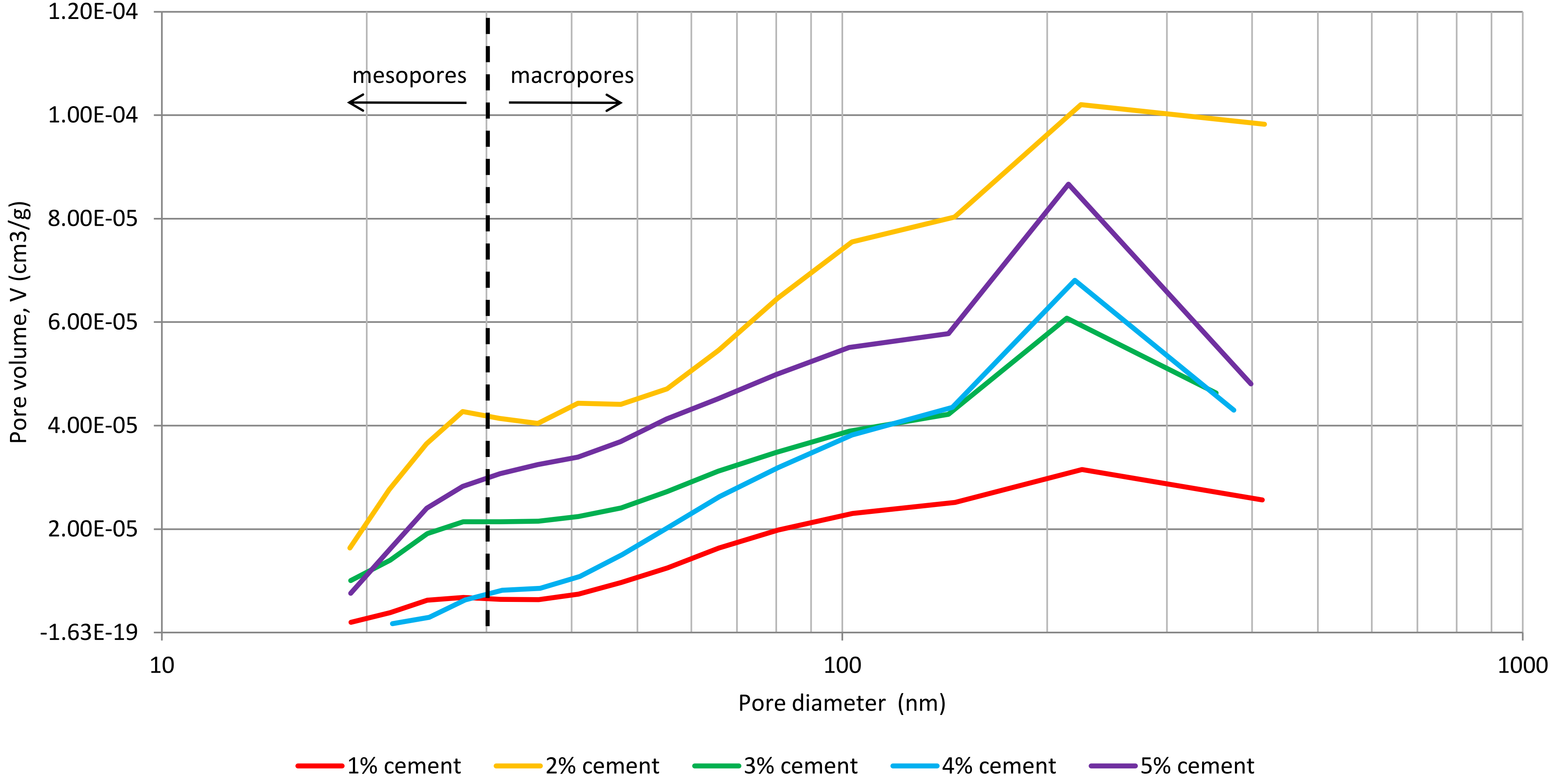


Figure 5.23: Pore size distribution of cement treated basecourse after 7 days curing

In comparing the results between day 1 and day 7, the volume of mesopores has decreased significantly for 3%, 4% and 5% cement content mix, while it has increased for 1% and 2% cement. This suggests that the 3%, 4% and 5% cement content materials undergo shrinkage with the calculated volume to be 50%, 21% and 27% of that ascertained at day 1. The 1% and 2% cement content have instead doubled in pore volume, suggesting that the specimens swelled.

5.6 Linear Shrinkage Test

The linear shrinkage sample set for cement treated base course fines with 1% to 5% cement content is shown in Figure 5.24 below. A red line is drawn to identify the original length of the specimens.

The measured length of the specimens is summarised in Table 5.10 below. As noted in the photo, the 1% and 2% specimens have swelled, the 3% has remained unchanged, and the 4% and 5% cement content specimens have shrunk.

Table 5.10: Linear shrinkage test results

Cement Content	1%	2%	3%	4%	5%
Strain	+1.84 swell	+1.56 swell	+0.04 swell	-0.08 shrink	-0.16 shrink

The measured strain however, is relatively low and this suggests that shrinkage damage to specimens with 3% or more cement content may be minimal. In-situ, actual performance may vary.

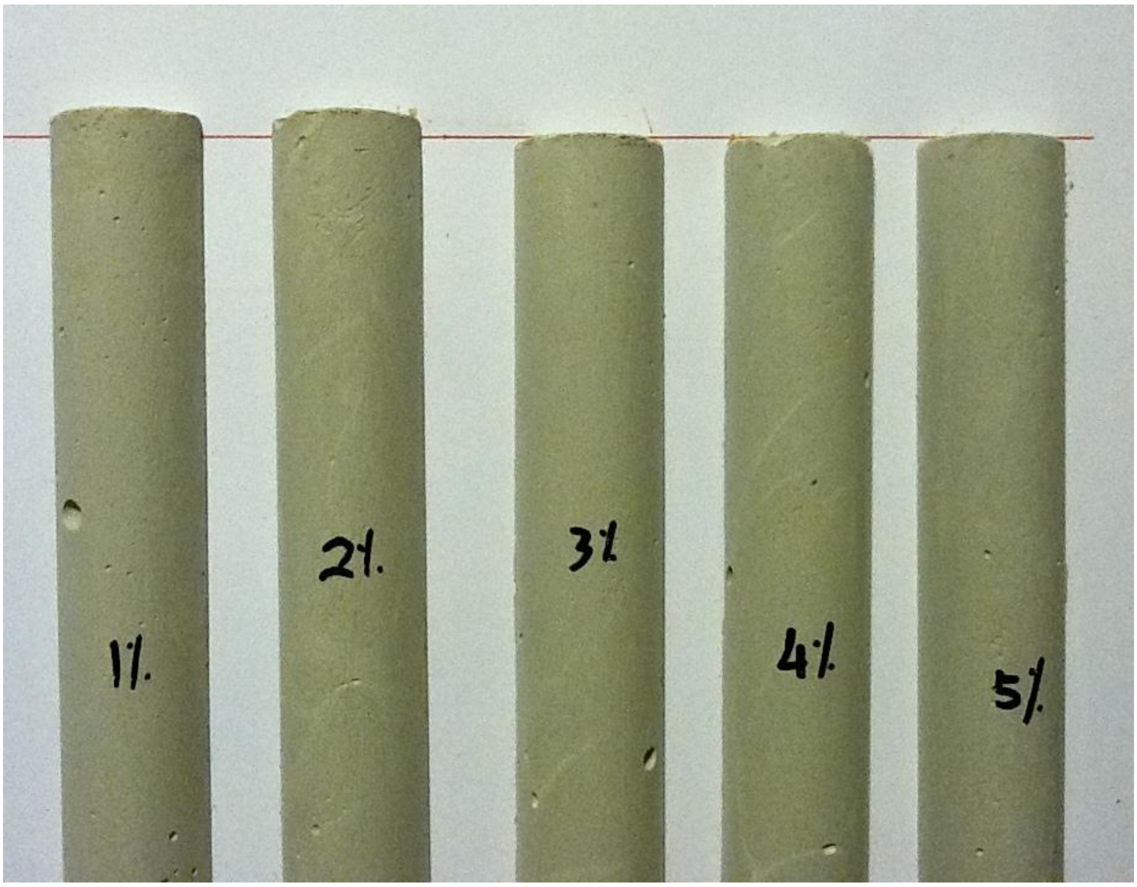


Figure 5.24: Linear shrinkage specimens - red line showing initial condition

The results correlates to the observed pore size distribution changes as covered in the previous section.

5.7 Wheel Tracking Test

This section presents the results from the Wheel Tracking Test which is aimed to assess the erodibility of cement treated crushed rock. A typical profile of the eroded surface of a 6% cement content slab, at the completion of 5000 passes, is shown in Figure 5.25 below.

As shown from the profile of the slab surface in Figure 3 above, the most severe erosion is evident in the centre of the slab. In order to ascertain a more representative erodibility index of the material, the central 50mm of the slab was measured. The lesser erosion evident towards the edge of the slabs was in all likelihood, caused by the deceleration of the wheel tracker. Surface inspections after each test were undertaken to ensure that the readings were not distorted by the presence of any deposits of large aggregates on the surface.

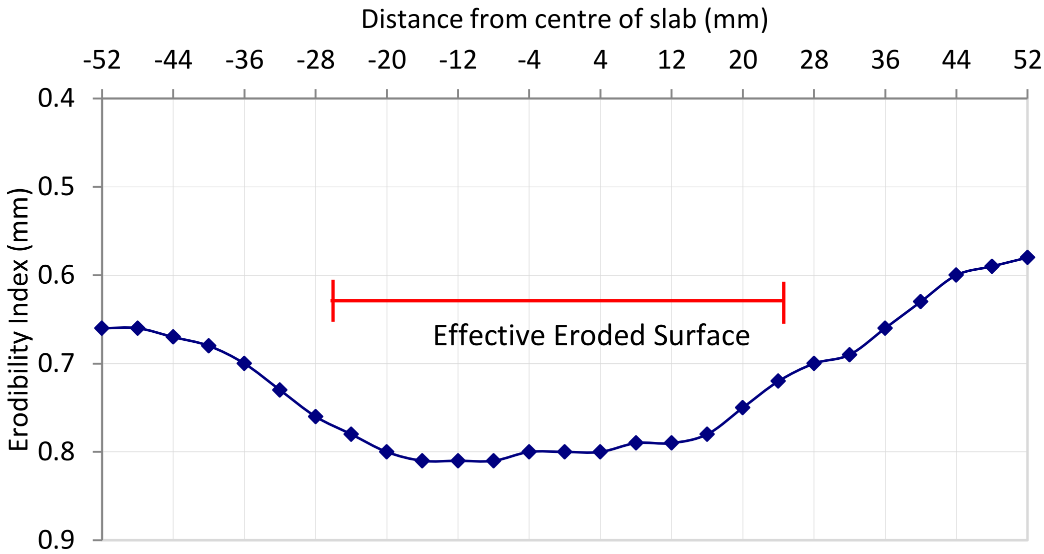


Figure 5.25: Typical Profile of cement treated crushed rock slab after 5000 Runs

Taking the average erodibility against the number of runs for all specimens, Figure 5.26 shows the development of erosion. From the data collected (see Figure 5.26), the Erodibility Index, ε can be calculated, as summarised in Table 5.11.

Table 5.11: Erodibility for various cement content

Cement Content	1%	2%	3%	4%	5%	6%
ε_{max}	0.41*	0.48	0.54*	0.61	0.61*	0.75
R^2	-	0.58	-	0.531	-	0.435

The Erodibility Index of the specimens, taken as the maximum eroded depth after 5000 passes, obeys the power law which is characterised by a sharp initial increase before achieving a resilient state.

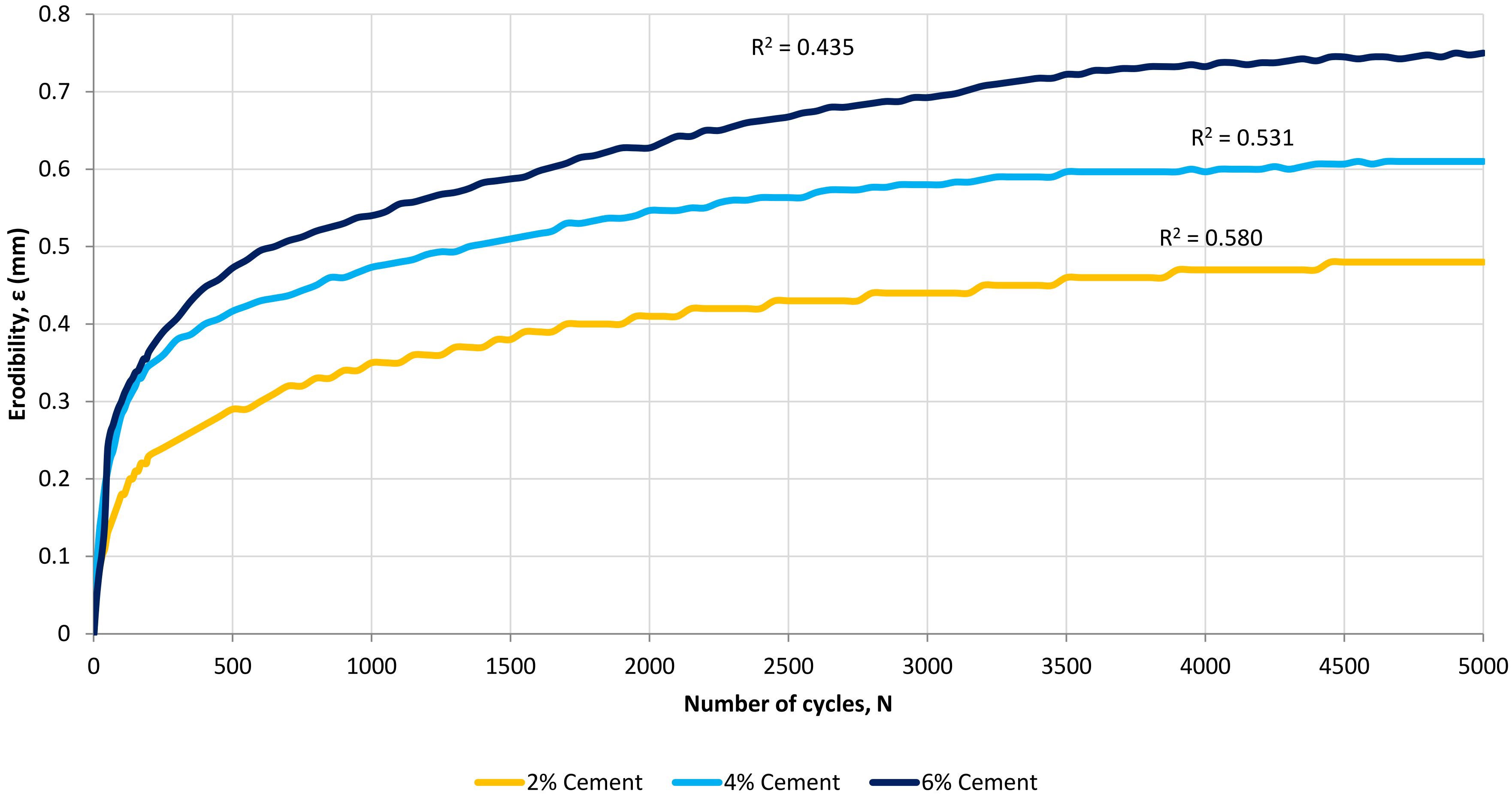


Figure 5.26: Erodibility, ε (mm) vs. number of cycles, N

The Erodibility Index also shows a positive linear relationship with cement content. The Erodibility Index increases by approximately 0.1mm with an increase of 2% cement content. This suggests that the increase in cement content would result in faster surface deterioration of unsealed roads. An explanation of this observation can be traced to the change in water to cement ratio. The water to cement ratio, from 2% cement content to 6% cement content, decreases from 3 to 1.17, which potentially means that the cement paste develops a higher propensity to migrate to the base of the slab during curing periods. As this occurs, less cement paste is exposed on the surface. This is supported by the visual observations of the specimens prepared during the tests. The increased cement content showed a more pronounced concentration of cement paste on the surface of the slab. Figure 5.27 below shows a typical finished surfaced of a 6% cement content slab.

Figure 5.27: Typical surface depression after 5000 runs

One concern regarding the analysis undertaken, is the relatively low least square regression achieved from the analysis, i.e. $R^2 \approx 0.5$. This clearly indicates that there is some variability in the results.

This is likely to be caused by the limitations involved in using the Cooper Wheel Tracking Test. The machine did not provide control measures to maintain the moisture of the specimens throughout the test, unlike the South African Wheel

Tracking Test used by De Beer (1989). In addition, the temperature control system blew directly onto the specimens which expedited the drying process. As a result, throughout the test, the specimens underwent significant fluctuations in moisture content, particularly on the surface.

Furthermore, due to delays between testing and the handling of specimens, the soaking period was varied ± 2 hours which resulted in a trend in the Erodibility Index. Preliminary observations showed that moisture conditions at testing significantly impacted upon the Erodibility Index. These observations however, are still premature and will not be reported in this text.

5.8 Analysis Summary

In summary, the preceding material properties and their varying behaviours when exposed to changes in cement content are summarised in Table 5.12 below.

Table 5.12: Summary of results and analyses

Material Property	Behaviour
Unconfined Compressive Strength, UCS	• Increases linearly with cement content within tested range (low cement content)
Compressive Modulus, E_{UCS}	• Increases (polynomially) with cement content plateaus at 4%
Indirect Tensile Strength, ITS	• Increases linearly with cement content within tested range (low cement content)
Cohesion, c	• Increases linearly with cement content within tested range (low cement content)
Internal Angle of Friction, Ø	• Decreases linearly with cement content within tested range (low cement content)
Flexural Bending Strength, FBS	• Increases linearly with cement content
Breaking strain, $\mu\varepsilon_{f\text{-}breaking}$	• Decreases exponentially with cement content plateaus at 2% cement content
Flexural Modulus, $E_{flex\text{-}static}$	• Increases linearly with cement content
Elastic Strain Coefficient, p	• Generally 50%
Elastic Strain Limit	• Materials with more than 3% cement develop fatigue ○ 3% at 85 με ○ 4% at 55 με ○ 5% at 75 με

Material Property	Behaviour
Load Damage Exponent, LDE	• Generally decreases with cement content • Observed hike due to increased resistance to deformation (stiffness)
S-N Curves	• Reduces exponentially when applied strain is more than elastic strain
Fatigue Life, N	• Reduces with increased strain application • Reduces with increased cement content
Maximum Dielectric Value, DV_{max}	• Averages at 8.2 for all cement content
Inflow Rate, i	• Decreases with cement content
Unconfined Compressive Strength post Tube Suction Test, UCS_{TST}	• Margin between UCS_{TST} and drying reduces with increased cement content
Volume of Mesopores, V_{meso}	• Mesopores increase with cement content
Volume of Macropores, V_{macro}	• 1%, 2% swelling
Linear Shrinkage, LS	• 3%> shrinkage
Erodibility Index, ε	• Decreases with cement content

Given the above understanding of cement treated crushed rock, the following chapter presents discussions of the analysis against the objectives of the dissertation presented in Section 1.5.

6 Discussion

Building on the experimental data collected from the laboratory work undertaken as part of this dissertation, this chapter presents critical discussion in reflection of the motivation of the research as discussed in Chapter 1. The topics discussed in this topic include:

1) A review of the behaviour of cement treated crushed rock

2) Numerical modelling of pavements

3) Reclassifying cement treated basecourse

6.1 A Review of the Laboratory Program and Material Behaviour of Cement Treated Crushed Rock Basecourse

This section reviews the laboratory programs carried out in this dissertation. It summarises the material characteristics of cement treated crushed rock basecourse, drawing on the data and analyses of the results presented earlier. It addresses objectives 1, 3, 4 and 5 of this dissertation.

6.1.1 Ultimate Strength and Serviceability

As discussed in Chapter 3, the strength parameter considers the serviceability of a pavement. This is defined as its propensity to undergo deformation under traffic loading as opposed to its ultimate strength. The experiments carried out as part of the program of tests for this dissertation on material strength includes the compressive (UCS see Section 5.2.1), tensile (ITS see Section 5.2.2) and flexural (FBT, see Section 5.3.1) tests. The tests provide the ultimate strength of the materials under different loading conditions and their corresponding stress strain relationship (modulus).

6.1.1.1 Strength Behaviour

The relationship between cement content and the ultimate strength is linear within tested range (low cement content) in all tests undertaken. When cement is added to crushed rock basecourse, the physical interlock between aggregates decreases marginally because cement acquires the space in between aggregates, acting as a form of lubricant. However, this effect is minimal as the cement content is only a minor fraction of the overall volume of the composite. Instead, this cement content provides an appreciable gain in cohesive strength that binds the aggregates together. With an increasing volumetric content, the cement content becomes a denser matrix and develops a stronger bond. Furthermore, the water cement ratio is decreased with increasing cement content as discussed in Section 5.2. The hydration of cement constituents is more effective and forms a denser matrix that is stronger and contains less voids. As noted in the Mohr Circle envelope calculations in Section 5.2.3, the shear strength component based on material interlock remains fairly constant and the increase in shear strength is primarily realised by the increase in cohesion.

This increase in ultimate strength gained from increasing cement content indicates that the pavements can withstand an increasingly higher traffic load prior to an abrupt shearing or puncturing failure. However, pavements rarely reach its ultimate strength during load as the pavement layer is supported by a subbase or subgrade, allowing it to perform as a continuously restrained structure. Instead, for pavements constructed with cement treated crushed rock basecourse over stiff subbase or subgrade, the serviceability of the pavement material can be inferred. This inference, supported by the compressive modulus assessed from the tests undertaken, which shows the materials capacity to resist deformation. Based on the results and analyses undertaken or this dissertation, it is noted that the compressive modulus plateaus after 4% of cement treatment. The above strength test results suggest that the serviceability of the material has a limit, despite an increase in ultimate strength. For cement treated crushed rock, this was found to be at 4% cement content mix. This can be explained by the strain incompatibility between the aggregate and the cement matrix.

As the volume of cement content increases, the compressibility of the material changes. Its dependency on material interlocks decreases and shifts towards a dependency on the cohesive forces developed by the cement matrix. The modulus assessed in this dissertation does not provide any direct implication to the in service performance of pavements. This is because the tests are undertaken in an unconfined state, a condition contrary to that of a pavements in service condition.

When overlaid with a weaker subbase or subgrade, the material has a propensity to flex. It therefore depends on the tensile and flexural property of the material, which is directly relative to the volumetric content of cement in the material. The cement acts to hold the material together and its relationship with cement content is defined by a linear relationship. This is explained by the increase of the density of cement constituents and the lowered water cement ratio, two conditions that promotes stronger bonds. In other words, when overlaying a weaker subbase or subgrade, the cement treated basecourse has a lower service life as the flexural modulus is much lower than that of its compressive modulus.

6.1.1.2 Relevance of Ultimate Strength Measurements

Bearing in mind that the UCS pertinent to this dissertation is in gross excess of the expected values, it is implicated that the crushed rock treated with as little as 1% cement content behaves as a bound material when referencing to Austroads accepted guidelines. Nevertheless, as repeatedly highlighted in the analyses, a linear relationship exists between UCS and cement content. This makes the measurement an important and practical tool for industry to benchmark expected behaviour of material. This is also very useful to ensure material compliance and be an integral part of quality control management during construction works.

In other words, there is a need to calibrate UCS results to correlate against fatigue performance and/or moisture susceptibility for each specific mix design depending on the material as typically seen for concrete construction works. A technical specification should outline the various performance attributes required of the material and its corresponding UCS value rather than specifying UCS as a technical requirement in itself. Subsequently, this dissertation asserts that the UCS is not a universal classification measurement as presented by current Austroads guidelines. This is further discussed in Section 6.2.1.

With regards to ITS and FBT, the existence of a linear relationship for these specimens provides an opportunity using these strength results for a similar purpose as discussed above. The ITS and FBT results further provide the benefit of assessing the modulus of the material should they be fitted with strain gauges. This assists in the design of the roads based on current design methodologies. It is recognised however that testing equipment for FBT is rarer and the uniformity of specimens across the plane of failure for a FBT is less consistent.

6.1.1.3 Numerical Relationships

Based on the data analyses, the relationship between UCS, ITS and FBT vs. cement content for cement content ranges typical to road stabilisation in Western Australia (cement content of 1% to 5%) is shown below:

$$UCS_{28\text{-}day} = 0.65 \text{ x cement content (\%)} + 4.0 \tag{6.1}$$

$$ITS_{28\text{-}day} = 0.218 \text{ x cement content (\%)} \tag{6.2}$$

$$FBT_{28\text{-}day} = 0.432 \text{ x cement content (\%)} \tag{6.3}$$

Given that the strength parameter is a material benchmarking tool, the critical behaviours for cement treated crushed rock therefore relies instead on its fatigue behaviour, shrinkage propensity, durability and erodibility. This is discussed in the following subsections.

6.1.2 Fatigue

Flexural strains develop when a weak subgrade or subbase is laid over cement treated crush rock basecourse. As discussed in the preceding subsection, the support rendered by the weak subgrade/subbase is insufficient and causes the pavement layer to undergo flexing. The magnitude of flexural strain is further dependent on the stiffness of the basecourse layer itself; stiffer layers imply a lower propensity to deflect.

The fatigue parameter for cement treated basecourse considers the resistance of loss in stiffness of a pavement against repeated traffic loads through a transient condition, failing when the stiffness reduces to 50% of its original value. Damage due to fatigue occurs when repeated strain is applied onto the cement treated basecourse. This applied strain transfers to the material structure and produces permanent damage manifesting as microcracks that form discontinuities within the cement matrix. With every applied cycle, damage gradually accumulates within the material, reducing the effectiveness of load transfer and thus the apparent stiffness of the composite material. However, as implied in the literature review and analyses, the phenomenon occurs only when the applied strain is in excess of a minimum damaging strain. The concept of minimum damaging strain and initiation of fatigue is discussed in the subsequent subsection.

6.1.2.1 Initiation of Fatigue – Minimum Damaging Strain

When a cement treated basecourse is stressed, the strain experienced by the specimen will cause fatigue. However, this only occurs when sufficient strain energy is supplied. The minimum strain required to cause fatigue is known as the minimum damaging strain. The minimum damaging strain has been assessed by this dissertation by utilising the Elastic Limit Test.

As presented in Section 5.3.2, the Elastic Limit Test involves the application of increasing strain loads in a staged manner. In each stage, the stiffness of the material remains generally constant when the applied strain is less than the damaging strain. When the applied strain is more than the damaging strain, the stiffness reduces within the staged load. When the data is combined and presented in a continuously staged sequence, it is noted that a distinct relationship in the form of a power function exists between the applied strain and its corresponding stiffness. The minimum damaging strain can be detected when stiffness reduces within the applied strain stage and deviates from the power relationship. This minimum damaging strain is a result of permanent damage to the material and is effectively the first stage of fatigue behaviour as presented in Section 3.4. This feature is uniquely characterised by a high rate of decline during the initial stage of the stiffness vs. number of cycles (S-N) curves, and is a trait to identify a material's propensity to undergo fatigue.

The damaging strain increases with cement content and corresponds with the cease of stress-strain linearity or the elastic limit. The fraction of minimum damaging strain to the ultimate strain is denoted as k_{ε}. The fraction of minimum damaging strain, k_{ε}, as measured shown in Section 5.3.1, is lower for materials mixed with 3% cement content and more. The values are all less than 10%, less than the proposed

k_ε by Department of Transport (Deparment of Transport 1986) of 25%. However, it is important to note that the breaking strain range is within the typical ranges for damage to occur as presented by Austroads (Austroads 2010), where the targeted strain is approximately 80 με – 100 με.

The low k_ε is believed to be caused by the relatively high breaking strain of the specimen as measured in the FBT tests. The measurements from the FBT tests showed that the breaking strain is between 1000 – 1500 με for cement treated crushed rocks with more than 3% cement content, i.e. mixes that were determined to exhibit a damaging strain within the tested strain range. This range is considered relatively higher than typical values, and is believed to be an attribute of cement treated crushed rocks used in Western Australia. Furthermore, another potential cause of the high breaking strain is the testing regime. It is recognised that the normalised flexural stress and flexural strain behaviour as presented in Section 5.3.1 does not truly show the non-linearity of the stress strain curve, and does not reflect the calculated *p*-value of the test. The raw test data obtained in the laboratory provided some problematic results due to the non-uniformity of the loaded surfaces which showed a few unloaded movement in the stress strain curve. The limitation of the results is further discussed in Section 7.2.

Nonetheless, the existence of a minimum damaging strain as low as 3% of the breaking strain for cement treated crushed rock materials with more than 3% cement content means that the fatigue is almost certain to occur when these materials are used. With cement treated crushed rocks less than 3%, the propensity to undergo fatigue is limited – a point further discussed in Section 6.1.2.1.

The strong correlation between fatigue performance and strain suggests that the tensile stresses required to propagate microcracks are generated due to the strain incompatibility between the aggregate and cement matrix. As the specimens undergo deflection, different strain rates are experienced by the aggregate and cement matrix; the aggregates being less susceptible to deformation. This results in a restrained condition that develops tensile stresses within the cement matrix. By considering the concept of fracture mechanics, sufficient stresses ahead of the tip of the microcrack is required in order to propagate microcracks. In other words, when sufficient strain energy is supplied due to the strain incompatibility, fatigue damage of the composite occurs. Similarly, where the applied strain does not exceed the minimum damaging strain, the fatigue test results have shown the material remains in perpetuity and damage does not accumulate within the material.

From the discussion above, it can be concluded that the fatigue response and rate of damage is a function of strain. The fatigue relationship for cement treated basecourse is thus discussed in the subsequent section.

6.1.2.2 Fatigue Behaviour and the Load Damage Exponent

Upon exceeding the minimum damaging strain, cement treated crushed rock basecourse experiences fatigue damage. The behaviour of this fatigue damage is defined by the stiffness vs. number of cycles (S-N) curve and is in the form of a power function. This corresponds to the first two stages of the fatigue as covered in Section 3.4. The S-N curve is characterised by the following equation:

$$S = S_0 N^b \tag{6.4}$$

Rearranging and considering the number of cycles at failure yields,

$$N_f = \left(\frac{S_f}{S_0}\right)^{1/b} \tag{6.5}$$

where S_f = stiffness at failure (typically 50% of initial stiffness)

N_f = number of cycles to failure

S_o = the initial stiffness of the material

$1/b$ = load damage exponent (LDE)

As noted in Section 3.4, the service limit of a pavement or the failure criterion occurs when its stiffness reaches 50% of initial stiffness, i.e. S_f/S_o = 0.5. Thus, the equation can be simplified to $N_f = 0.5^{LDE}$ for new pavements. Furthermore, given the measured insitu stiffness of a pavement, the model allows the remaining service life of a pavement to be calculated at *n* number of years by taking

$$N_n = N_f - N_x \qquad (6.6)$$

where N_r = remaining life

N_f = number of cycles to failure

N_n = number of cycles to date

The unresolved subject in the above model is thus the definition of the LDE. Based on the experimental data in Section 5.3 and the literature review in Section 3.4, the relationship between fatigue and strain is inseparable and from the results presented in Figure 5.15, a clear drop in LDE is seen between the application of 200 με and 400 με, which give rise to the dependency of LDE with strain.

Also, revisiting the data and analyses presented in Section 5.3, in particular the minimum damaging strain vs. cement content relationship, breaking strain vs. cement content relationship (Figure 5.10) and Load Damage Exponent vs. cement content relationship (Figure 5.15), it can be seen that by increasing cement content these three parameters generally decreases. In other words, the material behaviour defines its fatigue response and the gain in stiffness of the specimen results in a higher propensity to suffer fatigue. Moreover, all three results show a deviation from their defined inverse relationship, e.g.

i. the LDE rose significantly in the LDE vs. cement content relationship in Figure 5.15 for 4% cement content

ii. the breaking strain vs. cement content relationship rose significantly for 4% cement content

iii. the minimum damaging strain vs. cement content dropped significantly for 4% cement content

Although the above may be attributed to experiment variability, it also hints that there is an appreciable interrelationship between the three parameters, strain and the material characteristics. So far this dissertation has only shown a relationship between cement content and the above parameters and not their relationship against LDE. Figure 6.1, Figure 6.2, and Figure 6.3 below shows the parameters in isolation to LDE.

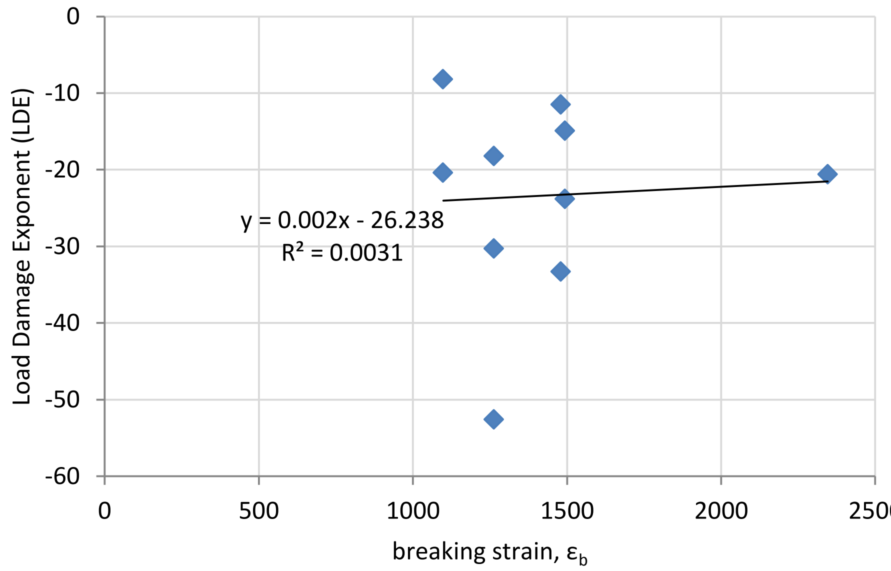


Figure 6.1: Load Damage Exponent LDE vs breaking strain, ε_b

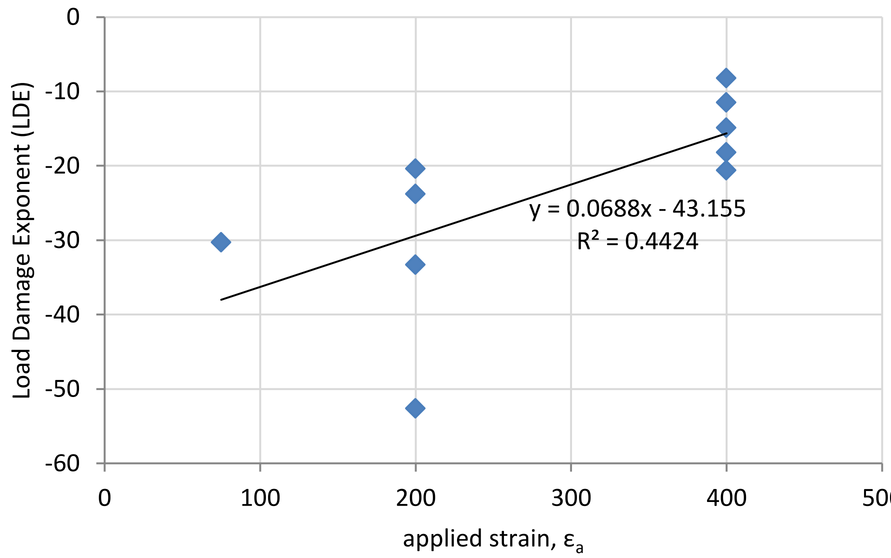


Figure 6.2: Load Damage Exponent LDE vs applied strain, ε_a

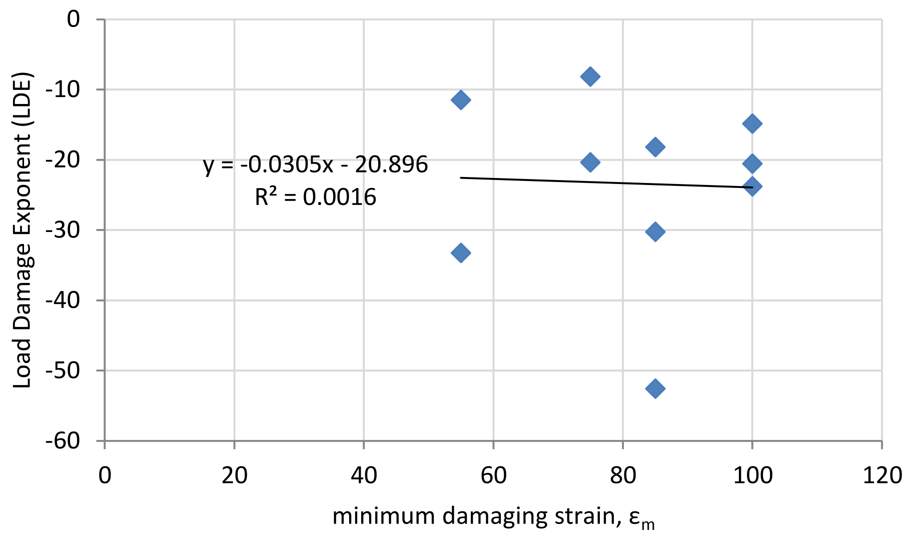


Figure 6.3: Load Damage Exponent LDE vs minimum damaging strain, ε_i

As seen above, the breaking strain and minimum damaging strain do not show any statistically credible relationship with LDE. Instead there is an appreciable linear relationship between LDE and applied strain. The strength of the relationship is statistically low, measuring an R^2 of 0.442. The applied strain however does not consider material characteristics and it shows some level of scatter at each applied strain. Furthermore, by extrapolating the results, it does not support the observation of pavement perpetuity below the minimum damaging strain.

Since it has been established that the LDE is a function strain, and based on the previous description of fatigue damage, the extent of damage for each repetition is hypothesised to be a ratio of the applied damage in excess of the minimum damaging strain against the breaking strain. That is to say, the LDE is a function of a ratio explained by the applied strain in excess of the minimum damaging strain divided by the breaking strain, i.e.

$$LDE = f\left(\frac{\varepsilon_a - \varepsilon_i}{\varepsilon_b}\right) \qquad (6.7)$$

where ε_a = applied strain

ε_i = minimum damaging strain

ε_b = breaking strain

Combining the data and analyses presented in Section 3.4 produces Figure 6.4 below showing the proposed function for LDE.

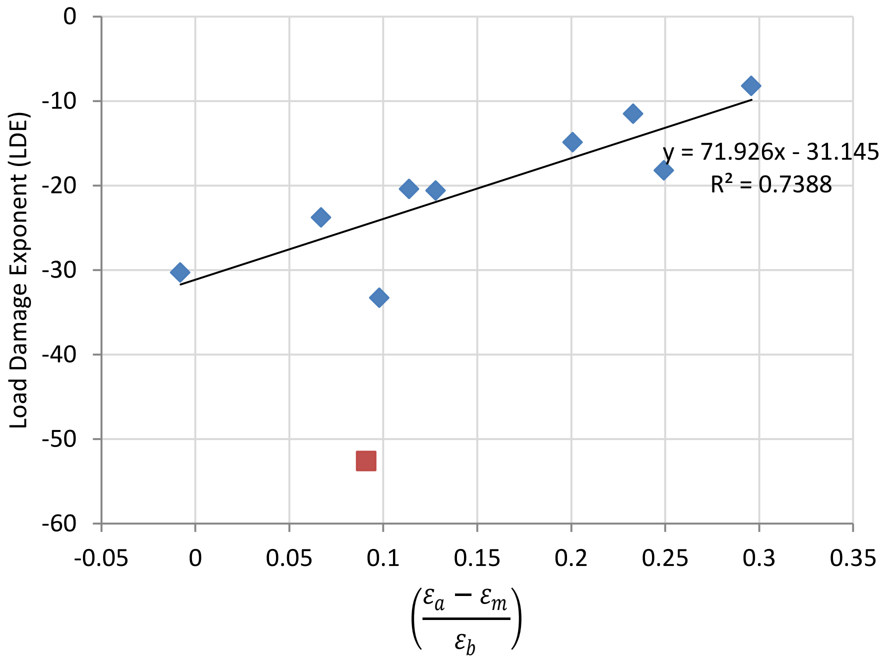


Figure 6.4: LDE vs. $\dfrac{\varepsilon_a - \varepsilon_i}{\varepsilon_b}$

It can be seen that a distinct linear relationship exists between the proposed function and LDE. The linear relationship is statistically strong with a least square regression, R^2, of 0.738. The relationship can be summarised based on the best fitting curve as,

$$N_f = 0.5^{72\left(\frac{\varepsilon_a - \varepsilon_i}{\varepsilon_b}\right) - 31} \tag{6.8}$$

For $0 < \left(\frac{\varepsilon_a - \varepsilon_m}{\varepsilon_b}\right) < 0.433$

The relationship supports the analyses presented above in that the pavement performs perpetually from a fatigue perspective when the minimum strain is not exceeded. A data point has been removed from the set, i.e. the LDE of 52.6. The author has chosen to treat this data point as an experimental error. This is justified by the significant deviation to the observed trend and the dubious results collected from the test – the analyses undertaken in Section 5.3.2 also showed that the 3%

cement content mix experienced an uncharacteristically high initial stiffness, high LDE and produced dubious readings. Based on the above function for LDE, a mathematical relationship can be established that if the applied strain is less than the minimum damaging strain, the pavement will perform in perpetuity. Likewise, should the pavement be strained in excess of 0.433, the pavement is expected to underperform as a cement treated layer.

Fatigue damage is translated to an in service pavement due to the continuously reduced stiffness of the pavement. The propensity of the pavement to deform also increases because of this transference. It is recognised that this change in material structure will alter the strain response of a material. However, it has been shown that the underlying support layer primarily determines the maximum permissible strain experienced by the cement treated basecourse layer and thus the fatigue strength of the material. The maximum permissible flexural strain will be influenced by the subgrade strength and the vehicular load applied from the top of the pavement. Also, based on the observations of the experimental result, it is not the development of tensile strains that causes the degradation of fatigue, but the coalescence of cracks within the material due to strain incompatibility. Overall, this reiterates the importance of having a sound solid subbase or subgrade under the cement treated basecourse layer to minimise the development of strain. In validating the strain behaviour of the material under traffic load, the strain response of a pavement is assessed in Section 6.3 below through a basic finite element model.

6.1.2.3 Review of experiment methodology

The four point bending test under constant strain had provided sufficient data to characterise the fatigue behaviour of cement treated basecourse material. Two sets of data were collected by performing an Elastic Strain Limit tests and an ultimate bening strength test on similar samples. This allows the data to be correlated effectively. For the Elastic Strain Limit, an analysis of the stiffness vs. the number of cycles was undertaken to determine the strain required to initiate fatigue. Meanwhile, the ultimate bending strength assesses the breaking strain of the specimen under flexural load. These two parameters were then used to calculate the fatigue life of a pavement

The ESL provided a good indication to the strain required at the onset of damage. However the tests have several inherent limitations:

1) the specimen size available at Curtin University is limited to 65mm x 50mm in cross section. This may make the specimen unrepresentative of the expected insitu behaviour of the material.

2) the fragility of the specimens encumbers the transfer in and out of the two different testing rigs when the same specimen is used.

3) the application of strain resulted in noise interference during the collection of data. Particularly, low strain application resulted in a high level of noise during data reading, making the determination of the average stress difficult.

Furthermore, the fatigue testing method utilised in this dissertation has shown to provide a consistent power function defining the behaviour of the material. Nontheless, it is recognised that this testing procedure also has its limitation. Significantly, the time required to fully test specimens would entail only testing one specimen per day. The time consuming nature of the test is of greatest impedance to the research. The variability of the test settings to represent actual insitu pavement performance is also questionable. This limitation is further discussed in Section 7.2.

6.1.3 Shrinkage

The shrinkage parameter considers the strain that will develop in a pavement structure due to shrinkage of the cement matrix and the subsequent development of cracks. In the composite structure of a cement treated basecourse layer, the cement matrix are bound to the aggregates that it surrounds and acts as a restraint for the cement matrix while it shrinks. This restraint causes the development of stresses within the cement matrix that result in the accumulation of stress ahead of the tip of the microcracks. Once the stress exceeds the breaking limit, cracks propagate within the matrix and breaks down the effective cohesion of the composite material. The shrinkage cracks will continue to propagate through the basecourse layer and 'reflects' onto the pavement seal, ultimately leading to visible damage on the pavement. The shrinkage problem has traditionally been a difficult concept to quantify and have been controlled by changing the mix design.

Based on the results presented earlier in Section 5.6, the shrinkage strains experienced by the specimens are +18,400 με, +15,600 με, +400 με, -800 με and -1,600 με for 1% to 5% cement content respectively, where positive values represent swelling and negative values represent shrinkage.

These results indicate that the 1% and 2% cement content mixes undergo significant compressive stresses due to the swelling of the cement matrix. Nevertheless, since the compressive strain capacity of the material is approximately 30,000 με (se Section 5.2.1), these swelling strains suggests that the material develops minimal permanent deformation when only 1% to 2% cement content is added.

Material with 3% cement content showed the least volumetric change, bordering on a neutral state and measuring only 400 με of compressive strain. This measurement constitutes to approximately 0.1 mm change in length, suggesting that 3% cement content performs the best in terms of shrinkage.

For specimens with 4% and 5% cement content, the shrinkage strains experienced by the specimens are 4 to 10 times larger than the minimum damaging strain of the material as discussed in Section 5.3.2, and are very close to the ultimate flexural / tensile strain capacity as described in Section 5.6. This propounds that permanent damage will occur for mixes with 4% cement content and severe cracking will ensue for mixes containing 5% cement content. These cracks would significantly reduce the serviceability of a pavement and result in early distress.

Furthermore, the volumetric changes of mesopores, i.e. pores measuring 2.5 nm to 30 nm, measured using the nitrogen adsorption method exhibit a substantial correlation to the results attained for the linear shrinkage test. This is in agreement with the literature reviews presented in Section 3.8 where the mesopores generate the largest suction forces. When water evaporates from mesopores, capillary action is sufficient to generate stresses effecting the overall structure of the composite material. Similarly, the increase in mesopores also indicates that the material will swell, which is believed to be caused by the swelling behaviour of fines of the virgin material. As the speicmens used for the tests are materials passing 425µm, there is a possibility of the specimen containing expansive material.

Nevertheless, results in Section 5.5 and 5.6 show an appreciable relationship between mesopore concentration and linear shrinkage. When the volumetric distribution of mesopores in 1 gram of material increases the overall material swells. Likewise, when the volumetric distribution mesopores in 1 gram of material decreases, the material shrinks. This relationship is plotted in Figure 6.5 below

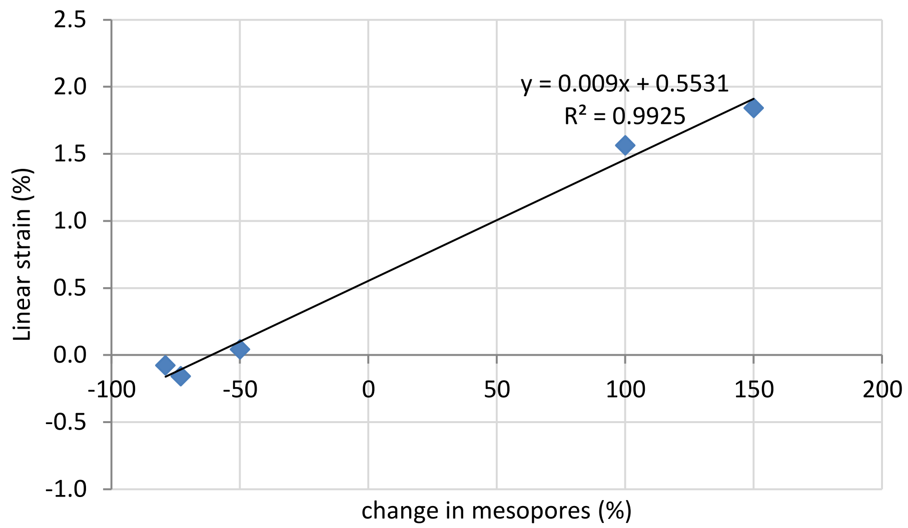


Figure 6.5: Change in mesopores (%) vs. linear shrinkage, LS (%)

As can be seen from the diagram above, a statistically strong relationship exists between the volumetric change in mesopores against the linear strain of a specimen. The relationship between the two parameters is defined by the following:

$$\text{Linear shrinkage (\%)} = 0.009 \times \text{mesopore change} + 0.553 \qquad (5.6)$$

The above transfer fucntion shows that the material generally swells due to the increasing volume of mesopores, presumably due to the expansive nature of the virgin material. However, it takes more than -60% change in mesopores in order to result in shrinkage behaviour of the material. This is to say that the material is more prone to expansion than to shinkage over a 7 day period. An optimum mix design can therefore be selected to minimise shrinkage, which is 3% cement content for the crushed rock basecourse material studied by this dissertation. However, the results are admitedly preliminary and further verification is required in order to provide something concrete to the practicing community. Furthermore, Moreover, the measured strain is arguably larger than that typically expected of in service pavements material due to the size of the specimen and the volumetric ratio of fines which means a larger surface area to volume ratio exists and therefore more shrinkage is expected.

6.1.3.1 Review of Laboratory Methodology

The shrinkage tests applied in this dissertation which includes a combination of Linear Shrinkage test and Nitrogen Adsorption has shown that the measurement of pore size distribution can be used to predict the shrinkage potential of a material. This combination of lab tests allows the assessment of the fundamental structure of the material at a nanoscale and correlating its results back to an observed behaviour as a composite material. The volumetric change in pore volume of mesopores strongly correlates to the measured linear shrinkage strain.

This means that by calibrating linear shrinkage results from a the nitrogen adsorption method, there is a possiiblity to provide a good indication on the material shrinkage behaviour by studying its pore structure. As the shrinkage rate of materials tyically stops after reaching a limit, a potential also exists to assess a materials insitu condition to assess whether it will undergo any further shrinkage and predict future volumetric changes that may effect its serviceability.

Nevertheless, an obvious limitation exists for the nitrogen adsorption methodology is the sample size. The sample size for each test is approximately 1g, which opens the uncertantiy of its reproducability and repeatability. More tests should be undertaken with the combination of XREM to ensure that each sample used for nitrogen adsorption is consistent.

6.1.4 Durability

The durability parameter has not been traditionally considered in the design of cement treated basecourse of pavements in Australia, primarily because it is not a well understood parameter in pavement engineering. It has traditionally been considered as an issue in areas susceptible to severe climatic changes, e.g. hot summers and cold winters where the pavement undergoes wet-dry or freeze-thaw cyclic stresses. This dissertation introduces another criterion for consideration, i.e. cement stabilisation's durability against the chemical retardation process known as carbonation. This criterion is particularly important as it dictates the permanency of the benefits gained from the stabilising process and thus the realisation of its benefit for the required service life of the pavement.

As presented in Section 3.7.2, the primary catalyst for the phenomenon of carbonation is water. This dissertation has shown that water travels through cement stabilised basecourse due to the sorption potential of the material, debunking the misconception that stabilised basecourse is impermeable. Besides triggering carbonation, the fluctuation of water levels in cement treated basecourse also causes other forms of degradation as discussed in Section 3.7.1. It is proposed that an assessment on the durability of cement treated crushed rock basecourse can therefore be based on its susceptibility to moisture ingress by applying the Tube Suction Test. This testing methodology allows the development of an implicit numerical quantification of the durability for cement treated crushed rock basecourse.

As discussed in Section 2.5.3, current MRWA design guidelines prohibit the use of modified basecourses and omits the strength gained from cement treatment due to concerns of the permanency of the stabiliser. These concerns stem from insitu observations of the Reid Highway trial sections constructed by MRWA as presented in Section 2.5.2. In those pavement trials, it was noted that the modification methodologies practiced in Western Australia was under threat of disappearance. The materials that have performed poorly after several years of service included Hydrated Cement Treated Crushed Rock Basecourse (HCTCRB) and cement treated basecourse materials with less than 1% cement content.

These insitu observations are consistent with the analysis of this dissertation in that low cement content basecourses are highly susceptible to moisture ingress as they possess a higher sorpitivity (refer Section 5.4 for detailed results). The increase in cement content generally reduces sorptivity because the materials become less porous with the build up of cement matrices within voids.

The process of producing HCTCRB involves remixing of stabilized materials. This causes a detrimental effect to the material as it increases its susceptibility to moisture ingress. Similarly, crushed rock materials with low cement content have a higher water cement ratio as shown in Section 5.1. This makes them highly porous with less dense cement matrices being formed. This ultimately increases the diffusion interface areas between cement phase and water, increasing the rate of carbonation.

Furthermore, the results have confirmed that the ingress rate of moisture into pavements obeys the principles of unsaturated flow theory, i.e. the rate of sorption is a function of the square root of time. The unsaturated flow theory is also similar to the theoretically carbonation depths typically used for concrete, providing the added confidence that the approach is sound. The results indicate that the material sorptivity generally reduces exponentially with the increase in cement content, with 3% in the median of high sorpitivity and low sorptivity materials. Also, by examining the pore size distribution presented in Section 5.5, there is a potential correlation between the pore size volume and sorptivity. Figure 6.6 below shows the relationship between macropores after 1 day of curing against sorptivity. Despite the high statistical relationship, the results are premature and do not provide any reliabile conclusion as no other relationship could be established from other available data.

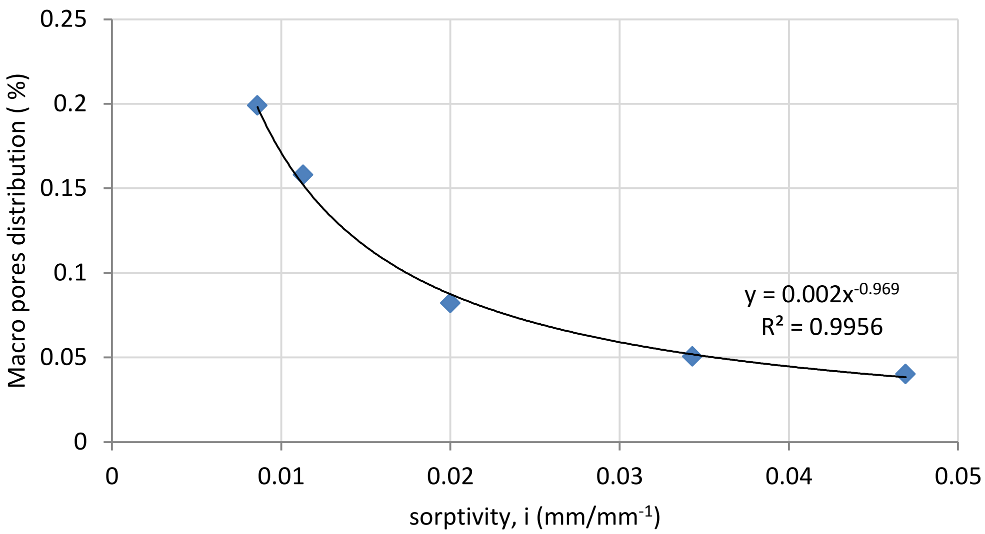


Figure 6.6: Macropores volume distribution (%) vs. sorptivity (mm/min^{-1})

This is further supported by the UCS measurements at the end of the soaking periods. The residual strength of materials, i.e. the UCS strength of the material after 10 day soaking versus the UCS of dry specimens, increases with cement content due to the reduced water content. This observation indicates that the measurement of sorptivity also provides a quantification of the residual strength as well as the characteristics of pavement in an unsaturated condition.

6.1.4.1 Tube Suction Test

The Tube Suction Test (TST) has shown to be a viable method to assess the behaviour of cement treated crushed rock basecourse under an unsaturated state, including its moisture susceptibility, residual strength after soaking and sorptivity. The procedure involves a three step approach of first drying a cement treated crushed rock specimen, followed by a 10 day soaking period and finally completing with a UCS test. Details of the processes are explained in Section 4.2.6. The dielectric values measured form the TST provide a reliable representation of the water content of the specimens as seen from the results presented in Section 5.4.

The procedure uses typical specimen sizes for geotechnical purposes that can be recycled to undergo other testing upon completion. This makes it a comfortable test to be applied in industry with existing tools. The specimen size also means that the UCS tests used to assess the residual strength of the material after soaking can be undertaken by applying known standards and procedures. As demonstrated in this dissertation, the measurements from the test provide the parameters required to assess the sorptivity of cement treated crushed rock basecourse, in turn providing a quantifiable measurement of durability.

The reliability of percometers to measure water content also means that once the dielectric values of a material are calibrated, the percometer may be used as a non-destructive measurement device to assess water content of insitu pavements. This is a highly favourable tool as the plant and labour required for the test is relatively cost effective to other non-standard tests currently available in the market. The procedure involved in undertaking measurements is simple and easy to carry out. The TST can also be extended to unbound granular material often used in the United States.

6.1.5 Erodibility

The erodibility index is applicable only for unsealed roads and may not provide any other useful data for sealed pavement application. However, with Western Australian population growth and the expansion of rural communities, the luxury of sealing roads may not be present all the time. In such situations, the erodibility parameter would provide a good indication of the behaviour of cement treated crushed rock basecoruse when used in an unsealed road.

The experiment data and analyses presented in Section 5.7 have shown that the increase of cement content increases the erodibility of the material. This is hypothesised to be a result of the increase in fines on the surface of the tested slabs, resulting in increased erosion.

However, the measured erodibility in all instances is trivial with maximum erodibility of 0.75mm for crushed rock slabs treated with 6% cement content.This presents an opportunity for further research to standardise the testing procedures that can provide the industry with useful data.

6.1.5.1 Review of Laboratory Methodology

As highlighted by De Beer (1989) covered in Section 3.9, one key important parameter for erodibility testing is the moisture condition of the specimens. The Cooper Wheel Tracking Machine available at Curtin University is not capable of accommodating a water bath to maintain the moisture condition of the specimen. As the nature of the test requires a long duration, the specimens typically dry out and proved to be inconsistent. This can be seen from the statistically low correlation of the results. The method of travelling the slab also limits the possibility of retrofitting a water bath to the testing rig, accelerating and decelerating the slab, would cause constant spillage. Significant modification would be required in order to control moisture conditions of the slabs.

The final readings of the test are also affected by the uneven surface of the finished specimen due to the cohesion between the steel drum compactor and the specimen surface. This is generally overcome by screeding the surface of the specimens while ensuring minimal disturbance to the specimen. The other limitations of the test include the lack of flexibility to alter the magnitude of applied stress.

Nevertheless, this dissertation has presented a preliminary concept for a measurement of erodibility for cement treated crushed rock basecourse and thus pioneering the opportunity for its use in unsealed roads. The methodology developed here can aslo be used for other surfacing requirements.

6.1.6 Summary of Material Behaviour

The above subsections have provided a thorough discussion on the works undertaken in this dissertation to characterise the behaviour of cement treated crushed rock basecourse. This included a combination of observations made during the experimental work and the literature review undertaken. Distinct behavioural traits are identified for the various mix designs of cement treated crushed rock basecourse, which can be used to define the application of the material. This is further covered in the following Section.

6.2 Reclassifying Cement Treated Basecourse

This section presents the discussion surrounding a key objective of the dissertation viz. the development of an appropriate mechanism to classify cement treated materials for pavement basecourse. It first presents a discussion on the limitations of current classification criteria based on the experimental data and analyses. This is then followed by proposed classification methodologies.

As discussed in detail in Section 2.2, the fundamental purpose for material classification is to assist pavement engineers to identify the performance attributes of the material and understand its behaviour during service. This will allow the engineer to design a fit for purpose pavement given the application, environmental condition and design life of the road. Ultimately, this design efficiency allows a pavement to meet sustainable imperatives through the minimisation of material usage.

In reflection of the above objective, current guidelines for cement treated materials as outlined in Section 2.2 classifies the material into either “modified”, “lightly stabilised” or “stabilised”, according to its UCS range and distinct performance attributes. The two defining parameters aim to determine whether cement treated basecourse is to be designed for fatigue failure or as an unbound material. The following subsections provide a discussion of the applications and limitations of the current classification criteria in Western Australia.

6.2.1 Discussion on UCS ranges and Quantitative Benchmarking of Fatigue Performance

As the laboratory data and analyses presented in Section 5.2 suggests, the measurement of Unconfined Compressive Strength (UCS) does not hold any mechanical significance. The values attained for cement treated crushed rock used in Western Australia is in gross excess of typical ranges recommended for “modified” (0.7 MPa to 1.5 MPa), “lightly stabilised” (1.5 MPa to 3MPa) and “stabilised” (more than 3.0 MPa). Furthermore, extrapolation of the results also shows that the material would have attained a significantly high UCS despite being untreated.

This incompatibility has been covered repeatedly throughout this dissertation, emphasising the need to identify other mechanisms to improve material classification and thus meet the objectives of pavement design. It is seen that among the reasons for this incompatibility is the dependency of UCS measurements on the source aggregate used in the preparing the pavement material. It is vital to note that the majority of materials originally used to develop the ranges proposed by Austroads are sourced from East Australian States. Given the geographical disparity between Western Australia and other states, the physical construct of these aggregates tend to exhibit different levels of compressibility. In consideration of the different climatic and geological conditions across Western Australia, the adoption of a blanket rule for Australia is injudicious.

Although the UCS ranges provided by Austroads are recognised by this dissertation to be but a guideline, practitioners often adopt them within technical specifications under concerns of professional indemnity in contractual engagements and the lack of other guidelines. As seen in Section 5.2, the misinterpretation of the UCS ranges causes various issues to material usage in Western Australia, where the practice is generally to limit the application of cement to less than 1% cement content in order to prevent “stabilisation”.

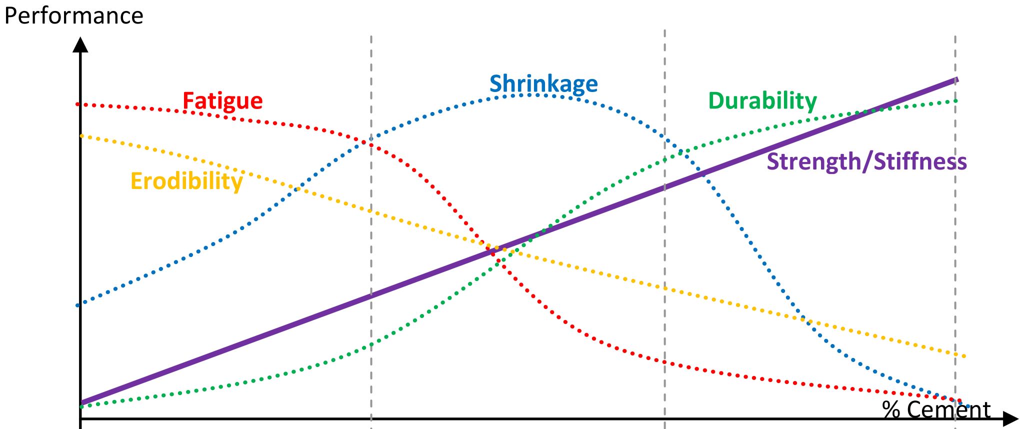
Furthermore, the current performance attributes adopted by Austroads to classify cement treated basecourse materials primarily focuses on the propensity for a material to undergo fatigue. This is aimed at allowing the pavement engineer to select the appropriate transfer equation to design the pavement thickness and its subsequent service life. However, as shown in this dissertation, there are other performance imperatives to consider when designing pavements. These include:

- fatigue
- erodibility
- moisture sensibility and durability
- shrinkage

The relevance of each of the criteria above is dependent on the application of the road and mix design of cement treated crushed rock basecourse, i.e. cement, water and aggregate. The following section therefore further discusses on an alternative classification based on application and mix design

6.2.1.1 Classification Based on Application and Mix Design

The experimental results have shown that the primary change in behaviour is dependent on the cement content of the material. Figure 6.7 graphically shows the indicative interrelationship between cement content, the gain in strength and the performance properties of cement treated crushed rock in Western Australia based on the work of this dissertation.



Cement Content	Low 1% - 2%	Medium 3%	High 4% - 5%
Advantages	- Fatigue irrelevant - high erodibility	- Minimal volumetric change - Balanced performance	- High strength, high durability
Disadvantages	- Low strength - Swelling - Low durability	- Susceptible to fatigue	- Highly susceptible to fatigue - Undergo shrinkage
Application	- Unsealed roads - Arterial Roads	- Highways	- Floodways - Interstate
Equivalent Austroads Classification	Modified	Lightly Stabilised	Stabilised

Figure 6.7: Advantages and disadvantages of different classification

Figure 6.7 above outlines the advantages and disadvantages of increasing cement content and the subsequent application of the composite product. As noted, the different mix designs provide various advantages and disadvantages that dictate the applicability of the material. The material characteristics of low cement content mixes (1%-2%) make them suited for low volume and low traffic loads. Subsequently, the medium cement content mix (3%) has more balanced characteristics and is deemed suitable for highway construction. High cement content mixes (4% to 5%) are distinctly noted for its high strength and durability but are susceptible to fatigue and shrinkage. Consequently, high cement content mixes are more suited for low volume but heavy load applications (interstate links) and road sections frequently exposed to harsh conditions (floodways).

The above also shows a correlation with the equivalent Austroads Classification, implying that the current classification types adequately differentiate the various behaviour groups. Nevertheless, the above discussion suggests a revision to the proposed performance attributes by Austroads with the introduction of three new factors, namely shrinkage, erodibility and durability. These factors are critical performance attributes and are relevant to cement stabilisation.

6.2.1.2 Water Content in Mix Design

Thus far, water content for the material has been selected for optimum moisture content under modified compaction test. Although not studied in detail, the water cement ratio has been noted to impact material behaviour due to the changes in density of the cement matrix. Therefore, it plays a crucial role in the determination of material applicability. This section presents a diagram of the interrelationship between water cement ratio and the different material products based on the literature review and results undertaken. Figure 6.8 below presents the different categories of materials achieved from varying cement content, water content and to a lesser extent the compaction effort. The figure is adapted from Thom (Thom 2010) and PCA (Portland Cement Association 2005).

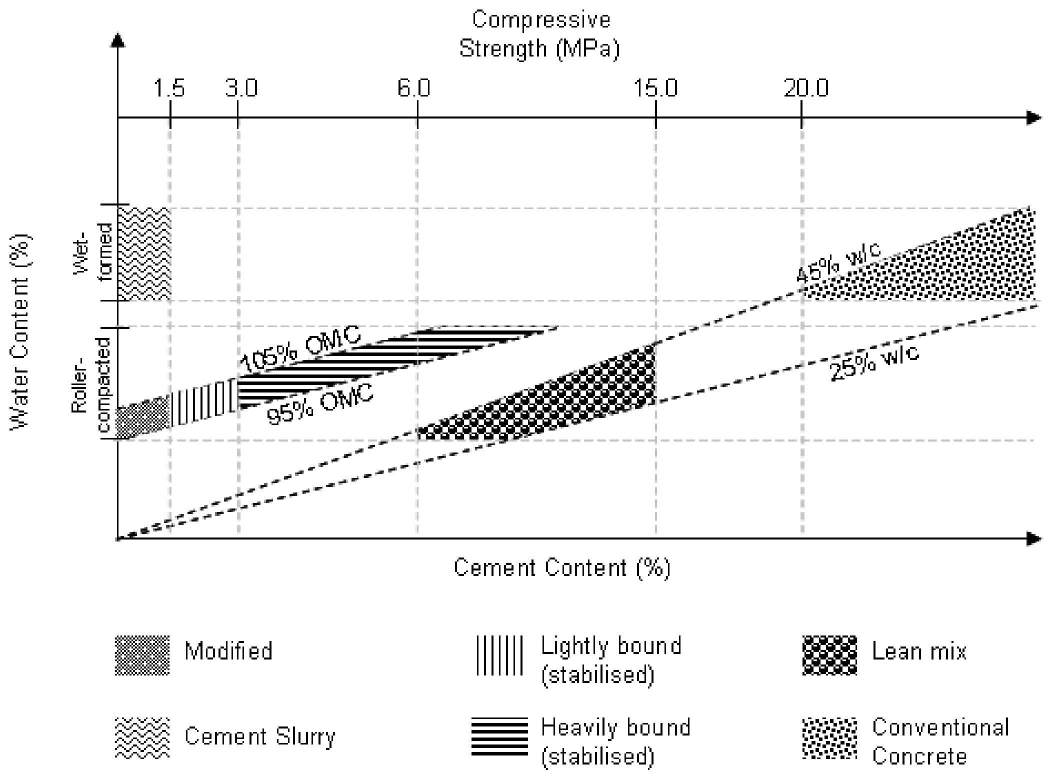


Figure 6.8: Mix design chart for cemented materials

Varying the composition of water and cement content allows various materials to be formed, including the cement treated basecourse, lean mix concrete, slurry mixes and conventional concrete. For low water cement content mixes, compaction effort is required to provide material strength while material with higher water content is wet formed. This approach of classifying materials based on water cement contents presents itself as an alternative option to material classification.

The required water content to achieve compaction for the tested material means that the cement are overly saturated, porous and weak in strength, a distinct trait that differentiates cement treated basecourse from conventional concrete. However, since the resistance of the material is predominantly gained from the shear resistance of the aggregates, the cement matrix that forms around the aggregates provides added strength.

As noted from Figure 6.8 above, the OMC for compaction of cement treated basecourse is the OMC of the parent material + 0.25% for every 1% in cement content. This relationship runs parallel to the minimum water required for effective hydration to take place, i.e. a w/c ratio of 0.25. It is believed that this occurs due to the absorption of water by the cement paste for hydration and the reduction in fines due to the conglomeration of fines within the cement matrix.

It is however recognised that this section requires further evaluation and study, a note discussed in 7.2 which touches on the limitation of this dissertation.

6.2.2 Quantifying the Reclassification of Materials

The discourse above identified that current classification methodology based on categorizing materials as “modified”, “lightly stabilised” and stabilised” adequately differentiates material properties. Even so, using UCS as a measurement of these classifications has been found to be inappropriate. The following sections discuss quantification of the various classifications based on the actual physical traits of the materials, which includes the portion of cohesion in strength, the onset of fatigue, shrinkage tendency, durability and shrinkage.

6.2.2.1 Portion of Cohesion

The progression from modified to stabilised classification is the shift from a dependency on shear strength gained from material interlock to tensile strength gained from the cement matrix. In other words, a method to quantity this shift is by assessing the cohesion of the mix design contributing to the shear strength of the material. Figure 6.9 below shows this relationship.

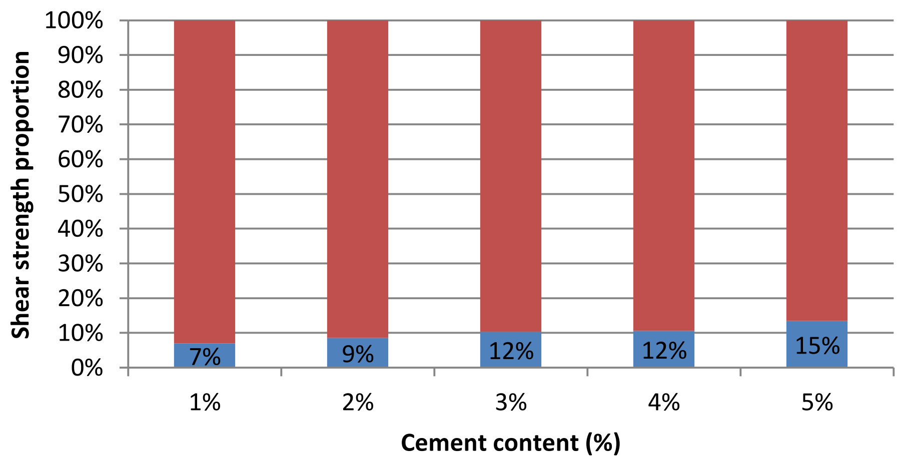


Figure 6.9: Portion of cohesion in shear strength

The above results are generated from the UCS and ITS tests as covered in Section 5.2, both test methods already familiar to the Australian soil testing industry. The results indicate that the percentage of cohesion contributing to the overall shear strength of the material increases with cement content. It is hypothesised by this dissertation that an appreciable percentage contribution is approximately 10% of the total shear strength, which would imply that more than 90% of the material depends on material interlock. However, no direct inference can be made by just analysing the percent contribution of cohesion to the design of the material, which largely is dependent on performance attributes.

6.2.2.2 Onset of fatigue - Minimum Damaging Strain

Since the premise for quantitatively determine the various classification of cement treated crushed rock is to primarily identify the propensity of a material to undergo fatigue, it is the opinion of this dissertation that a measurement directly linked to fatigue initiation better achieves this objective. Since the degree of a fatigue is a function of minimum strain damage, ε_i as demonstrated in Section 6.1, this dissertation puts forward the approach of using Elastic Strain Limit testing to ascertain the onset of fatigue damage. Namely, an incremental strain is applied in stages until a deviation from the power function is detected.

As proposed by Austroads (Austroads 2010), the typical strain levels experienced by cement treated basecourse layers is less than 100 με. It is proposed that a material is to be deemed susceptible to fatigue should the onset of fatigue be detected within a tested range of 100 με.

6.2.2.3 Shrinkage Tendency

Based on the Linear Shrinkage tests, the tendency of a material to shrink evidently change from a swelling characteristic to a shrinkage characteristic.

6.2.2.4 Durability

Durability increases with cement content due to the reduced sorptivity of the material. Sorptivitiy reduces more distinctly between 1% to 3% cement content compared to the reduction of sorptivity between 3% to 5%.

6.2.2.5 Summary of Quantifying Cement Stabilisation Classifications

Based on the preceding discussion, the different quantifiable measurements to be checked in order to ascertain material performance attributes can be summarized in Table 6.1 below.

Table 6.1: Summary of quantifying cement stabilisation classifications

Cement Content	Modified	Lightly Stabilised	Stabilised
Strength (% cohesion)	<10%	≈10%	>10%
Fatigue (minimum damaging strain)	>100	>80	<80
Shrinkage (linear shrinkage)	+strain (swell)	≈ 0	-strain (shrink)
Durability (sorpitivity)	>0.1	0.05 < x <0.1	≤0.05
Erodibility (erodibility index)	0.4	0.5	>0.6

Regardless, an appreciable and distinct linear relationship does exist for UCS and cement content. By establishing a corresponding UCS value for each known material type, the material mix design can be calibrated to the UCS value and used by the industry as a quality control mechanism.

6.3 Validation through Numerical Numerical Modelling of Fatigue Response

A Finite Element Model (FEM) is developed using the software package Strand7 to validate the strain behaviour of cement treated crushed rock basecourse. Using results from the study undertaken, the insitu behaviour of a pavement is also predicted.

A model of the Four Point Bending Test undertaken for the ELT as seen in Section 4.2.5 is constructed to validate the material properties identified in Section 5.3 which. The pavement model used by Austroads (refer Section 3.2) is then presented.

6.3.1 Model Assumptions

The assumptions made to develop the two numerical models are presented in the following subsections:

6.3.1.1 ELT Model

Figure 6.10 below shows the model constructed for the ELT.

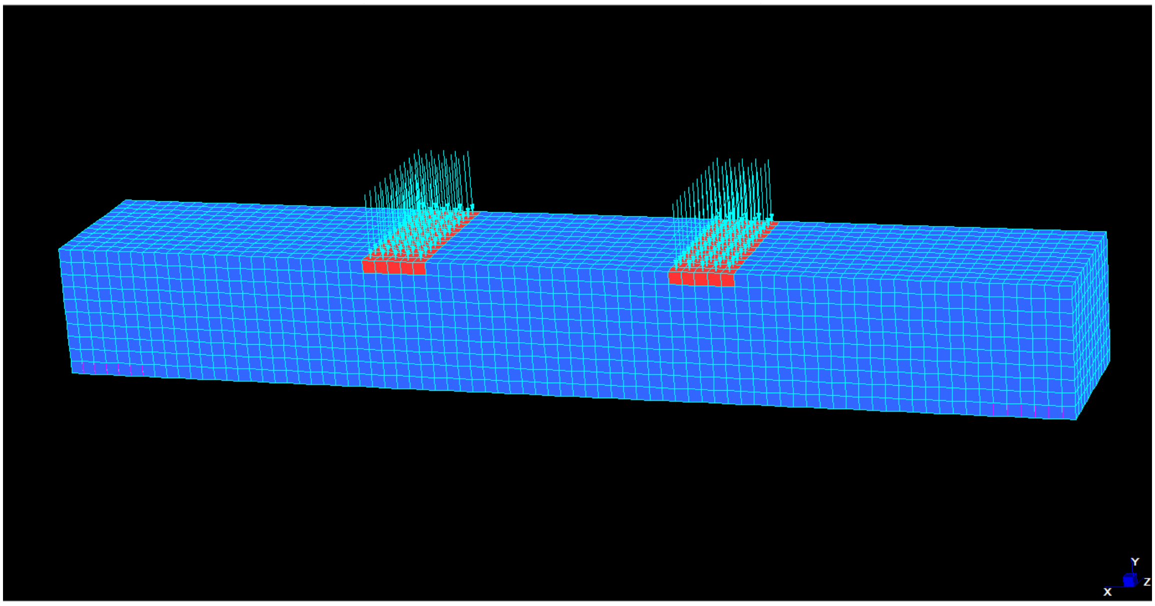


Figure 6.10: Four point bending test model

Parameters of the model are summarised below.

- Beam thickness = 65mm
- Beam width = 54 mm
- Beam length = 390 mm
- Loading platen widths = 25 mm
- Support = 25mm wide roller supports at 133.5 mm centres
- Elastic linear performance calibrated from ELT results

6.3.1.2 Pavement model

The pavement model has been developed based on the Equivalent Standard Axle Load (ESAL) configuration as presented in Section 3.2 and is shown in Figure 6.11 below. As the pavement is symmetrical, only half of the axle is created.

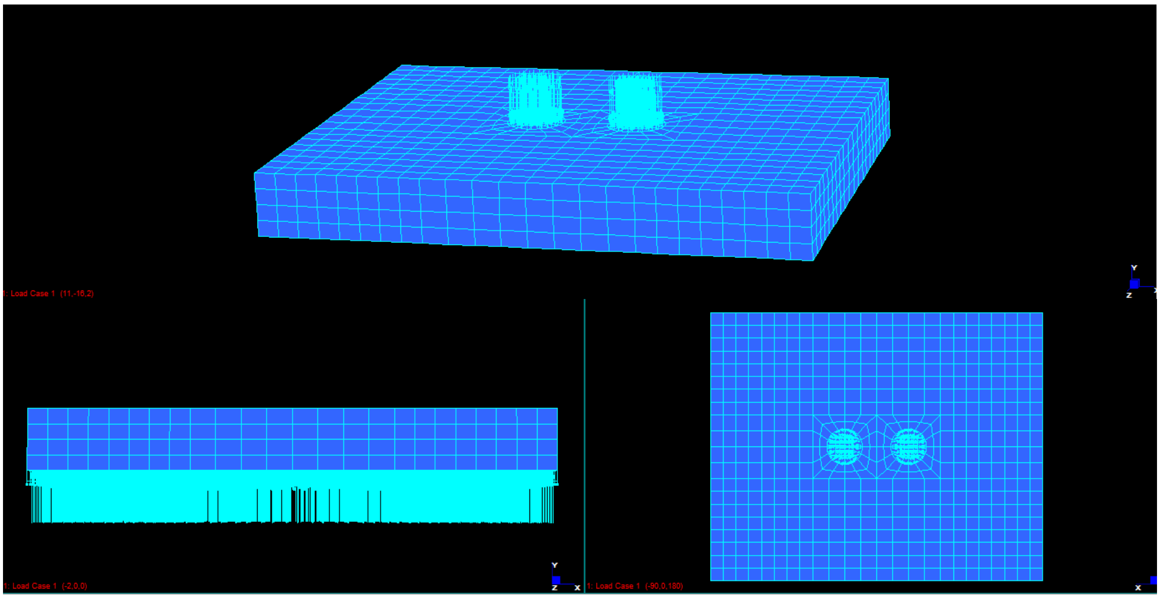


Figure 6.11: Pavement model showing perspective model (top), cross section (lower left) and plan (lower right)

Parameters of the model are summarised in below.

- Subgrade reaction, k_s = 1/3 to 1/8 x 10000 x CBR (kN/m^3) (6.9)
  (Nascimento and Simoe 1957)

  Therefore, = 11,500 kN/m^3 corresponding to 5% CBR

  = 34,500 kN/m^3 corresponding to 15% CBR

  = 69,000 kN/m^3 corresponding to 30% CBR
- Pavement thickness = 150 mm and 200 mm
- Slab width = 1750 mm
- Slab length = 1000 mm
- Restraint conditions = fully pin support on outer edges to prevent sliding
- Diameter of load = 92mm as per Austroads (Austroads 2008)
- Applied stress = 750 kPa as per Austroads (Austroads 2008)
- Elastic Linear performance based on the specific performance tables calibrated from above

6.3.2 Results of Model

The experimental result for the ELT test vs. the model is shown in Figure 6.12 below.

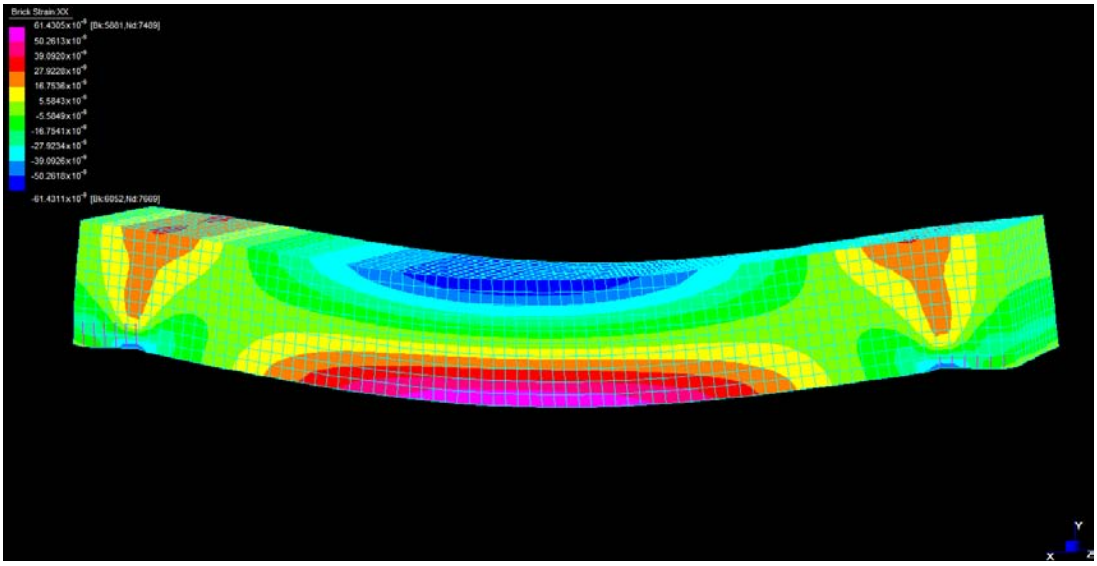


Figure 6.12: Results of four point bending test

As anticipated, the Four Point Bending Model illustrated above shows that a fairly constant strain envelope is sustained at the bottom fibres of the specimens.

The model above validates the material model generated in Strand7, showing that it conforms to the ELT results. The material property is then imported into the pavement model and a linear elastic analysis is conducted. The overall deformation for the pavement model is shown in Figure 6.13 below.

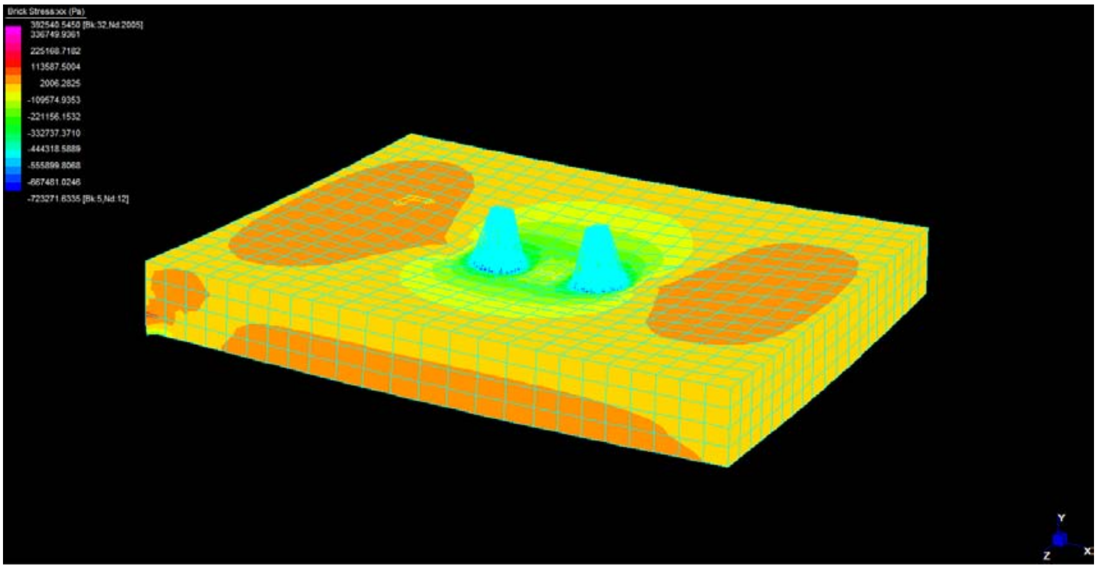


Figure 6.13: Results of pavement model – perspective view

Figure 6.13 presents a typical pavement slab deformation model where a build up of compression forces immediately under the footprint of the wheel loads is observed (blue and green contours). However, some minor tensile stresses are developed on the outer edges of the wheel (brown contours). This is due to the restraint of the confinement of adjacent pavement.

A cross section is taken through the XY plane as shown in Figure 6.14 below to demonstrate the deflected shape and stress contours of the model.

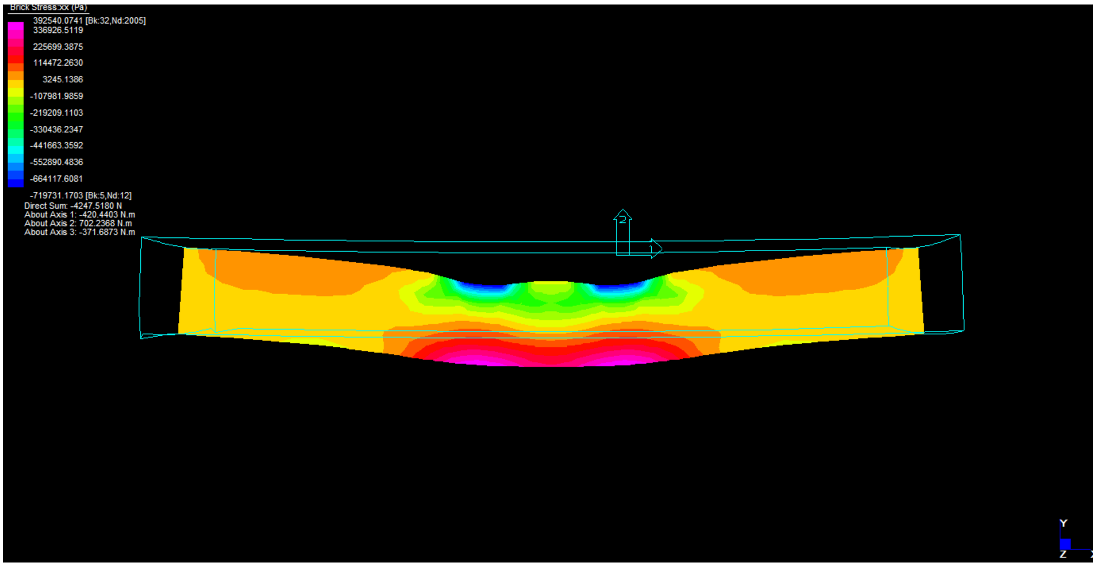


Figure 6.14: Results of pavement model – section through centre of slab

A concentration of tensile stresses is noted at the base of the slab, agreeing with the phenomology and structural model presented by Austroads. Interestingly, the restraint conditions of the slab emulates that shown on the Four Point Bending Test, suggesting that the four point bending test does provide a reasonable representation of in-service conditions.

Furthermore, a cross section is taken on the XZ plane to demonstrate the plan contours of the stresses at the base of the slab above the subgrade. This is shown in Figure 6.15 below. As seen a concentration of tensile stresses (pink and red shades) are evident with compressive stresses developing immediately adjacent to the wheel loading locations (blue shades).

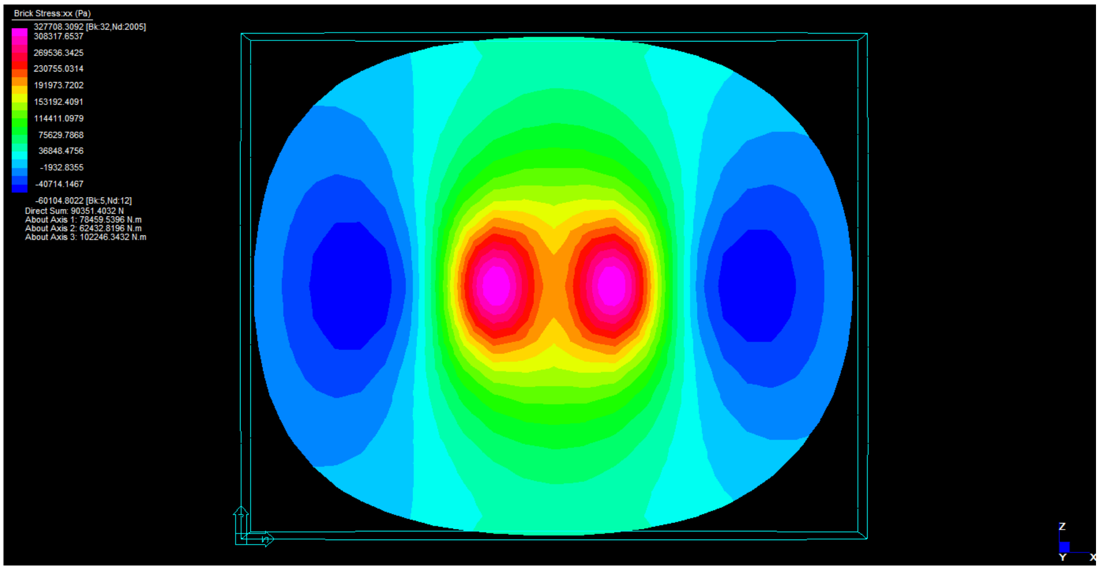


Figure 6.15: Results of pavement model – plan section through base of slab

By comparing the above data, the strain at the base of the slab can be calculated and the service life of the pavement subsequently determined.

Table 6.2 below shows the data from the numerical model.

Table 6.2: Predicted service life table

Cement Content	Pavement Thickness	Subgrade CBR					
		5		15		30	
		strain	N	strain	N	strain	N
3%	200 mm	466με	6.2×10^2	380 με	1.9×10^4	325με	1.6×10^5
4%	200 mm	157με	6.9×10^7	139με	1.3×10^8	124με	2.1×10^8
5%	200 mm	97με	7.9×10^8	89με	1.1×10^9	81με	1.6×10^9

Table 6.2 provides an indication of the interrelationship between subgrade CBR, but is not intended to be used for design. It may be developed further by addressing the limitations of this dissertation as covered in Section 7.2.

6.4 Summary of Discussion

In conclusion, this chapter has proposed an alternate approach to cement treated basecourse classification. This approach provides the engineer with more information in selecting materials fit for the intended purpose of a pavement. The chapter has also quantified the classification criteria and validated the fatigue model of cement treated crushed rock basecourse with a numerical model. These outcomes subsequently address the aims and objectives of this dissertaton, identifying limitations of the scope of research work undertaken and allowing recommendations to be drawn. These are covered in the subsequent chapter.

7 Conclusion

The literature review, extensive experimental work and numerical modelling conducted in the writing of this dissertation has produced a framework leading to a better understanding of cement treated crushed rock basecourse in Western Australia. Characteristics of the material, viz. strength, fatigue, durability, shrinkage and erodibility, have been identified, leading to a reclassification of the material, that allows the pavement engineer to optimise pavement design. Although providing significant advancement into the field of pavement engineering, the work undertaken in this dissertation is but the groundwork for more extensive studies. In closing, this chapter presents:

- A review of the objectives
- Limitations of the works
- Recommendations to the field of pavement engineering

7.1 Review of Objectives

The section provides a review of the dissertation against the objectives as presented in Section 1.5. It serves as a quick summary against these objectives and portrays the significance of the work.

1) *Develop and undertake a laboratory program to assess the fatigue mechanism of cement treated crushed rock with various cement content*

   The ELT was developed to assess the minimum damaging strain, providing an indication of strain at the onset of fatigue. This is combined with the repeated strain test of the Four Point Bending test typically for assessing the fatigue response of asphalt.

2) *Develop a fatigue model of cement treated crushed rock basecourse with various cement content and validate the fatigue model through finite element modelling methodologies*

A strain based model was developed as shown in Seciton 6.1.2 which shows a statistically moderate relationship (R^2 > 0.7) to the measured fatigue performance of cement treated crushed rock, i.e.

$$N_f = 0.5^{72\left(\frac{\varepsilon_a - \varepsilon_i}{\varepsilon_b}\right) - 31} \quad (7.1)$$

3) *Develop and undertake a laboratory program to assess the shrinkage potential of cement treated crushed rock basecourse with various cement content*

The N^2 adsorption and Linear Shrinkage test showed a good correlation and potentially explains shrinkage performance of cement treated crushed rock. The N^2 adsorption test analyses the change in pore structures and specifically allows the measurement of mesopores contained in the cement and fines mixture. This measurement correlates with the Linear Shrinkage test undertaken.

4) *Develop and undertake a laboratory program to assess the mechanism of moisture ingress into cement treated crushed rock with various cement content and subsequently its effects against stabiliser permanency*

The Tube Suction Test methodology provides an indication of the sorptivity of cement treated crushed rock basecourse and confirms that the rate of moisture ingress is related to the unsaturated flow theory. This also correlates with known theory concerning the extent (depth) of carbonation

5) *Develop and undertake a laboratory program to assess the erodibility of cement treated crushed rock with various cement content*

A preliminary Wheel Tracking Test method has been developed to ascertain the erodibility of cement treated crushed rock basecourse. The test method provides a promising approach to assess a material's erodibility when used for unsealed roads. The test also shows a correlation between erodibility and cement content.

6) *Develop a classification methodology based on the characteristics assessed in the research*

   The current classification methodology is expanded by providing an added dimension of durability, shrinkage and erodibility. This allows the pavement engineer to better select a mix design based on the intended application of the road. A further classification methodology based on water cement content is proposed but requires further development.

7) *Develop recommendations for stabilised pavement guidelines for Western Australia and Australia as a whole*

   Recommendations developed from this dissertation are covered in Section 7.3. In general the recommendations covers opportunities for future wor and revision of ERN9 to allow for outcomes of this research.

Accordingly, the dissertation has met all of its objectives, and provided a solid framework for further investigation of other important factors yet to be addressed. The development of testing methodologies will allow future researchers and practitioners to undertake similar experimental work to characterise cement treated materials. By collating these future works, the data pool will improve confidence in the developed methodologies, potentially allowing them to be rolled out nationally. Nevertheless, limitations exist for the work undertaken. This is covered in the following section.

7.2 Limitations

The works undertaken by this dissertation is limited by the following:

- the repeatability and reproducibility of the works undertaken in this test have not been assessed. This can be achieved by undertaking more samples and by different operators.

- The least square regression, R^2 of the data collected suggests that the statistical strength of the collected data can be improved. The R^2 for fatigue data collected in Section 5.3 and erodibility index in Section 5.7 is to be improved further before use in industry.
- The sample size used in this dissertation is relatively low due to the scope of the work and the available timeframe. This is especially true for fatigue tests due to the time consuming nature of the testing procedures.
- The 1% cement treated crushed rock specimens used for fatigue testing are very fragile. Although some specimens were successfully used to complete testing, a high loss rate was experienced, limiting the sample size.
- other parameters potentially affecting the results have not been studied. These include:

  a. Aggregate Type

  The dissertation has focused only on crushed rock basecourse found in Western Australia. As emphasised in the previous chapter, aggregate type plays an important role in determining material behaviour

  b. Water cement ratio

  Section 6.2 showed that the water cement ratio plays an important role in material classification. This area was not studied by this dissertation as it is beyond the scope of the research. The dissertation has applied the OMC to achieve MMDD.

  c. Particle Size Distribution

  The process of varying the particle size distribution constitutes as a mechanical stabilising process. There is an opportunity to combine mechanical stabilisation and chemical stabilisation to achieve an improved material performance. However, this was an area not studied by this dissertation as it is beyond the defined scope and objectives.

d. Binding Agent

Countless types of cement exist in the industry. Although GP cement has been traditionally used in Western Australia, there is a tendency to use Low Heat (LH) cement in current construction works to minimise the disadvantages associated to cement stabilisation, e.g. shrinkage.

e. Variation in Curing Method

The curing method adopted in this dissertation controls the relative humidity of the curing process but does not study the impact of relative humidity and to the cement hydration process. Various other curing methods exists that can potentially better emulate insitu curing conditions.

f. Temperature

The geographical span of Western Australia means that the state experiences various climate conditions ranging from the high $40^{o}C$ to low $15^{o}C$ throughout the year and at different locations. The study thus far has been undertaken in an controlled ambient temperature environment.

g. Cement Curing Time

The curing time of specimens plays a significant role in material behaviour. The dissertation has controlled curing time to 7 days to improve the turnaround of laboratory data and to emulate the preference of practitioners.

h. Presetting Time

The setting time for cement treated crushed rock after mixing but prior to compaction has been identified to affect the characteristics of the material. This factor has not been assessed in this dissertation.

i. Compaction Effort

The modified proctor compaction has been used in this dissertation only. The affects of varying compaction effort has not been studied.

- Constraints of Equipment

  The low level strain tests undertaken in this dissertation showed a high level of noise. This is caused mainly to the fluctuations of supplied air pressure to the apparatus which experienced numerous surges. The specimen size undertaken were also considerably small due to the limitation of the testing jig available at Curtin University.

- Numerical modelling assumptions

  the numerical modelling works undertaken in Section 6.3 are based on the experimental data of the research which in turn is limited by the factors. The applicability of the design data to in situ roads is not recommended and further investigation is advised.

7.3 Recommendations

The following recommendations are made:

- the study of cement treated basecourse can be expanded by considering various other parameters that have not been studied as covered in Section 7.2.

- the application of nanotechnology and other chemical additives to manipulate the behaviour of cement treated crushed rock baescourse is to be undertaken.

- the repeatability and reproducibility of the works can be improved by undertaking similar studies in other locations and by other operators. This will improve the confidence of the developed methodologies and encourage national adoption.

- the proposed fatigue model has some merit and meets the postulation of strain and fatigue. Upon further experimental work, trial pavements can be constructed with GRP sensors to validate the fatigue model. The key measurement is the stiffness of the pavement over the number of repeated equivalent single axle loads. In validating the model, future works can utilise the proposed fatigue model by only undertaking the ELT and FBT, which provides the minimum damaging strain and breaking strain. These two tests provide a relatively higher turnaround compared to full fledge fatigue testing.

upon the validation of the fatigue model as per the point above,

- Table 6.2 as shown in Section 6.3 can be formalised by undertaken further FEM modelling works. The Table can be then used as a quick guide to practitioners to adequately design for cement treated crushed rock pavements. It is also recommended that FEM be used more extensively in pavement design.
- it is recommended that UCS be used for quality control purposes only and discontinue as a blanket classification, unless specific UCS values have been calibrated against the performance attributes required of the material.
- pavement design using cement treated crushed rock basecourse is to be completed by adopting the revised classification methodology as presented in Section 6.2. This will include an assessment of the application of the road and required attributes of the road. The classification methodology builds upon current classifications by Austroads, but includes other essential attributes, viz. shrinkage, durability and erodibility.

- develop minimum cover concepts or impermeable barriers for pavements constantly inundated to improve the durability of cement treatment against carbonation

- review the use of Hydrated Cement Treated Crushed Rock Basecourse due to its high permeability and hence high susceptibility to carbonation. The dissertation has shown that a fundamental method to limit carbonation is to decrease permeability, which opposes the concept of HCTCRB.

- use of a larger rig in performing Four Point Bending test to minimise issues surrounding the fragility of the material and potentially eliminating size effects. The rig should also have more precise air supply to minimise noise during fatigue tests.

- the current durability study provides only an inferred durability of the cement treated crushed rock. It is recommended that further studies be undertaken to specifically assess the carbonation rates of materials and to formulate the numerical model of the chemical and mechanical mechanism of carbonation.

- revise MRWA ERN9 to allow for the use of stabilised materials by undertaking the necessary steps to design for the following properties based on the transfer functions proposed in this dissertation:

    - durability

    - fatigue

    - shrinkage

    - erodibility for unsealed roads

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APPENDIX A

Publications

Sustainability Assessment of Hydrated Cement Treated Crushed Rock Base (HCTCRB)

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1 Introduction

From 2000 – 2004 alone, $7.73 billion have been spent on road rehabilitation in Australia (BITRE 2008) with 1250 Mt CO_2 equivalent emission released and 13.9 terajoule of energy consumed for road constructed in the reported year (Hendrickson et al. 2006). Though with the abundance of natural aggregates in Australia, crushed rock used in road construction are nevertheless non-renewable resources and should be efficiently utilised. This invokes an economical and sustainable urgency to select optimum base course materials to reduce the environmental and economical footprint of road construction.

Mroueh et al. (1999) highlighted that the most significant environmental burden in road construction is caused by the manufacturing and transport of road construction materials where the dominant environmental loading is the consumption of natural materials. This reaffirms the pivotal step of optimising pavement material selection to improve the sustainability of road construction. The increase of number and sizes of road vehicles also require higher performing pavements and the use of higher quality but scarce pavement materials.

Stabilisation allows the use of otherwise mechanically inadequate materials for heavier traffics (Nikraz 2009), therefore, in consideration for logistics and resources availability, stabilisation has been deemed as a viable solution to improve sustainability of the transport industry (Ministry for the Environment 2003). The use of cement stabilised base course materials to increase the service life of pavements potentially reduces the economical and environmental footprint of Australia's road network.

In Western Australia, a relatively new engineering advance product known as Hydrated Cement Treated Crushed Rock Base was developed as a form of modified cement treated base by Main Roads Western Australia to provide a solution for increased performance requirements of heavily trafficked pavements. It has since been used at Reid Highway, Tonkin Highway, and major freeways, at a total estimate of 250,000 tonnes (Kelley 2009) to date and is seeing further increase of application.

1.1 Hydrated Cement Treated Crushed Rock Base (HCTCRB)

HCTCRB was a brainchild of Main Roads in light of premature failures caused by excessive deflection of the base course layers between South Street and Forrest/Yangebup Road on the Kwinana Freeway in 1992 (Butkus 2004).

It is a product of cement treated crushed rock which is stockpiled to allow hydration for a period of days and disturbed/retreated to form cement coated crushed rocks, ultimately used as a form of unbound granular material. It was identified that the cement treatment of the crushed rock base provided increased mechanical performance (Butkus 2004; Jitsangiam and Nikraz 2007) and extended pavement serviceability.

In comparison to HCTCRB, Cement Treated Base (CTB) is predominantly unfavoured in Western Australia because it undergoes shrinkage cracking during hydration (Chakrabati and Kodikara 2007) which eventually results in the early distress of pavements. However, studies have provided other mitigation measures for shrinkage cracking, such as, material proportioning, using additives, construction control and other physical alterations (George 2002; Adaska and Luhr 2004 ; Scullon et al. 2005). Among them is the successful practice of micro-cracking applied in the United States developed by the Portland Cement Association (2005). Microcracking basically forms miniscule networks of fine cracks right after initial curing to prevent the formation of wider and more severe cracks (PCA 2005).

Nevertheless, with its prematurity in the industry, not only are the mechanical properties of HCTCRB has yet to be fully understood, but more importantly the economical and environmental impact of its use has yet to be studied.

1.2 Sustainability assessment of pavements

With the severe impacts of road construction, studies on sustainability and the selection of pavement materials or technology have been undertaken extensively around the world. This section discusses briefly on approaches for sustainable pavement selection.

The predominant approach in assessing sustainability had been focused on Life Cycle approaches, i.e. to evaluate the complete life cycle of pavements to determine best practices and options. With the triple bottom line in mind, assessments have been generally conducted to ascertain environmental and economical impacts of road construction. Life Cycle Cost (LCC) analysis has been the prominent tool utilised to determine whole of life cost and have been utilised to assess environmental impacts by taking into account the factor on a monetary model as shown in Chan (2007).

In relation, Life Cycle Inventory Analysis (LCIA or LCA) as per ISO14040:1998 which is established based on inputs and outputs of each life cycle stages have been a well received and widely used tool in the evaluation of environmental loadings caused by construction material. The Built Research Establishment (1998) had been a proponent of the use of LCA in construction, establishing a database of environmental performance of building products in 1998. Moreover, LCA has been recognised by industries as an accepted tool for asphalt products and laying processes (Bird et al 2004). Specific LCI studies had been under taken by Eskola et al. (2004) and Birgisdóttir (2005) to develop LCI models for the determination of environmental impacts. Chiu et al. (2007) was also utilised LCI to ascertain areas of improvement for the production of pavement materials.

Nonetheless, Life Cycle Inventory assessments have potential for misuse as it is open for interpretation by the analyser (Jacquetta et al. 1994), leading to biased and erroneous results, however, it was also iterated that with proper control and standardised approached, it is a significant tool serve as a base line for making decisions.

2 Objective of study

The concept of sustainable development, as presented in the Brundtland Commission, has identified the pivotal role engineers have in designing for the future. In reflection to the issues presented on pavement materials and stabilisation, the objectives of this report are

i. determine the sustainability of HCTCRB and CTB with microcracking based on their economical and environmental performance through a Life Cycle approach
ii. determine the applicability of HCTCRB and CTB based on sustainability considerations
iii. provide recommendation to the engineering and road planning community on the improvement of the supply and production of HCTCRB to achieve improved sustainability

As is the case in Western Australia, this research assumes a scenario where the use of stabilisation for increased mechanical performance is required and crushed rock is the choice of material.

3 Assessing the sustainability of pavement materials

As discussed in Section 2, the application of life cycle assessments allows a more holistic and acceptable approach in engineering to determine the sustainability of materials. A pilot study based on Life Cycle Analysis (LCA) is therefore undertaken to determine the sustainability of HCTCRB and CTB, focusing on the environmental and economical implications of utilising either of the pavement materials.

The results are then used as performance indicators to perform a multi criteria analysis (MCA). The approach to the evaluation undertaken by this paper is shown in Figure 1 below.

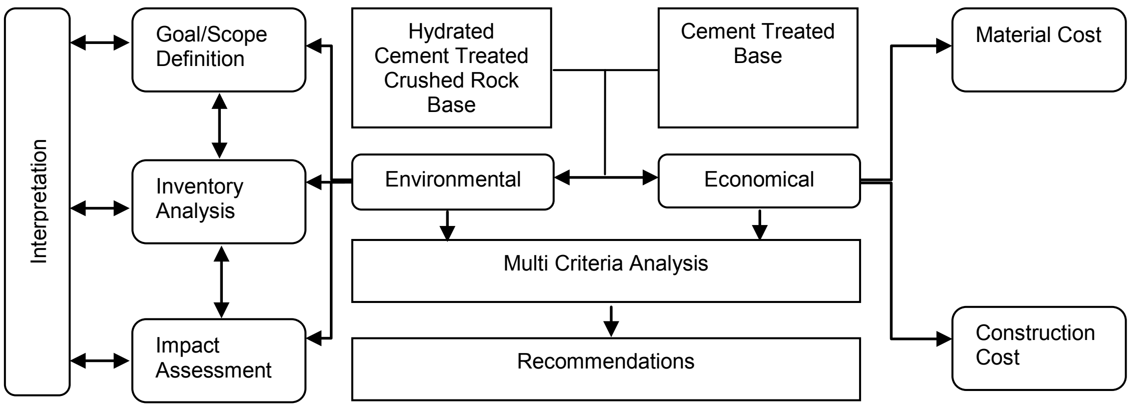

Figure 1. Methodology diagram of sustainability assessment

The LCA is conducted as per ISO 14040:1998 which involves a four phase approach, i.e. goal definition, inventory analysis, impact assessment and interpretation. Following that, an economical evaluation is determined from material, construction and maintenance cost incurred from the selection of the material. The two factors are then weighted and evaluated based on a multi criteria analysis.

3.1 System boundary

The system boundary to evaluate the sustainability performance between HCTCRB and CTB encompasses the critical life cycle stages that more apparently distinguishes the two materials namely, batching and construction. Figure 2 below showcases the system boundary and the major input parameters and relevant consequential impacts.

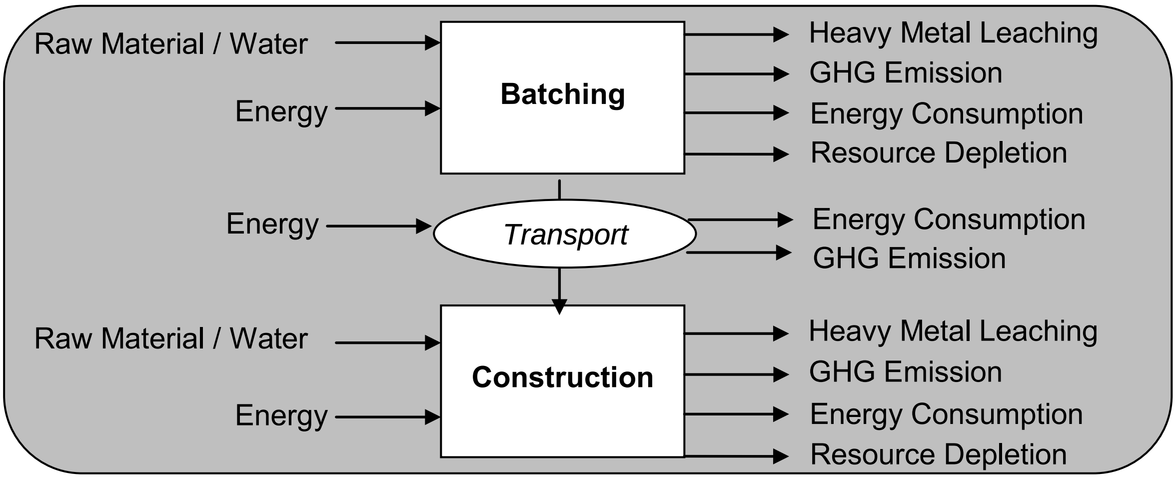

Figure 2. System boundary of assessment

The system boundary is narrowed to the two life cycle stages based on a gate to gate assessment as the loadings associated to sourcing of raw materials is reflected its consumption. Furthermore, stages beyond construction are discounted as pavement maintenance primarily involves resurfacing of the wearing surface. As for issues with frequency of maintenance, the design service life is standardised. Finally, the end of life of the pavements are not directly considered in this paper but are touched briefly in Section 4.2.5, where the potential for material recycling based on material selection is discussed.

The output environmental loadings have been also limited to in this pilot study based on existing studies of road construction life cycle analyses (Eskola et al. 2001) where the significant environmental impacts of road construction are atmospheric emissions, energy and resource depletion as well as chemical leaching. Similarly, Mroueh et al. (1999) has presented the key environmental loading of road construction based on expert groups and its corresponding scale. The top 5 items of the list and the equivalent generalised environmental loading assessed in this paper are shown in Table 1 below:

Table 1. Top 10 environmental loading of road construction

	Averaged score	*Generalised assessment*
Consumption of natural materials	10.0	Resource Depletion
Heavy metal to soil	9.4	Leaching
Fuel consumption	7.5	Energy Consumption
NO_x to atmosphere	7.0	GHG Emission
Energy consumption	6.9	Energy Consumption

As for the economical considerations, the associated material, transportation, storage and labour costs are accounted for throughout the two life cycle stages based on typical Bill of Quantities used for the construction of the designed pavements. As discussed in the following section, the service life of the pavements will be designed to similar number of years as to provide a better comparison, and hence will predominantly evaluate the initial capital cost, which reflects studies undertaken for stabilising techniques of road rehabilitation (Smith and Vorobieff 2005).

3.2 The functional unit

The sustainability evaluation applies the *functional unit* of one kilometre highway designed by applying standard pavement design procedures as per Main Roads design guideline (Butkus 2004) used in the Reid highway test sections. The choice of functional unit allows provisions of the effects of individual mechanical properties of the pavement materials to be evident governing material consumption. The cross section of selected pavement designs applied for evaluation are designed to allow optimum usage of the pavement materials as shown on Figure 3 below.

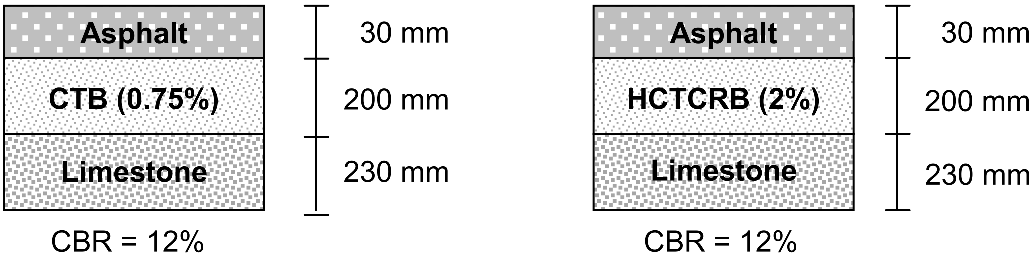

Figure 3. Pavement design

3.3 Production and construction process of pavement materials

This section details the batching and construction processes of the two different pavement materials. The investigation into the two life cycle stages determines the magnitude of input and output parameters of over each life cycle stage.

3.3.1 HCTCRB production and construction

Hydrated Cement Treated Crushed Rock Base is pre-mixed at quarries with typically 2% cement by using a loader. The premix is then passed through a pugmill at OMC. Following, the cement treated crush rock is then set in prepared forms and stockpiled at the quarry to allow for hydration to occur, with interim remixes to prevent setting up (Butkus 2004). After 24 - 48 days (Kelley 2009), the materials are then put through a mill to be disturbed and ultimately used as typical base course granular materials, i.e. the materials are trucked to site and spread on the subgrade with a grader.

3.3.2 CTB with microcracking production and construction

Cement treated base for this evaluation are assumed to be batched from plants. The production of CTB is similar to the typical batching processes of concrete where the required amount of cement which ranges from 0.5% to 10% is mixed with crushed rock and loaded onto concrete trucks. Once transported to site, CTB is poured onto the pavement and allowed to cure for 48 to 72 hours by sprinkling. Following, the microcracking process is undertaken by a minimum 12-ton vibratory roller travelling at 3.2km/h to 4.8km/h (PCA 2005). Upon satisfactory completion of microcracking, the base layer is further moist cured by sprinkling for an additional 72 hours.

3.4 Applied sustainable assessment

In view of the current practices in sustainability assessment, a pilot gate to gate life cycle inventory analysis based on the system boundary defined allows the fulfilment of objectives defined. References are sought based on life cycle inventory database where applicable. The following section discusses the investigation results and its interpretation thereof.

4 Assessment results

From the methodology set out in previous section, a pilot life cycle inventory analysis was undertaken based on the inputs and outputs as shown in Figure 2 for the pavements sections presented in Figure 3. The selected outputs of the pavement material batching and construction and its interpretation thereof is discussed as follow.

4.1 Pavement score

The pavement scores based on the LCI are presented in Table 2.

Table 2. Relative pavements loading percentage

	CTB			*HCTCRB*		
	Batch	Construct	**Life**	Batch	Construct	**Life**
Environmental (30%)						
Resource Depletion				+0.9%		**+0.9%**
Energy Consumption		+5.5%		+41.4%		**+16.7%**
GHG Emission		+4.6%	**+2.6%**	+2.6%		
Chemical Leaching		+	**+**	+		
Economical (70%)						
Direct Cost		+50.0%	**+6.1%**	+18.8%		
Sustainability score						
Relative %						**+3.0%**

The table shows the relative % difference of each assessment criterion over the two assessed life cycle stages, i.e. batching and construction and the whole life cycle. The values indicate the % higher sustainable loading imposed by the pavement at the corresponding life cycle stage.

Environmentally, HCTCRB incurs a higher loading in terms of resource depletion at +0.9% and energy consumption +16.7%, while CTB imposes a higher loading in terms of green house gas emission at +2.6% and chemical leaching into soil. Data for chemical leaching into soil are limited and site oriented, and hence is evaluated based on qualitative assessment.

The construction of CTB incurs an economical loading that is significantly higher than HCTCRB, i.e. at +50.0%. The direct cost of the CTB pavement is 13.7% higher than the HCTCRB pavement.

Overall, the weighted sustainable score shows the HCTCRB has a 3% better performance.

4.2 Interpretation and discussion of results

With the results presented in Section 4.1, this section provides interpretation of the data and provide discussions of the processes of the life cycle stages involved in the construction of stabilised pavements using HCTCRB and CTB. It discusses the environmental and economical loadings determined from the assessment while touching on issues of transport, storage, construction timeline and the afterlife of each pavement.

4.2.1 Environmental loading

Based on the system boundary and constraints as discussed in Section 3.1, it is inferred from the relative scores presented in Table 2 that the use of HCTCRB results in the depletion of resources. The total required tonnage of material for CTB is marginally lesser than HCTCRB as the cement forms mortar during hydration of the material. The remixing of HCTCRB in its batching also potentially results in a lost of cement material during to milling, which is a due cause for production improvements of HCTCRB as discussed in Section 5.

The construction of CTB layers require more passes by compactors to apply the microcracking, which results in an increase of energy consumption, cost and GHG emission. On the other hand, as the production HCTCRB requires additional remixing through a pugmill which is relatively more significant than the GHG emission caused by microcracking procedures. The total energy use through the life cycle of the pavements is therefore primarily governed by the energy use during batching. The energy requirements noted from the study allows the identification of improvement processes as discussed in Section 5.

The inventory analysis also noted that the predominant contributor to GHG emission is governed by transportation of materials and use of diesel fuelled plants. The additional construction works required for microcracking as discussed in Section 3.3.2 causes more GHG emission relative to the typical grading of HCTCRB. Moreover, the total tonnage deliverable by a dump truck is relatively higher than standard cement trucks, resulting in more trips required for the delivery of batched CTB. With transport being a major factor, further discussion is provided in Section 4.2.4 with regards to delivery distances.

Since the mixing of cement and the hydration process for the production of HCTCRB is conducted at the quarry, which is assumed to be in a more controlled environment, the chemical leaching of the cement paste to soil is better mitigated. In contrast, CTB is laid on site to be cured which may cause significant infiltration of toxicity into soil. Based on physical bonding between the aggregates, the moisture susceptibility of HCTCRB is believed to be higher compared to CTB which may potentially result in the "washing out" of cement into the soil. Further studies are required to understand this characteristic of both HCTCRB as discussed in Section 5.

Nonetheless, in overall the environmental loading is identified to be not majorly significant as shown in Table 2. As the assessment is conducted under specific constraints other factors such as distance, rehabilitation potential, material availability, delivery efficiency, etc will affect the outcome of the environmental loadings of the pavement materials.

4.2.2 Economical loading

As reviewed in Section 3.1, the economical loading is principally determined from the direct cost as maintenance of the base course layer is generally limited throughout the life cycle of the pavement. The cost associated to maintaining pavements to reach its full service life is largely associated to rehabilitation of the wearing surface.

In this assessment, the direct cost incurred by CTB is primarily due to the construction costs associated to site supervision and prolonged risk to wet weather with extended construction on site as a result of i) the extended construction time to allow CTB to cure on site before microcracking, ii) the additional compaction effort required to perform microcracking after CTB has set. The batching cost, although more significant for HCTCRB, is dominated by the cost of cement which does not outweigh the construction cost of CTB. For discussion purposes, the application of microcracking in Australia would also run a risk in over compaction, causing excessive cracking, due to the limited knowledge of the technology.

4.2.3 Multi criteria analysis and sensitivity analysis

The final sustainability assessment from the results has been undertaken using a multicriteria analysis of weighting the environmental loadings as per the rankings established in Table 1 and then weighing the economical and environmental loadings. The pilot study of this paper takes into account a 70% economical and 30% environmental weighting which results in an indicative result of HCTCRB being 3.0% more sustainably sound.

To further evaluate the weighting selected, a sensitivity analysis was undertaken. The analysis allows the estimation of a convergence point between the weighting to evaluate a scenario where CTB would be deemed as sustainable to HCTCRB. This would in turn verify the selected economical to environmental rating as well as to assist in identifying scenarios where CTB would be a more plausible choice. Figure 4 below shows the sensitivity analysis, where the convergence point is estimated to be at the weighting of 59% economical and 41% environmental under the constraints studied.

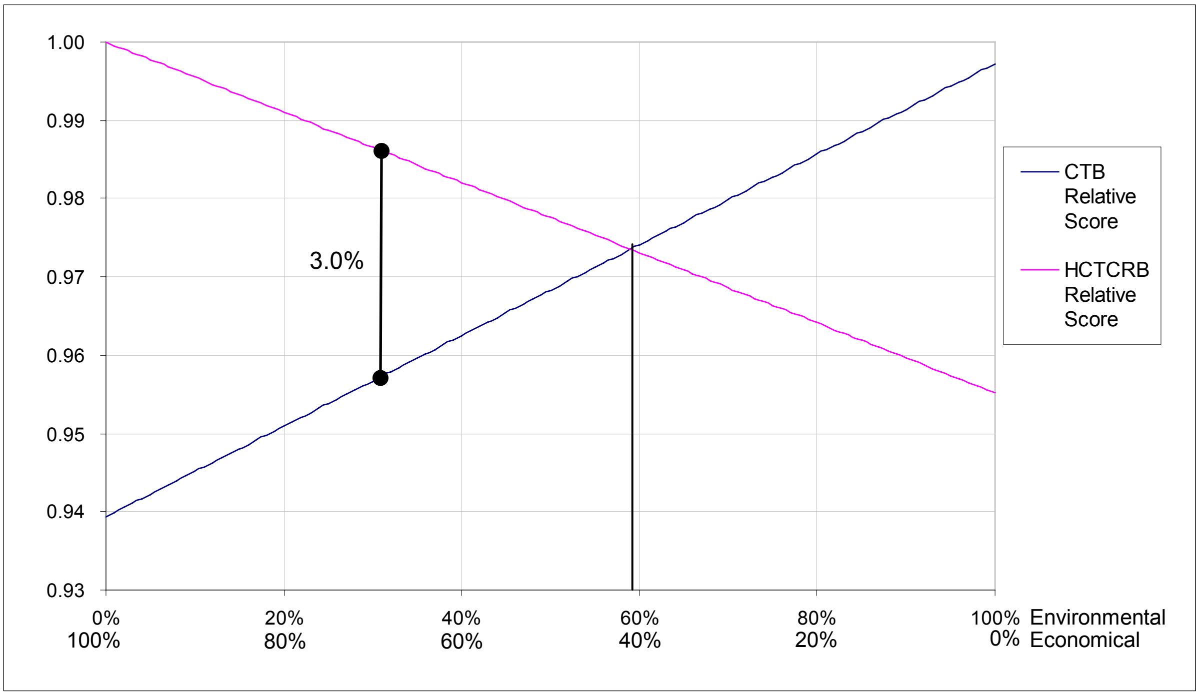

Figure 4. Sensitivity analysis

It is therefore inferred from the multicriteria analysis that HCTCRB is a more sustainable solution in stabilising pavement. The sensitivity analysis also shows that HCTCRB would be in general circumstances a more sustainable solution with the applied weighting.

4.2.4 Transportation and in-situ mixes

The delivery distance of the materials reflects as a significant factor in the determination of the sustainability assessment. With different characteristic densities of CTB and HCTCRB, the total number of trips varies resulting in more GHG gas emissions and fuel consumption for HCTCRB at the batching life cycle stage.

However, the delivery of batched CTB is significantly limited by the setting period, which hence limits the use of CTB within a confined proximity where a batching plant is available. In-situ CTB mixes can be used as an alternative, but would require an increase in thickness and more advanced equipments as provisions for quality assurance.

4.2.5 Afterlife rehabilitation potential

The rehabilitation potential of both pavements is deemed highly plausible with existing CTB pavements requiring two times more reclamation effort compared to HCTCRB (Austroads 2007). However, rehabilitation of pavements through Full Depth Reclamation (FDR) techniques which grinds and mixes existing pavements with cement to form new layers of CTB has been identified in numerous literatures as being a sustainable solution. Under these circumstances, an in-situ recycled CTB base course layer would therefore greatly outweigh the use of HCTCRB.

5 Conclusion and recommendation

In summary, this paper has discussed the application of life cycle analysis combined with a multi criteria analysis to act as a pilot study in determining the sustainability of engineering modified base course materials, i.e. Hydrated Cement Treated Crushed Rock Base (HCTCRB) and Cement Treated Base (CTB) with microcracking.

The use of HCTCRB in pavements consumes more material and energy. However a pavement with CTB will result in higher green house gas emissions, increased potential for chemical leaching, and higher costs. Through a multicriteria analysis, and considering the constraints and circumstances studied in this paper, HCTCRB has been identified as being a more sustainable solution compared to CTB. Nonetheless, delivery distance, rehabilitation potential and availability of resources remain key factors in determining the right base type for road construction.

More importantly, the following assessment has been successful in identifying potential improvements in the production and construction of road pavement with HCTCRB and CTB with microcracking as follow:

i. Improve recovery of cement lost in the remixing processes of HCTCRB
ii. Investigate and improve remixing sequences of HCTCRB to reduce total energy used for the production
iii. Ensure hydration of HCTCRB at quarries are duly controlled to prevent chemical leaching
iv. Understand the moisture susceptibility characteristics of HCTCRB to determine potential chemical leaching
v. Utilise CTB with microcracking as part of Full Depth Reclamation road rehabilitation.
vi. Improve CTB and microcracking knowledge base in Western Australia to optimise its use
vii. Plan CTB pavements to minimise construction cost.

As a final point, stabilisation of base course materials to improve otherwise mechanically inadequate materials for heavily trafficked pavements are essential for sustainable road construction, nevertheless, more studies are required to understand the characteristics of HCTCRB and CTB in order to optimise the sustainability of the transport industry.

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MECHANISTIC CLASSIFICATION OF CEMENT TREATED BASE IN WESTERN AUSTRALIA

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ABSTRACT

In the past decade alone, the BITRE has indicated an increase of 40% in road users, escalating demands for quality pavements to service unprecedented traffic conditions. An abundance of crushed rocks are available in Western Australia but do not meet strength requirements for road construction. However, cement treatment of crushed rocks, forming Cement Treated Crushed Rocks (CTCR), improves the mechanical properties of the material, allowing wider application. In order to streamline the mix design of CTCR, the classification of its behaviour is pivotal. Austroad classifies cement treated pavement materials as either being modified or bound based on its Unconfined Compressive Strength (UCS) and performance attributes. Bound materials are defined by its susceptibility to fatigue failure which, in the mechanistic-empirical design for flexible pavements, is dictated by the flexural modulus. However, in the study of damage mechanics, fatigue life is suggested to be an accumulation of micro-scale damage in lieu of dependency to ultimate stresses. Strain dependent damage functions are used phenomologically to explain the evolution of fatigue for various engineering materials. This paper therefore investigates a theoretical relationship between strain and fatigue life prediction supported by a laboratory investigation on the use of UCS for classification. This is achieved by providing regression analysis with strain parameters used in fatigue life prediction. The Indirect Tensile Strength (ITS) test is also employed to this end. It is observed that strain at onset of micro-cracking coalescence (ε_{30}) is determined to be independent of test type undertaken and potentially capable of acting as a more superior blanket classification for cemented materials.

INTRODUCTION

In the past decade alone, the BITRE (2009) has indicated an increase of 40% in road users, escalating demands for quality pavements to service unprecedented traffic conditions. Western Australia has an abundance of crushed rocks but nevertheless do not necessarily meet performance requirements for heavy traffic loads. When cement is treated to crushed rocks an interlocking matrix between the aggregates is created, forming Cement Treated Crushed Rocks (CTCR). This allows the otherwise mechanically unsuitable material to be used in road construction.

Nonetheless, Main Roads Western Australia (MRWA) limits the use of CTCR due to its tendency to undergo shrinkage and fatigue failure. With the lack of interest, very limited studies of CTCR exist in Western Australia. This paper presents a theoretical development and laboratory investigation to examine the applicability of the existing Austroads recommended classifications of cement stabilisation in WA.

CLASSIFICATION OF CEMENT TREATED PAVEMENT MATERIALS

Pavement materials are traditionally recognised as either exhibiting unbound or bound behaviours. It is this alteration of mechanical behaviour that has formed the basis of CTCR classification. The degree of binding is designated as either being “modified” or “stabilised”.

Modification is a result of applying small amounts of cementitious binders, typically lime or chemical binders (Austroads 2006; Jenkins 2002). It is not intended to improve mechanical (strength) performance, but instead aims to reduce plasticity and moisture susceptibility (Austroads 2006), thus minimising rutting potential and increasing durability.

Stabilisation is the addition of cementitious binders so that improvements in mechanical performance (strength) are achieved. The level of stabilisation is also differentiated to two broad categories, i.e. lightly stabilised and stabilised (Austroads 2006).

In pavement mix design, the distinction of the mechanical behaviour under varying cement content is particularly important to the industry as it dictates the cost, serviceability and design life of a road. However, there is no exact distinguishing factor between each category because of overlapping traits (Jenkins 2002).

AUSTROADS CLASSIFICATION CRITERIA

The identification of classification points has generally been empirical in nature and primarily based on experience. Austroads Pavement Design Guideline classifies cemented materials based on its unconfined compressive strength (UCS) and performance attributes. The recent classification guideline, adapted from Austroads (2006), is presented in Tab 1 below.

Tab 1: Austroad (2006) Classification of Cemented Materials

Classification	Testing Criteria	Performance Attributes
Modified[1]	0.7 MPa < UCS[2] < 1.5 MPa	Flexible pavement subject to shear failure within pavement layers and/or subgrade deformation.
Lightly Bound (Stabilised)	1.5 MPa < UCS[2] < 3 MPa	Lightly bound pavement which may be subject to tensile fatigue or subgrade deformation.
Bound (Stabilised)	UCS[2] > 3 MPa	Bound pavement which may be subject to tensile fatigue cracking and transverse dry shrinkage cracking.

[1] Modification is typically achieved by addition of lime, polymer or chemical binders.
[2] Values determined from test specimens stabilised with GP cement and prepared using Standard compactive effort, normal curing for a minimum 28 days and 4 hour soak conditioning.

UCS is widely accepted as the classification criterion for cemented materials within the transportation industry because of its relative ease and speed to undertake (Vorobieff 2002). The standardisation of procedures (AS5101.4) and the availability of testing frames or moulding equipment in typical geotechnical laboratories are added advantages. Nonetheless, variations of sample preparation methods and limiting UCS ranges exist across the different states in Australia and internationally (Vorobieff 2002). Although they are in general agreement of the Austroads recommendations, the variability of the parameters are a clear reflection on the uncertainty in defining CTCR behaviour.

The Unconfined Compressive Strength (UCS) provides an indicative measure of the normal stress and cohesive shear strength of the cement matrix, which has been believed to express the degree of binding achieved. It does not provide any input to design, but is primarily used for classification.

In retrospect to its predecessors, the current classification provides a performance description of class, which highlights the failure mechanism of the material. It supersedes the previous approach which was based on UCS and design flexural modulus. The flexural modulus has since been removed as a classifying criterion and instead reflected as a parameter in calculating the fatigue life of bound materials shown below.

$$N = RF\left[\frac{11300/_{E^{0.804}} + 191}{\mu\varepsilon}\right]^{12}$$

where, E = flexural modulus
$\mu\varepsilon$ = load-induced strain
RF = reliability factor for cemented materials fatigue

This paper is in agreement with the performance attributes as a classification methodology as seen in Tab 1, in particular the postulate that tensile fatigue failure is the defining criterion for the identification of stabilisation. Nevertheless, the validity of the testing criteria is investigated. A phenomological theory development and preliminary laboratory investigation is undertaken.

THEORETICAL DEVELOPMENT OF FATIGUE LIFE PREDICTION: CLASSICAL MECHANICAL PARAMETERS VS. DAMAGE EVOLUTION

Fracture and damage mechanics for Cement Treated Crushed Rocks

In the study of continuum damage mechanics, fatigue is characterised by the accumulation of damage on a micro-scale. The quasi-brittle nature of CTCR causes the material to undergo damage in the form of nucleation of voids formed from the coalescence of microcracks (Balbo and Cintra 1996). This fracture process involves creation of new surfaces in the material, a phenomenon much better described by energy principles than by classical mechanics (Lee et al 1997, Alliche and Francois 1990).

Moreover, based on the strain equivalence principle (Lemaitre 1985; Lee et al 1997, Sirdoff 1981), the concept of damage can be represented by introducing a damage function. It is derived on the basis that the virgin material and its continuum model must contain equal strain energies when subjected to similar global displacements. It is represented based on the degradation of elastic modulus which results in a lowered capacity to store strain energy, i.e.

$$\bar{E} = E_0(1 - D)$$

The strain behaviour of a damaged material may therefore be represented as follow,

$$\varepsilon = \frac{\sigma}{\bar{E}} = \frac{\sigma}{E_0(1 - D)} = \frac{\bar{\sigma}}{E}$$

As shown in equation above, prior to the onset of damage, i.e. $D = D_o = 0$, the linear elastic postulate is observed. This is attributed to the weak interfacial transition zone and existing microcracks. A model of the damage evolution is described in fracture mechanics as shown in the Fig 1 below adapted from Gdoutos (2005).

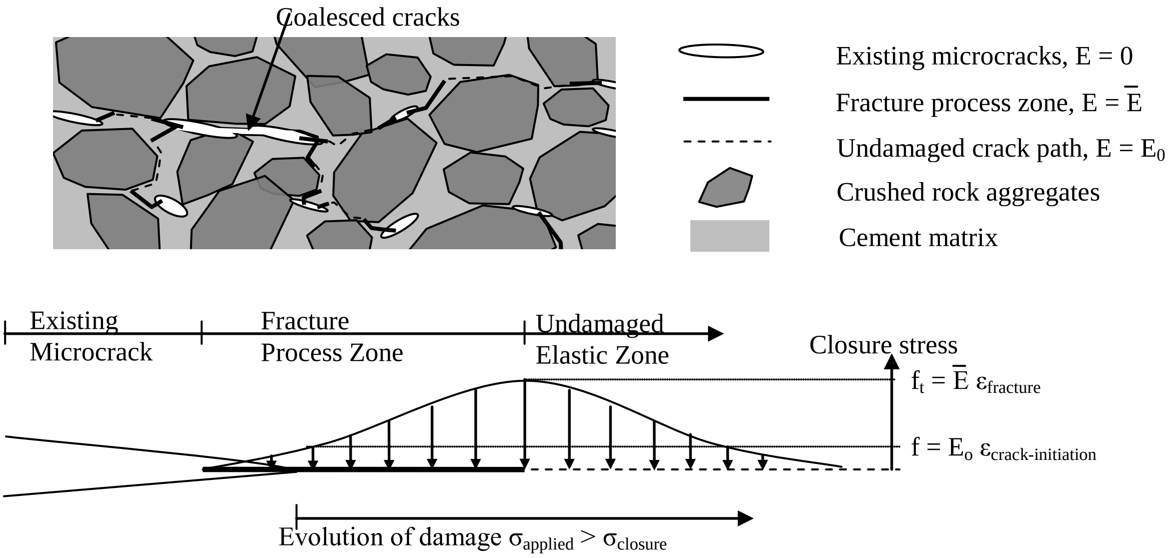

Fig. 1: Micro-cracking coalescence model

The development of crack length is as shown in the Fig 1 and the principles of strain equivalence shows a close relationship between fatigue and strain. This is discussed in the following subsections.

The fatigue and strain relationship

The evolution of crack length in ductile materials such as steel are defined in classic fracture mechanics by the Paris Law, $\frac{da}{dN} = C(\Delta K)^m$ where, ΔK = stress amplitude factor, and C and m are material constants. However, as described in the preceding section, effective stress of damaged quasi-brittle materials can be represented by strain. The phenomenology presented by Paris has thus been recognised as the *power law* for the design of flexible pavements (), with fatigue life explained as a logarithmic function of strains. Studies of cement treated materials (Kaplan 1963; Alliche and François 1992; Karihaloo and Fu 1990; AARB 2010) also demonstrated the imminent relationship between fatigue and strain.

Furthermore, the strain measured at the termination of elastic linearity, characterised as the initiation of damage or micro-crack, is observed as a material constant (Kaplan 1963, Karihaloo and Fu 1990, AARB 2010). This is in contrast to ultimate stresses and strain which are dependent of the type of test undertaken (Karihaloo and Fu 1990).

Nevertheless, the methodology adopted by Kaplan (1963) in identifying the onset of cracking defined as the cease in proportionality requires sound judgement and precise measurement of strain. It is a difficult procedure to repeat. Further development by Williams (1986) however shown that the proportionality of stress strain relationships ceases generally at 30% of the maximum load, a simplified assumption that is adopted by this paper.

Based on the principals discussed in preceding sections, the evolution of damage can thus be explained by a strain dependent function from the onset of damage to failure, i.e.

$$\frac{dD}{dN} = \int_{\varepsilon_{crack\ -initiation}}^{\varepsilon_{failure}} f(\varepsilon)$$

Where $\varepsilon_{\text{crack-initiation}}$ = initial strain exhibited for micro-crack coalescence (when applied force > 0.3 P)
$\varepsilon_{\text{failure}}$ = max tensile strain

The development of a fatigue model is communicated in future publication by the authors. It is the interest of this paper to investigate the compatibility of UCS to material classification and investigate instead a plausible categorisation using strain at the initiation of damage.

LABORATORY REGIME

Given the strong dependence of fatigue life prediction to strain, an investigation between the relationship of the existing classification criteria and strain is undertaken. A regression analysis is investigated between unconfined compressive strength (UCS) and indirect tensile strength (ITS) against strain at the crack initiation or onset of damage, defined as the strain at 30% of maximum load (ε_{30}), and strain at failure, measured at 95% of maximum load (ε_{95}).

Two standardised tests are undertaken, i.e. the Unconfined Compressive Strength Test to AS5101.4 (AS 2008) and Indirect Tensile Strength Test to AS1012.10 (AS 2000). Specimens are prepared as a function of cement content, ranging from 2%, to 5% in 1% increments, compacted to 100% modified MDD, and cured 28 days prior to testing. Specimens are cured in a controlled humidity environment and soaked 4 hours prior to testing. A minimum of 3 specimens are prepared for each mix design.

DISCUSSIONS LABORATORY RESULTS

UCS and ITS results against cement content, ε_{30} against cement content and ε_{95} against cement content are plotted as follow.

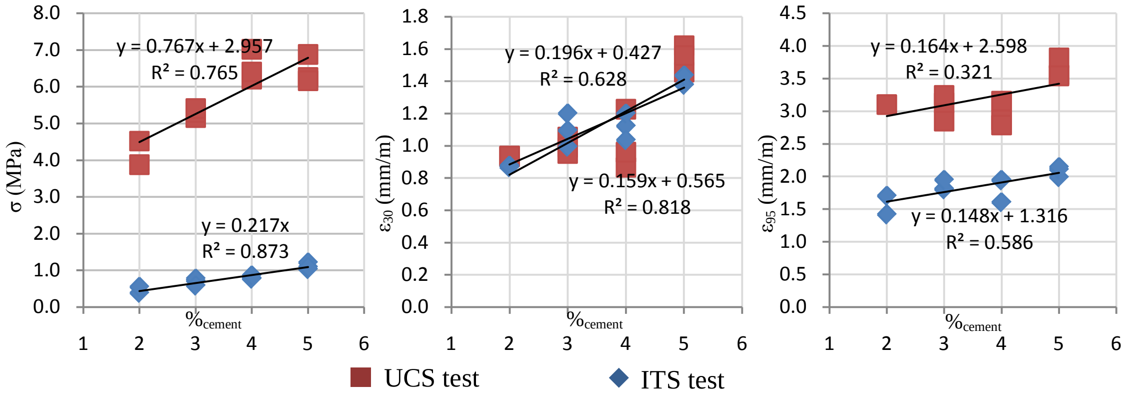

Fig 2: Chart A: Stress vs. % cement; Chart B: ε_{30}vs. %cement; Chart C: ε_{95}vs. % cement

Chart A indicates a statistically strong linear relationship between cement content and the two sets of stresses. Chart B shows a strong linear regression relationship between ε_{30} and % cement with both tests showing very similar strain values. In contrast, strain near failure as shown in Chart C, defined as strain under 95% of maximum loading (ε_{95}), exhibits a statistically weak relationship to % cement.

Based on the recommendation by Austroads (refer Tab 1), it is implied from the lab results that Western Australian CTCR are "heavily bound" even at very low cement content. This categorisation is inconsistent with the physical state of the material during the laboratory testing which exhibited very weak cohesion between aggregates and the matrix. The UCS values on a linear regression also do not intersect with the origin. These observations points to other latent factors, primarily the compressive stresses being transferred through the aggregates and cement paste during UCS tests. It does not relate to the damage theory developed within this paper that fatigue life is a function of micro-crack coalescence.

In retrospect, the failure criterion of the tested Western Australian CTCR is possibly dictated by fatigue. As reported in MRWA's report (Buktus 2004), CTCR with cement content as low as 1.0% had been deemed susceptible to some form of fatigue failure and hence ruled out to be used for pavement structures in WA.

More importantly, the laboratory investigation has showed that strain at the onset of cracking is a material constant, independent of type of test undertaken. This is in contrast to the strain at failure where a consistent gap is noted between the laboratory results. Relating to the principle of strain equivalence introduced earlier in the paper, the damage threshold for CTCR under UCS loading is different to ITS loading, again supporting the inconsistency of defining fatigue damage threshold with ultimate stresses.

RECOMMENDATION AND CONCLUSION

The current classification of CTCR by ultimate stresses is suggested to be incompatible with principles of fracture and damage mechanics, which forms the postulate of fatigue life prediction. Instead, it is proposed that the strain prior to the onset of fatigue damage, typically at 30% of maximum load (denoted as $\boldsymbol{\varepsilon_{30}}$**)** potentially be used as a blanket classification criteria for CTCR. It exhibits a statistically strong relationship with stresses and mix design (% cement content) and acts independently from the type of test undertaken.

Based on the two tests undertaken , i.e. the Unconfined Compressive Strength (UCS) Test and Indirect Tensile Strength (ITS) Test, the strain at crack initiation for Western Australian CTCR, can be determined from the equation:

$$\varepsilon_{30} = 0.16 \times \%\, cement\ content + 0.565$$

Since, it is well accepted and recognised that UCS is an economical and speedy testing regime, a further advantage to the proposed approach is that a current practices can be retained. The difference being a different value is taken from the investigation.

This paper has presented a simplified phenomological assertion of the relationship between strain and fatigue life based on fracture and damage mechanics. It is established that strain prior to the onset of failure is independent of type of test undertaken. As strain is an input into fatigue life prediction, its application as a classification criterion would also further benefit the design process.

Further testing is required to establish a suitable ε_{30} range based on a fatigue damage model for CTCR, which will allow a more concise characterisation of the material. Further testing of other materials is also required to improve the validity of the proposed relationship.

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BRIEF BIOGRAPHY OF PRESENTER

Bay Yeo is a doctorate candidate with Curtin University of Technology where extensive research into the mechanistic design of pavements is undertaken through the university’s Pavement Research Centre. His research is focused primarily on the characterisation of cement treated crushed rocks. He is also currently a practicing civil and structural engineer with Sinclair Knight Merz.

FLEXURAL BEHAVIOUR OF CEMENT TREATED CRUSHED ROCK UNDER STATIC AND DYNAMIC LOADS

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ABSTRACT

Fatigue life of cement treated crushed rocks is one of the most complex concept in pavement engineering. From the testing regime, structural model and finally to the measurement criteria, there are a multitude of variables that can affect the fatigue response of the material. In asphalt testing, the four point fatigue bending test has gained general acceptance as a material characterisation tool capable to predict the fatigue life of asphaltic pavement layers. Its applicability to cemented base course materials is however arguably uncertain and widely untested. A laboratory regime which includes flexural beam testing and the four point beam fatigue test is thus undertaken. The flexural beam test investigates the flexural behaviour of cement treated crushed rocks under static loads, while the four point beam fatigue test seeks to extend the results into the fourth dimension – time. The four point beam fatigue test is completed by varying the mix design. The paper presents the challenges and results achieved from the test. It is recommended that the current limits that define the bound behaviour in Western Australia be reviewed against advanced understanding of flexural behaviour of cement treated base course.

KEY WORDS

cement stabilisation, base course, pavement, flexural behaviour, fatigue

INTRODUCTION OF STABILISED PAVEMENT DESIGN

Dating back to the first recorded stone-paved streets of Ur (modern day Iraq) in 4000 B.C., road pavements have evolved into the primary terrestrial network for freight and commuters. They are now recognised as an icon of men's ingenuity and a symbol of the modern built environment. However, despite the maturity of pavement technology, the design philosophy of pavements in Western Australia is predominantly built primarily on empirical data and historical observations, a method deemed unrepresentative of true pavement mechanical responses. Regardless, it is continued to be accepted by engineering practitioners in general to be an adequate approach for road designs of unbound pavements. This is due largely in part to the design methodology's ability to capture the inherent inconsistency of pavement materials, the multitude of different loading conditions and patterns, variability in construction and many other factors which make characterisation of pavements based on mechanical principles a challenging feat.

Nevertheless, when bound materials are to be used, the application of empirical design method is limited. The material is deemed to behave as an isotropic material with more consistent load response behaviour. Austroads (2008a) recommends the use of the mechanistic-empirical design procedures which calculates the life of a bound pavement as a function of its flexural properties. The two flexural properties used within this calculation are the limiting flexural strain and flexural modulus. The flexural strain is dependent on the applied load, modulus and thickness of the cemented pavement layer and is computed based on linear elastic theory with vertical modulus and Poisson ratio as input parameters.

CRUSHED ROCKS AND STABILISED PAVEMENTS IN WESTERN AUSTRALIA

Crushed rocks have been regarded in the past as a high quality material for use in Western Australia where natural occurring aggregates are not available. The material is produced from rock formations available in a multitude of locations and thus relieves the need to source high quality natural aggregates. However, during its use in the construction of Kwinana Freeway in 1992 (Buktus 2003), excessive deflection was measured as early as 4 years after completion of construction. Stabilisation was thus sought out as an option for improved performance.

Nonetheless, based on current Main Roads Western Australia's pavement design guideline, Engineering Road Note 9 (MRWA 2010), all gazetted public roads under the care of Main Roads Western Australia (MRWA) are not to be built with stabilised (bound) materials. Similarly, when smaller contents of cementitious binders are used and modified (semi-bound) materials are formed, no reduction in thickness of the pavement is allowed (MRWA 2010).

The lack of confidence towards the use of cementitious stabilisation on crushed rocks in Western Australia stems from failed past experiences. Stabilised crushed rocks were trialled in the past exhibited severe shrinkage and fatigue cracking which resulted in the cementitiously bound layer undergoing severe distress well before its expected service life. Since then, cementitiously bound pavements have been regarded as a bane for pavements.

With unsuccessful attempts in the use of bound pavements, modified materials have been investigated through the construction of trial pavements along Reid Highway in 1996 and 2003 respectively (Buktus 2003). Modified materials involve the addition of lower cement contents to granular materials without significant gain in tensile strength. The empirical design methodology was employed for the trial pavements on the basis that the modified materials behaved as unbound granular material. Various combinations of cementitious binders and granular materials were trialled.

The trial sections displayed varying degrees of results which can be found within MRWA (2010). In general, modified materials were deemed to not undergo early distress due to cracking. Nevertheless, the permanency of binders was the most significant issue with the use of modified material. This issue of permanency will be covered in other publications.

Until November 1997, Austroads' (known as NAASRA at the time) design methodology for cemented pavements was based on an empirical equation which calculates the design life of cemented pavements as a function of the modulus of the material (Austroads 2010). It was later discovered by Jameson et al. (1992) and Litwinzwicz (1986) that there was reasonable uncertainty with the relationship between modulus and pavement life. Thus, the basis of the current design methodology of design life as discussed in the previous section is born. Recent tests undertaken by ARRB (Austroads 2010) also indicated that different aggregate types result in different flexural responses.

With the predominant experience of MRWA in stabilised pavements in the 1970s to 1990s, the primary issue with unsuccessful application were most likely an issue with the design methodology applied and the lack of understanding of the material itself. Furthermore, materials specific to Western Australia will likely perform differently. This paper therefore investigates the flexural behaviour of cemented crushed rock in Western Australia.

UNCONFINED COMPRESSIVE STRENGTH AND STABILISED MATERIAL CLASSIFICATION

The unconfined compressive strength (UCS) test has been used locally and internationally for the classification of cementitiously stabilised pavements as the UCS is deemed representative of the degree of binding achieved by stabilised pavements. MRWA (2010) defines a materials exhibiting bound behaviour when any of the following criteria are met:

a) 7-day unconfined compressive strength (UCS) of the material exceeds 1.0 MPa;
b) 28-day UCS of the material exceeds 1.5 MPa; or
c) vertical modulus of the material exceeds 1500 MPa.

When the above criteria above are not met, the materials are considered as modified. Similarly, Austroads (2006) classifies cementitious stabilisation based on UCS, where UCS values of 0.7 to 1.5 MPa are deemed "modified" and UCS values more than 1.5 MPa are deemed "stabilised". However, the classification also considers descriptive attributes of each stabilisation categories. Stabilised pavements are described as susceptible to flexural fatigue, a point strongly relevant to this paper.

FLEXURAL PROPERTIES AND FATIGUE

Flexural properties are considered characteristic of the structural system of flexible pavements since flexible pavements essentially "flex" under traffic loadings. They are generally accepted as the most representative test for assessing the tensile capacity of concrete pavements (Griffiths and Thom 2007). In Australia, Austroads (2008b) have undertaken an extensive study to compare flexural properties of materials to characterise the behaviour of cemented base course under repeated loadings. In his report, the flexural beam test and the conventional cylindrical indirect tensile strength test were compared against readings from the Accelerated Loading Facility (ALF) located in Dandenong, Victoria. The flexural beam test was found to exhibit comparatively closer readings to in-service conditions shown by ALF (Austroads 2008b). Flexural properties of materials are derived specifically from the flexural beam test and explain the stress-strain relationship of beam specimens undergoing bending stresses.

Amongst the typically measured parameters are the modulus of rupture (also known as the flexural strength), i.e. the maximum allowable force applied onto the beam prior to rupture, the tensile strains at bottom fibres and the flexural modulus, i.e. the rate of change of tensile strain under stress. Beyond that, the fatigue response of a specimen can be derived by applying repeated loadings where laboratory results are extrapolated to predict in service conditions.

The modulus of rupture of a material is a unique measurement of a specimen's bending capacity and does not represent the true tensile capacity of a material. It is influenced by the thickness of the material as shown in Figure 1 below (Griffith and Thom 2007).

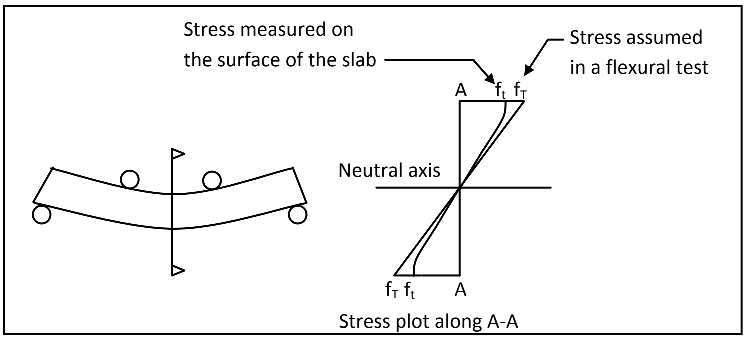

Figure 1: Observed tensile strength and true tensile strength

For flexural fatigue, the degree of fatigue susceptibility is represented by the Load Damage Exponent (LDE), i.e. the exponent of the relationship between the number of repeated loads and the pavement's mechanical property as shown in Equation 1.

$$N = (k/b)^{LDE} \qquad \text{Equation 1.}$$

where k = constant
b = mechanical property

As stiffness of a specimen is gained from the development of cement matrices, specimens develop a higher degree of brittleness, thus a lowered LDE. The phenomenon occurs when the elastic threshold is exceeded through applied stresses or strains where plastic strain manifested as minuscule cracks accumulate at the tensile fibres (Kaplan 1963; Balbo and Cintra 1996) ultimately reducing the section modulus, deteriorating the specimen till failure.

This paper investigates the flexural behaviour of cement treated crushed rocks using the four point bending test available at Curtin University's Pavement Research Centre. Static and dynamic tests were undertaken to ascertain the rupture modulus, fatigue response, flexural modulus and flexural strain. Furthermore, the Unconfined Compressive Strength and Indirect Tensile Strength Tests were performed to correlate the data attained from the flexural testing.

SPECIMEN PREPARATION AND TESTING REGIME

Crushed rocks sourced from Holcim quarry in Western Australia which meets engineering properties specified in MRWA Specifications 501 (MRWA 2011) for aggregates are used for this experiment. These properties are not presented in this paper and can be found in MRWA (2011). General Purpose (GP) cement is used for stabilisation. Specimens are compacted to modified dry density 2.26 t/m^3 at OMC are prepared for cement contents 1% through to 5%.

Table 1 below shows the optimum moisture content of each mix design. The specimens are then wrapped cured for 28 days in an encasement with constant moisture content and temperature.

Table 1 – Specimen Preparation Parameters

Cement Content (%)	1	2	3	4	5	6
Moisture Content (%)	5.75	6.00	6.25	6.50	6.75	7.00

Flexural Testing

The execution of flexural beam tests can be undertaken with either a 3-point or a 4-point bending test setup. In Australia and common to concrete specimens, asphaltic materials typically use the four-point bending test setup in fatigue related testing regimes. The four point bending test has the added advantage that it provides a uniform stress distribution between the two loading points. In contrast, the three point bending setup has the maximum stress concentrated locally below the loading point exposing only very limited area of the specimen to the maximum loads. This allows a wider region of controlled strain and stress applied onto the specimen. The four point bending setups used in this experiment are shown in Figure 2 below.

Figure 2: Four Point Bending Test Setup – Static and Dynamic

Further to the test setup, when applying repeated loadings for fatigue testing, the test can be run as either stress or strain controlled loading patterns. For asphaltic materials, stress controlled tests have been noted to emulate behaviour of more than 125 mm thick asphalt layers while strain controlled tests have shown better correlation to asphalt layers thinner than 125 mm (Austroads 2010). Such observations are however unconfirmed for cement treated materials.

In this research, beam specimens measuring 400mm (L) x 105mm (W) x 105mm (D) are prepared. The Four Point Bending Test Fixture for UTM fitted to the IPC Global Repeated Triaxial Load Test Apparatus was used for repeated load test and Instron 5500R Beam Testing Machine for static beam test. Both equipments are available in Curtin University’s Civil Engineering Labs. The Static beam test was undertaken to AS1012.11 (Standards Australia 2000), while the fatigue beam test was undertaken in strain controlled mode by applying haversine pulses at 3000ms. The fatigue tests are terminated after 24 hours, i.e. 28800 cycles, with the first 200 cycles allowed for initial conditioning.

Unconfined Compressive Strength and Indirect Tensile Strength Test

Unconfined Compressive Strength test was undertaken to AS5101.4 (Standards Australia 2008) with specimens measuring 105mm (D) x 101.5mm (H) and the Indirect Tensile Strength Test were specimens measuring 101.5mm (D) x 75mm (H). The STX-300 Triaxial Testing System is used to undertake the test. Specimens are soaked for 4 hours and cured for 28 day prior to test. A constant loading rate of 1mm/minute is adopted.

LABORATORY RESULTS AND DISCUSSIONS

The results for tests undertaken as part of this research are summarised in Table 2 below.

Table 2 –Experimental Results

Cement Content (%)	1	2	3	4	5
Unconfined Compressive Strength, σ_c (MPa)	3.85	4.51	6.12	7.09	7.43
Indirect Tensile Strength, σ_t (MPa)	0.22	0.46	0.69	0.80	1.11
Modulus of Rupture, σ_f (MPa)	0.50	0.81	1.39	2.05	2.20
Static Flexural Modulus, E_s (MPa)	2578	4035	5957	8362	9917
Dynamic Flexural Modulus, E_d (MPa)	1296	2393	2628	3823	8951
Initial Dynamic Flexural Stiffness, S_{do} (MPa)	1225	2241	2456	3504	8329
Dynamic Flex. Stiffness at Termination, S_{de} (MPa)	849	1357	1294	2238	4789
Fatigue Response, N	$\left(\frac{1488}{S}\right)^{27}$	$\left(\frac{2468}{S}\right)^{23}$	$\left(\frac{2766}{S}\right)^{16}$	$\left(\frac{5408}{S}\right)^{12}$	$\left(\frac{21723}{S}\right)^{7}$

Modulus of Rupture and Tensile Strength

As discussed in previous sections, the modulus of rupture of a material is dependent on the geometry of the specimen. When compared against indirect tensile strength, the modulus of rupture is different, thus substantiating the influence of specimen geometry and type to the measure of tensile capacity as seen in Figure 3. The flexural modulus of cemented crushed rocks in WA is shown to be typically double the magnitude of indirect tensile strength when a 100mm x 100mm specimen is used. This agrees with a similar trend in the work of Austroads (2008b), where the flexural modulus is typically in an order of magnitude of 40% higher than the indirect tensile strength test, where specimens measuring 200mm x 200mm were used. The effects of specimen size to the modulus of rupture are not a new field of study, however, its implication to flexural behaviour of pavements is something worth considering, and especially its implication to fatigue characterisation of cementitiously bound material.

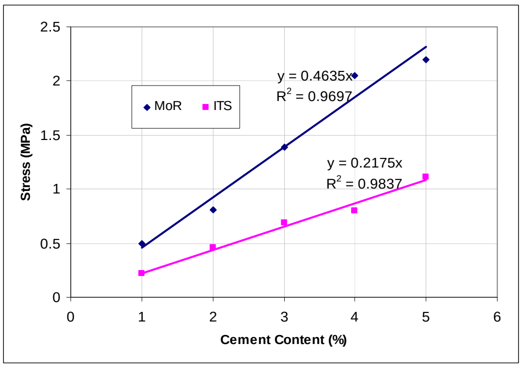

Figure 3: Cement content vs. Modulus or Rupture (MoR) and Indirect Tensile Strength (ITS)

Flexural Fatigue and Material Classification

The fatigue response of the specimens is discussed based on two observations, (1) the Load Damage Exponent (LDE) and (2) the signs of fatigue life phases. A constant rise in flexural modulus is evident through the addition of cement content into the mix as seen in Figure 4 below. This corresponds with an increase of susceptibility of the material to fail by fatigue which is inferred from the increasing LDE. A distinct linear relationship exists between LDE. Figure 4 below also shows the linear relationship between cement content and LDE.

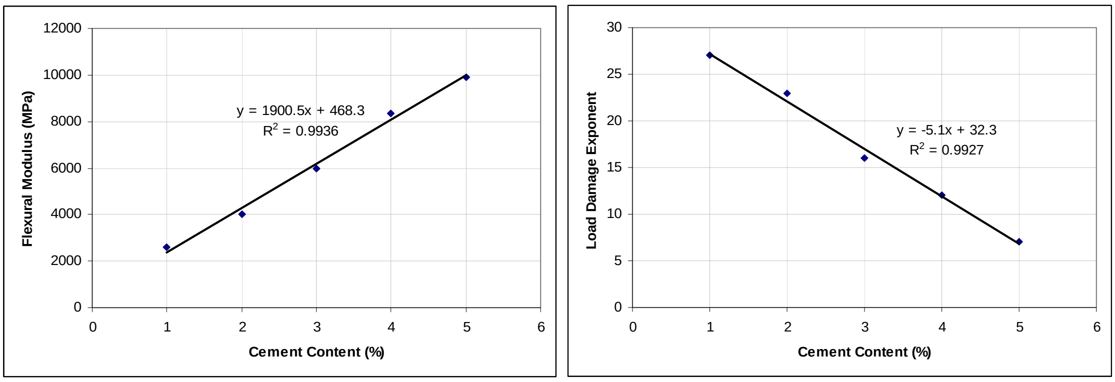

Figure 4: Cement content vs. static flexural modulus and Load Damage Exponent (LDE)

These points towards a development of fatigue susceptibility with cement content when 3 - 4% of cement is added to the material where the LDE ranges from 12 to 15. An LDE of 12 corresponds to the existing fatigue model adopted by Austroads for cemented fatigue, which means that the failure criterion for cement treatment of less than 3% would likely be dictated by other criteria.

Furthermore, specimens with more than 4% cement depicted distinct traits of fatigue development phases, i.e. a significant drop in modulus during the initial stages as shown in typical S-N curves in Figure 5.

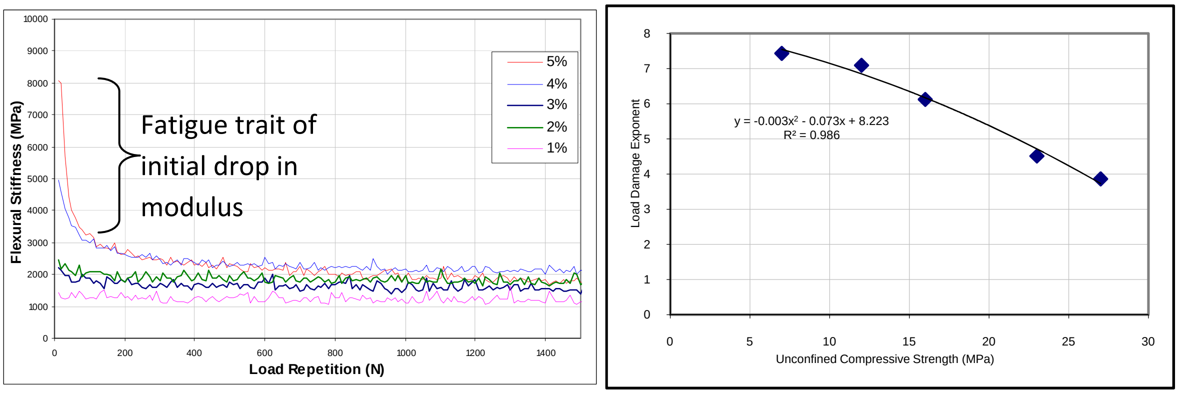

Figure 5: Typical S-N Curves and UCS vs. LDE

Correlating these observations back to UCS values of samples, the applicability of AUSTROADS classification and current MAINROADS classification should be reviewed. Bound behaviour of materials is exhibited at a minimum cement content of 3% with distinct development only at 4% addition. A distinct second degree polynomial relationship is shown between UCS and modulus of rupture and subsequently the load damage exponent (LDE) as shown in Figure 5.

LIMITATIONS

The measured flexural modulus from a static and dynamic test shows a noticeable variability between the two testing methods. The variability of results of this investigation beckons the need for more tests to be undertaken, especially to this regard. The results indicate that the testing regime for flexural moduli is an intricate task and a standardised methodology is to be developed. Admittedly, this is limited by the capacity of testing frames, loading equipments and duration tests.

CONCLUSIONS AND RECOMMENDATIONS

The experiences of Main Roads Western Australia in the past which now dictates the limitations of bound materials in Western Australia should be critically reviewed against current knowledge of flexural behaviour and fatigue. This will ultimately pave the way to the development of Western Australian pavement design guidelines with cementitiously bound layers. This paper has shown that the flexural behaviour of cementitiously stabilised crushed rock base course has a LDE of more than 12 at lower cement content. It is also critical to reassess the UCS classification methodology applied both locally in Australia and Internationally. As the purpose of classification is to explain the mechanical behaviour of a material and subsequently its failure criterion, the flexural behaviour can be investigated to be a more superior classification criterion. Moreover, standardised tests should be developed to ascertain the flexural properties of the material with specific interests in assessing the effects of specimen geometry. It is also prudent to increase the data pool of test results in Western Australia and thus the robustness of the research which is now undertaken at the Curtin Pavement Research Group.

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MOISTURE SUSCEPTIBILITY OF CEMENT STABILISED BASECOURSE

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ABSTRACT

Moisture ingress is a primary catalyst for pavement damage and plays a key role in the performance of pavement materials in service. Moisture intrusion eventuates to early development of deficiencies (potholes) due to “pumping” effects and reduced effective strength of the pavement. Cement stabilisation is one of the preventive measures that are applied to minimise moisture ingress into pavements. However, water induced damage is not completely eliminated since chemical retardation as a result of carbonation of cement also occurs with the presence of water. This phenomenon has been observed around the world and in Western Australia. An investigation is thus undertaken to ascertain moisture ingress in cement stabilised crushed rock base course. The Tube Suction Test (TST), developed by Texas Department of Transportation (TxDOT), is used. The TST is a non-destructive testing method that measures the dielectric permittivity of materials which is a measure of the moisture content in a material. The TST is pioneered in Western Australia to determine its applicability for laboratory investigation and use on cement stabilised crushed rock. The test utilises the Adek Percomenter™ to determine dielectric permittivity of cement stabilised crushed rock specimens with varying cement content. It is determined from the investigation that the TST is a highly potential tool for laboratory assessment of moisture susceptibility and strength.

KEY WORDS

Tube Suction Test, moisture, pavements, stabilisation

INTRODUCTION

The pivotal role that moisture plays in the behaviour of soil has been well established since the mid 1900s through the works of Casagrande and Terzarghi. Despite the importance of water during construction to achieve compaction, its detrimental effects on pavements cannot be dismissed especially since road embankments are so often exposed to harsh weather scenarios and are critical in maintaining access during and such severe climatic events. However, imperfections in compaction, inconsistent material gradation, construction defects, and other factors, leads to increased vulnerability of pavements to moisture induced damage.

Moisture enters pavements through various mechanisms which include infiltration, seepage, capillary and fluctuations in water table as shown in Figure 1 (Lay 1998) below. These mechanisms are highly dependent on the permeability of pavement materials and its subgrades.

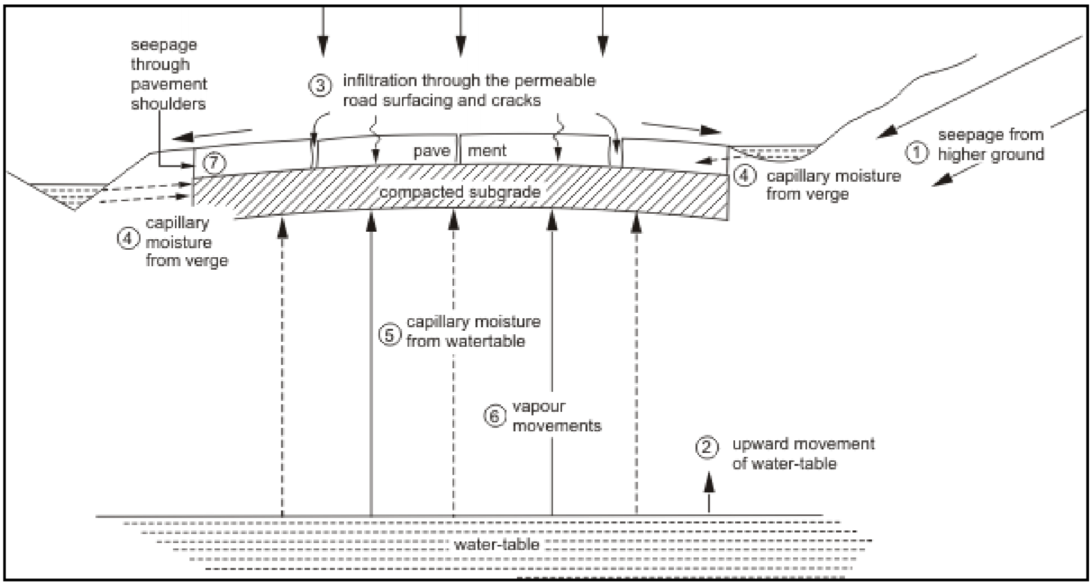

Figure 1: Mechanisms of moisture intrusion into pavement (Lay 1998)

With the combined effects of moisture ingress and traffic loading, voids are formed at the interface of pavement layers or at the subgrade. This induces a pumping effect or reduced subgrade support which eventuates to water induced damage such as potholes and alligator cracking as shown in Figure 2 below.

(a) (b)

Figure 2: Deteriorated pavements due to Water induced damage, (a) pothole (b) alligator cracking (pavementinteractive.org)

With this risk of undergoing water induced damage, pavements are designed with two control measures, i.e. to either allow water to drain freely through the pavements or to minimise water ingress into the pavement structural layers. While subterranean drains and permeable sub-base layers are used typically to achieve the first of the two methods, cement stabilised layers are constructed to assist in minimising moisture fluctuations of pavements by forming a theoretically impermeable layer.

However, even when a cemented layer is used, the issue of moisture ingress is not eliminated. The durability of cement stabilised base courses are compromised when moisture is present in the pavement layer. The Texas Department of Transportation (TxDOT) has in the past experienced the “disappearing” of stabilisation content through the service life of a pavement (Scullion et al. 2005). It was concluded that the loss of stabilisation was due to moisture ingress which causes a chemical retardation (Scullion et al. 2005). This is consistent with 100 case studies in South Africa (Paige-Green et al. 1990) and observations in trial pavements constructed in Western Australia. Main Roads Western Australia (MRWA) constructed several trial pavements in 1996 to observe the performance of cementitiously modified base course material, i.e. pavements with low stabilising content or disturbed prior to placement. Between the years 1996 to 2008, observations of these trial pavements noted that the stabilising agents within base courses have experienced a similar fate (Harris and Lockwood 2009). It was discussed within Harris and Lockwood’s report that this deterioration was associated to the chemical retardation process known as carbonation, a process that occurs with the presence of water.

Carbonation occurs in cemented pavement layers because the calcium bearing phases present are attacked by carbon dioxide of the air and converted to calcium carbonate based on the following process (Paige-Green et al. 1990).

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

Essentially, this means that a reversal of reaction occurs and the stabilising agents revert to its original components and are now more easily dissolved into water.

Furthermore, an inadequately compacted cement stabilised base course has also the propensity to allow water to be trapped at the interface between layers (Thom 2008). When loaded, a surge in pore pressure will also cause the “pumping” effect that will severely damage pavements.

This investigation therefore investigates the moisture susceptibility of cement stabilised base course in Western Australia using the Tube Suction Test (TST).

TUBE SUCTION TEST, DIELECTRIC MEASUREMENT AND ITS RELATIONSHIP TO STRENGTH

The earlier form of Tube Suction Test was developed by the TxDOT to analyse the behaviour of ground-penetrating radar (GPR) signals of pavement materials (Guthrie and Scullion 2003) to formulate non-destructive methods for assessing in service roads. From these tests, it was noted that the dielectric permittivity, E_R, of materials was capable of characterising pavement materials. Through further funded research and a joint investigation between the Finnish National Road Administration and the Texas Transportation Institute (TTI), a standard Tube Suction Test was developed to assess the moisture susceptibility of granular materials (Barbu and Scullion 2005). Further researches were then undertaken by Scullion et al. (2005), Guthrie et al. (2001) and George (2001) on the moisture susceptibility of cement stabilised materials using the Tube Suction Test with promising results to ascertain the durability of the material.

The Tube Suction uses the Adek Percometer™ to measure the dielectric value of cylindrical specimens. The measurement provides an indication of the total suction value of the material which is a function of matric suction and osmatic suction (Saarenketo 2000). These properties provide means to ascertain the susceptibility of moisture to enter the material and thus an indication of its propensity to undergo chemical retardation and pore pressure damage.

SPECIMEN PREPARATION AND TESTING REGIME

Crushed rocks sourced from Holcim quarry in Western Australia which meets Main Roads Western Australia Specifications 501 for aggregates are used for this experiment. General Purpose (GP) cement is used for stabilisation. Specimens are compacted to a target modified dry density 2.35 t/m^3 at OMC. The crushed rock sourced is widely used in Western Australia as base course material.

In undertaking this research, three specimens for each cement mix design measuring 105mm in diameter x 115mm in height are prepared for cement content by weight ranging from 1% to 6% cement. The specimens are then cured for 7 days in 70% relative humidity and at a constant temperature before complete dryback for 24 hours in an oven at 60^oC. Specimens are then wrapped with clear foil before being soaked in an enclosed water bath for 9 days with dielectric measurements undertaken every 24 hours using the Adek Percometer™ as shown in Figure 3 below.

Figure 3: Measurement of Dielectric Permittivity

Specimens are weighed in every reading and the specimens are tested for unconfined compressive strength to AS5101.4 (Australian Standards 2008) in the Concrete Laboratory at Curtin University as shown in Figure 4 below. The loading rate is maintained at 1.0mm/min typically.

Figure 4: Unconfined Compressive Strength Test

TUBE SUCTION TEST RESULTS AND DIELECTRIC TANGENT

The dielectric permittivity of specimens plotted against time over the course of the experiment is shown in Figures 5 (a) to (e).

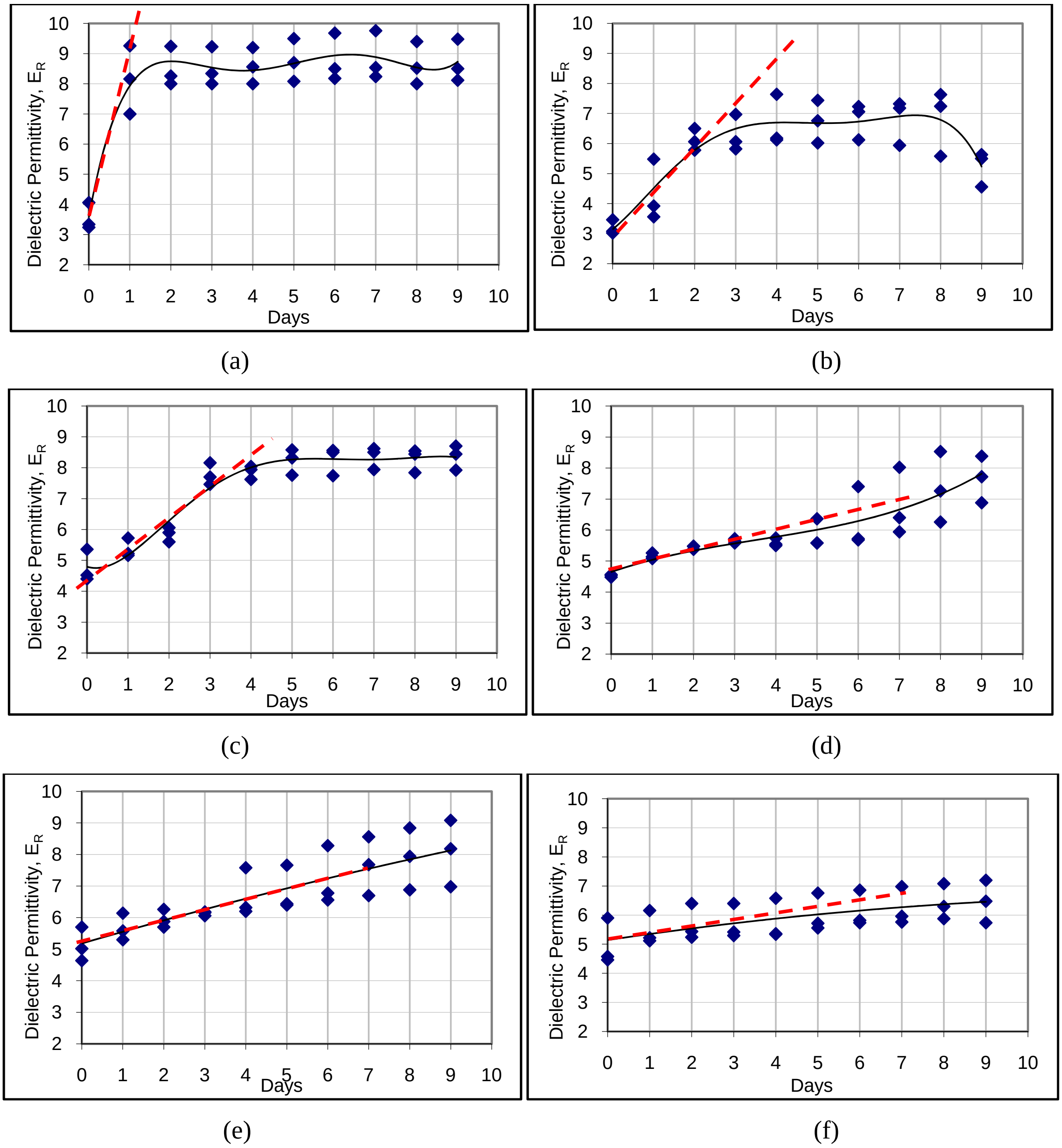

Figure 5: Dielectric permittivity over testing period for (a) 1% Cement; (b) 2% Cement; (c) 3% Cement; (d) 4% Cement; (e) 5% Cement; (f) 6% Cement

The dielectric permittivity of each mix design at the start of each test, i.e. after 24 hour dry back, generally rose when cement content increase. It ranges from approximately E_R = 3 at 2% cement to E_R = 5 at 6% cement. Within the tested timeframe, the 2% and 3% cement mix saw the specimens reaching a distinct saturation point while specimens with 4% cement content and above did not. It is also noted that specimens at 2% cement mix experienced a plunge on the final reading which can be associated to specimen mishandling. The profile of TST results test has also been discussed in Saarenketo (2000), where the plateau profiles shown for specimens 2% and 3% are typically noted as lower performing material compared to the constant profiles shown by the other specimens.

However, a more distinct trait noted from the Tube Suction Test was the initial rate of increase of dielectric permittivity measured from the tangent of the fitted curve (shown as a red dotted line on Figures 5 (a) to (e)), denoted as the dielectric tangent. The dielectric tangent generally represents the rate of moisture ingress and which can characterise the moisture susceptibility of stabilised specimens and subsequently the durability of the specimen. The analysis shows that the dielectric tangent reduces when cement content increases and a specific leap from 3% to 4% cement content can be observed. This indicates that moisture susceptibility of cement stabilised materials reduces with increasing cement content. By 4% cement content the moisture susceptibility reduces significantly which implies that a higher resistance to moisture ingress. The dielectric tangent is summarised in Table 1 below

Table 1: Dielectric Tangent from Tube Suction Test

Cement Content (%)	1	2	3	4	5	6
Dielectric Tangent (ε_R/day)	4.59	1.40	0.92	0.31	0.35	0.17

IMPLICATIONS OF TUBE SUCTION TEST

To ascertain the implications of the Tube Suction Test, the dielectric permittivity is compared against actual moisture content within specimens, the moisture content of specimens throughout the course of the tests are plotted against dielectric permittivity as shown in Figure 6 below.

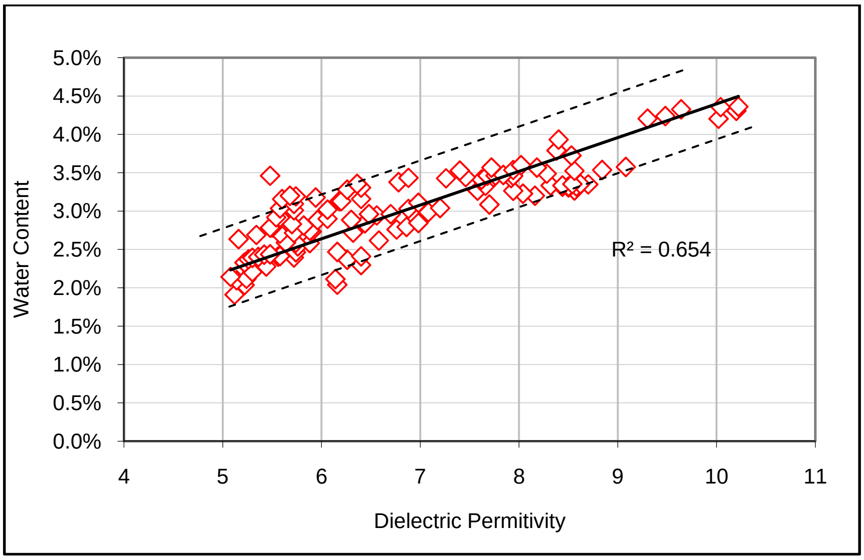

Figure 6: Dielectric Permittivity vs. Water Content %

Figure 6 shows that the Tube Suction Test shows a good representation of the moisture content of a specimen, with an R^2 of 0.654 and an error margin of ± 0.5%. From the test the relationship between dielectric permittivity and water content is given as,

$$\text{water content (\%), wc} = 0.4 \times E_R$$

Moreover, the measured dielectric permittivity is compared against UCS measured at the end of each test, i.e. at day 9. A very encouraging and strong relationship can be seen between the dielectric measurement and UCS of a specimen as seen in Figure 7 below.

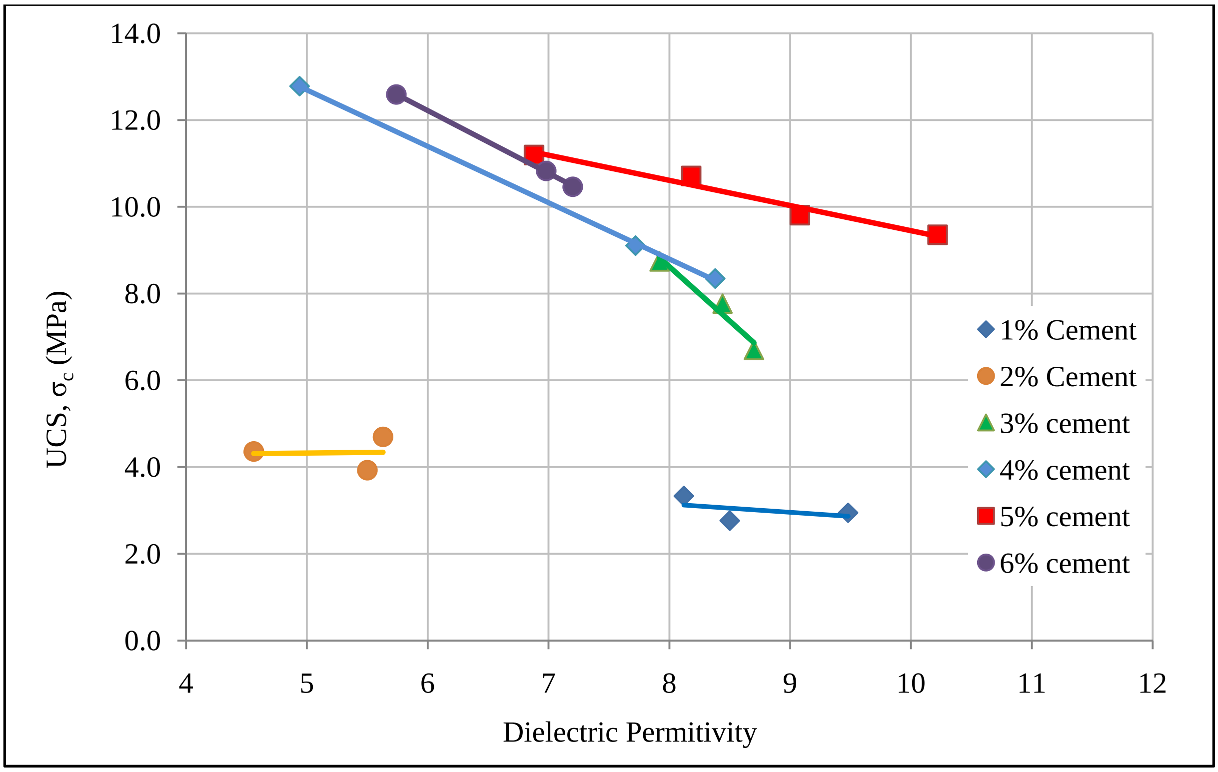

Figure 7: Dielectric Permittivity and UCS

A linear trend line is shown against the data in Figure 7 to provide an indication of the level of linearity of the tests undertaken for each mix design. The author recognises that the sample size of specimens investigated is relatively low, but a clear linear relationship can be developed between UCS and dielectric permittivity for specimens with cement content more than 3%. As a result, the Tube Suction Test can potentially be a superior strength test for in situ pavements in that it may be possible to infer in situ water content and strength of base course. However, more tests are required to ascertain the reproducibility, reliability and repeatability of the Tube Suction Test in Australia.

CONCLUSIONS AND RECOMMENDATIONS

The test undertaken in this paper has shown that the Tube Suction Test has significant potential to characterise materials in Western Australia both in a laboratory environment and in-situ for cement stabilised pavements. The Tube Suction Test allows the measurement of moisture content within a ±0.5% error margin which in turn can be interpreted as a measurement of durability. Also, the dielectric tangent measured in this test is capable of explaining the moisture susceptibility of specimens. In general, lower measurements of dielectric permittivity and dielectric tangent means a lower susceptibility to moisture ingress and thus a more durable cement stabilised pavement mix design. From the tests undertaken, at 3% cement content by mass, the base course specimen exhibits marked improvement against moisture sensitivity.

It is recommended that more laboratory tests are undertaken followed by in-situ tests using other materials. Furthermore, given the relationship between pore structure, permeability and shrinkage, it is in the author’s opinion that the Tube Suction Test can potentially be used as an indication of shrinkage potential of materials, a research currently undertaken by the author.

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Dynamic Effects on Fatigue Life of Cement Treated Crushed Rock

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Synopsis: Fatigue life prediction of cement treated crushed rock is not a well understood concept in pavement engineering. The complexity to understand fatigue response of pavement lies not only in the structural model or the defining criteria for measurement, but also includes the testing regime to be adopted. Despite the well established testing methodologies for asphalt, minimal literature and standardised laboratory regimes exists for cemented basecourse. Fatigue testing of these materials is complicated by a multitude of variables that has to be considered in order to provide meaningful and representative data for design. Recent studies have been undertaken in Australia recently to characterise the fatigue phenomenon using a four point bending test setup by Austroad. Nevertheless, the loading frequency on fatigue life has not been addressed explicitly. A laboratory investigation to assess the loading frequency during pavement testing is undertaken. The paper presents the challenges and results achieved from test.

Keywords: fatigue, dynamic loads, flexural beam test, cemented materials.

1 Introduction

Fatigue was first discovered after World War 2 and is a phenomenon that is not well understood in the engineering world. Even so, the study of fatigue predominantly revolves around homogenous materials such as steel and other metals. Regardless, fatigue damage is the predominant failure mechanism of stiff pavement layers, e.g. asphaltic concrete seals and cemented basecourse layers. These structures are made up of composite materials which add considerable complexity if a mechanistic explanation is sought to characterise their structural behaviour.

Austroads mechanistic design guidelines idealises the structural analysis of pavements using a multi-layered model (Austroads 2010a). Within the model, cemented base courses are characterised as bound materials having developed tensile strength from the formation of interlocking cement matrices between aggregates. The critical response of this layer is designed as the tensile strains at the base of the layer (Austroads 2010a), where distress is manifested as a bottom-up fracture. The model further assumes the pavement layer to be homogeneous, elastic and isotropic. These assumptions allow the simplification of the pavement structure for elastic analysis as part of the Mechanistic-Empirical design adopted in Australia (Austroads 2008). The M-E approach is selected as a compromise between the two idealisations since at either end of the spectrum the analysis will either be overly complex or unrepresentative (Austroads 2008).

With the advent of supercomputers available at men's fingertips, academia now seeks to produce a - paradigm shift in fatigue life prediction of cemented layers using mechanistic models. Significant works have been done for asphaltic material (Thom 2010) due to the financial incentives of improving the material. However, limited understanding has been developed for fatigue life prediction of cemented basecourse, especially in Western Australia.

The fatigue mechanism of cemented material is characterised as a reduction in stiffness (Austroads 2010b) caused by an accumulation of damage at locations of inhomogeneities (Balbo and Cintra 1996) rather than a distinct transverse rupture normally seen from ultimate loadings. The accumulation of damage by the pavement structure averaged across the volume of material affected from repeated traffic loads ultimately reaches a distress limit whereby the localised cement matrices disintegrates, consequently resulting in block cracking or aggregates returning to its original unbound mechanical state, a service stage known as equivalent granular phase (Austroads 2010a).

A recent study by Austroads (2010b) have shown that the four point bending test have shown a substantial correlation between the testing regime adopted and fatigue induced deflection of pavements loaded by the Accelerate Loading Facility. The testing regime adopts havershine pulses of 2 Hz which includes a 250ms rest period between pulses as shown. The selection of the loading frequency was dictated primarily by the time consuming nature of fatigue testing. However the impact of loading frequency, or the dynamic effect of loads, to the test was not discussed.

2 Dynamic Effects of Load on Pavements

The assessment of dynamic response for pavements is critical in anticipating road damage properties particularly with cement treated materials the dominant failure mechanism is fatigue (Beskou and Theodorakopoulos 2011). Beskou and Theodorakopoulos (2011) have provided a thorough and comprehensive literature review on the effects of dynamic loads on pavements, where various numerical models developed to date have been presented.

Under constant velocity, the deflected shape of pavements under a specific traffic load is similar at any given time, i.e. the deflected shape travels with the load (Kim and Roesset 1998). Conversely, at constant amplitudes, when velocity increases, more pronounced fluctuations occur and the maximum displacement and the effected region of the traffic load increases in the direction of the traffic movement while in the lateral direction only maximum deflection increases (Kim and Roesset 1998) as shown in Figure 1 below.

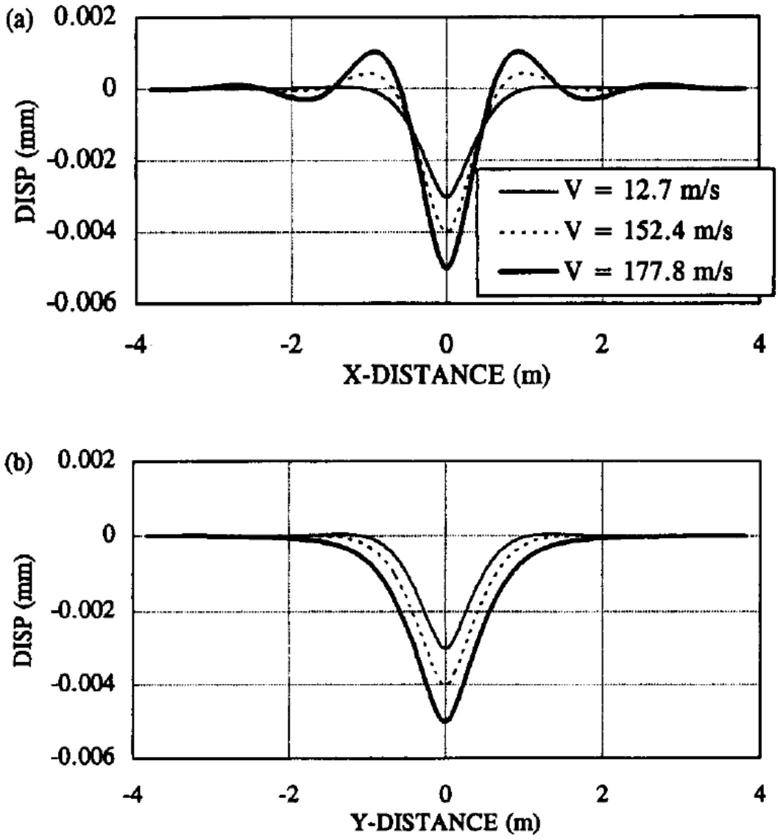

Figure 1: Effects of velocity on deflected shape along: (a) x-axis (moving direction); (b) y-axis (lateral direction) (Kim and Roesset 1998)

Nevertheless, due to the effect of damping, the deflection increases as the velocity increases to a critical resonant velocity, where deflection then begins to decrease. On this note, when pavements undergo moving harmonic loads for velocities before reaching critical velocity and after reaching velocity shows varying results as shown in Figure 2 below.

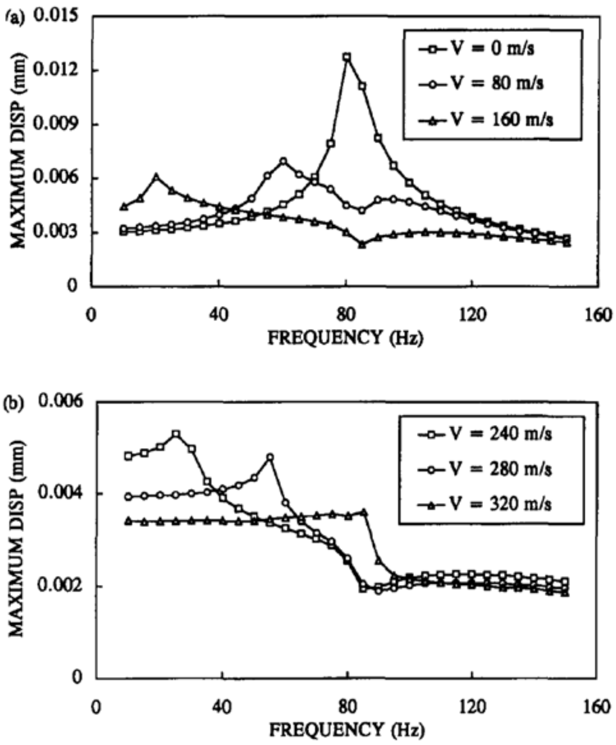

Figure 2: Relationship between maximum deflection, velocity and load frequency when (a) velocity are smaller than V_{cr}; (b) velocity are higher than V_{cr} (where V_{cr} = 195m/s) (Kim and Roesset 1998)

Based on the data above, it is observed that the dynamic effects as a function of loading frequency do affect the deflection behaviour of pavements.

3 Testing Methodology – Four Point Bending Test

As discussed, the complexity of characterising fatigue life does not only lie on the numerical properties to be considered, but also includes the various testing methods available. In general, two predominant form of testing are most common, first, the indirect tensile configuration and second, the flexural beam configuration. Neither configuration provides a realistic representation to actual pavements however, recent work by Yeo (2008), have shown a better correlation between flexural beam and in service pavements. This paper has therefore utilised the four point bending test.

Sufficient to say, the fatigue data of specimens are limited due to countless challenges of tests. Table 1 below therefore presents a discussion on each consideration applied to the test to isolate or control the assessment of dynamic effects:

Table 1: Testing Configuration for Beam Fatigue Test

Testing Configuration	Discussion
Duration of test	Austroads (2010b) have identified three stages of fatigue test, i.e. an initial stage, an effective fatigue life stage, and an equivalent granular stage. The longer the duration of the test, the higher the propensity to cause permanent deformation. Specimens used by the author for fatigue life characterisation has been used. Upon achieving a resilient state without undergoing fatigue damage, specimens are tested for an additional 1200 cycles to provide sufficient data to assess the dynamic effects
Loading Shape	Haversine load patterns are believed to be the most representative of traffic loading and have been adopted as seen in Figure 2 above.

Loading Type and Magnitude	The applied load may be a constant strain or constant stress. Realistically, load varies based on the thickness of the overlying pavement and the type of vehicles. Austroads (2010b) have identified a strong relationship between strain and fatigue life, the constant strain setting has been used. 75$\mu\varepsilon$ has been selected, representing typically 65% of the strain at break of cemented basecourse where appreciable damage is limited for 1200 cycles.
Rest Periods	Rest periods are the pause between successive loads and are important to the analysis of asphaltic materials as healing can potentially occur between applied loads. However, no literature has suggested that healing occurs in cemented material within the timeframe of the test, as the rehydration of broken bonds takes a much longer period and requires sufficient moisture to occur. No rest periods have been allowed with loads applied successively. This also assists in shortening the testing duration.
Size of Specimen	The size of quassi-brittle materials play an important role in the propagation of cracks, a mechanism closely related to fatigue development (Bazant 2003). With cement treated pavements typically built to 150 – 250mm thick and maximum aggregate sizes of the cement mix 35mm, the ideal specimen size would be at a minimum of 100mm x 100mm in cross section. Specimens measuring 390mm (L) x 63mm (W) x 50mm (D) are prepared for this test due to the limitations of the testing rig available.
Laboratory Compaction / Specimen Preparation	There is no current standard methodology to fabricate beams used for flexural beam tests. An in-house mould has been prepared and specimens are hand compacted with a modified Proctor compactor to MMDD of 2.35 t/m^3 at OMC.
Cement Content	Typical cement treatment for cement treated basecourse material ranges from 2% to 4% content by mass. Specimens with 2%, 3% and 4% cement content are produced and tested.

With the above testing configurations controlled, a laboratory test is undertaken to assess the effects of varying frequency. The loading frequency is varied for 4Hz, 10Hz and 100 Hz to provide a data range for assessment.

4 Equipment and Materials

The Four Point Bending Test Fixture for UTM fitted to the IPC Global Repeated Triaxial Load Test for asphalt materials available at the Curtin University Pavement Research Centre is used for the analysis as shown in Figure 3 below.

Figure 3: IPC four point Bending test rig

The test measures the flexural stiffness of a specimen when applied with a strain, where the flexural stiffness, S, is given as,

$$S = \frac{\sigma_f}{\varepsilon}$$

Crushed rocks sourced from Western Australia which meets Main Roads Western Australia (MRWA) Specifications 501 for aggregates are used for this experiment. Cement Type General Purpose (GP) conforming to Australian Standards AS372 is used for stabilisation and are mixed dry to the specimen prior to the addition of water.

5 Results for Dynamic Test

A typical result for a specimen with 2% cement content by mass is shown in Figure 4 below. Figure 4 presents the relationship between flexural stiffness under the three different loading frequencies applied for up to 1200 cycles undertaken after completion of fatigue tests. It is noted that a significant amount of noise is experienced for lower frequency testing, which can be potentially attributed to the difficulty of the pneumatic system to maintain a constant pressure with high rest periods. Also, to better visualise the results, an accumulative average line is shown on the scatter to determine whether any appreciable damage is still being absorbed by the specimen and to quantify the impact of the varying frequency.

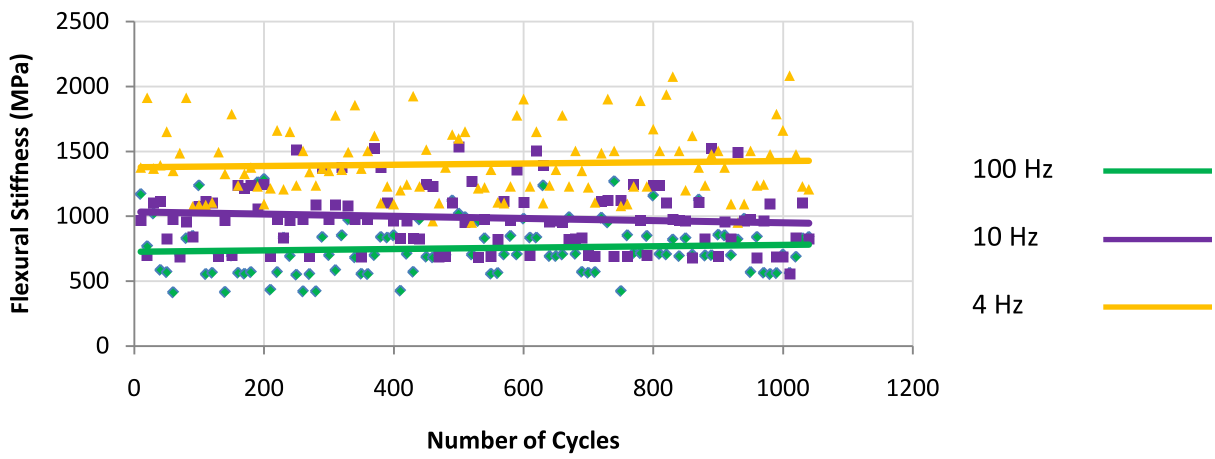

Figure 4: Typical specimen under 4Hz, 10Hz and 100Hz applied strain frequency

A distinct reduction in flexural stiffness, or stress required to achieve a constant strain, is observed from Figure 4 above. This observation is summarised along with specimens with 3% and 4% cement content by mass in Figure 5 below.

Figure 5 shows the results plotted on a logarithmic scale on the x-axis (frequency). It can be seen that a linear relationship exists between log-frequency and flexural stiffness, which suggests that a semi log relationship occurs between load frequency and flexural stiffness. Generally, the increase in frequency results in the reduction of flexural stiffness, indicating that the pavement is weaker and that a lesser load is required to induce a similar amount of strain. This in turn suggests that faster vehicular speed would induce more damage as discussed previously.

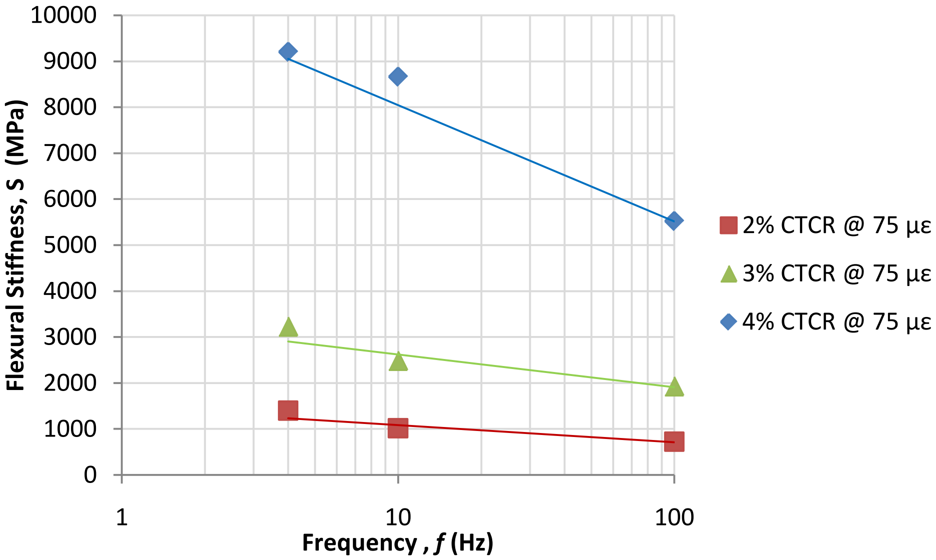

Figure 5: Flexural stiffness vs. frequency of applied strain

The data above also means that loading frequency affects the testing regime for fatigue and an assessment on what best represents traffic loading has not been addressed in this paper. This ambiguity also implies that the laboratory works may not yield reliable data for pavement testing. Future studies will be required in order to more confidently ascertain the implications of load frequency to pavements.

6 Conclusion

This paper has presented evidence to support that the increase in loading frequency will result in reduced stiffness, signifying the detrimental effects of vehicular frequency on pavements. The results are however indicative and the data do not provide a quantitative measure for pavement design.

Instead, the data provided in this laboratory investigation points out that ambiguity exists in the appropriate loading frequency to be used. Further studies are required in order to undertake laboratory investigations that are capable of providing meaningful results.

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Moisture Ingress of Cemented Basecourse

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Synopsis: Moisture ingress is a primary catalyst for pavement damage and plays a key role in the performance of pavement materials in service. This is evident from the issues faced as a result of high moisture sensitivity of crushed rocks used in the construction of Kwinana Freeway in Western Australia. Cement treatment is deemed a potential solution to reduce moisture sensitivity. The moisture ingress into cement treated crushed rocks can be based on the unsaturated flow theory and quantified with the term Sorpitivity, S, i.e. the square root rate of inflow volume. The linearity of the Sorptivity when plotting inflow volume against the square root of time, $t^{0.5}$ also provides an indication of the homogeneity of the material. This paper assessed the Sorptivity of cement treated crushed rocks based on results from the Tube Suction Test procedures developed by Texas Department of Transportation (TxDOT). The tests shows that Sorpitivity decreases with the increase in cement content which means that higher cement content reduces the moisture sensitivity of pavements. The Sorpitivity also showed a high least-square regression of $R^2 > 0.9$, which indicates that the materials are homogeneous.

Keywords: Sorptivity, cemented basecourse, unsaturated flow, pavement, moisture ingress.

1. Introduction

Crushed rock base course is an economical option for road construction in Western Australia due to its availability and economy, which lead to its use in the construction of Kwinana Freeway, Western Australia in 1992. However, prior to its completion severe deficiencies manifested as large curvatures measured with the Benkelman beam were measured in a number of sections along the freeway.

This prompted Main Roads Western Australia (MRWA) to undertake an in depth investigation of the behaviour of crushed rock base to better understand the basis behind the failures of Kwinana Freeway (Watson 1995). The investigation involved an extensive laboratory program followed by the construction of trial pavements.

It was identified that the failures were associated to the sensitivity of the basecourse to moisture ingress. The laboratory program therefore consisted of repeated load triaxial tests of crushed rock base course under various moisture levels and treatment. It aimed to identify the material's response to changes in density and moisture and also to study improvement options to minimise the effects of moisture. These options included drying back, cement treatment and modification in particle size distribution (Buktus and Lee Goh 1997). Trial sections were then constructed as part of Reid Highway to evaluate the recommendations concluded from the laboratory investigations.

As a prologue to this paper, the discussions for crushed rock base sensitivity to moisture and effects of cement treatment from MRWA's investigation are presented.

2. Moisture Sensitivity, the Hydration Test and Cement Modification

As part of the laboratory work commissioned by MRWA to investigate crushed rock base course, the effects of density and moisture to resilient modulus were assessed. Samples with varying densities and moisture were prepared to 100/80, 98/60, 98/50, 96/80 and 96/60 (dry density ratio/moisture ratio) to represent the in service conditions of base courses. The results of these analyses are shown in Figure 1 and 2 below (Butkus and Lee Goh 1997).

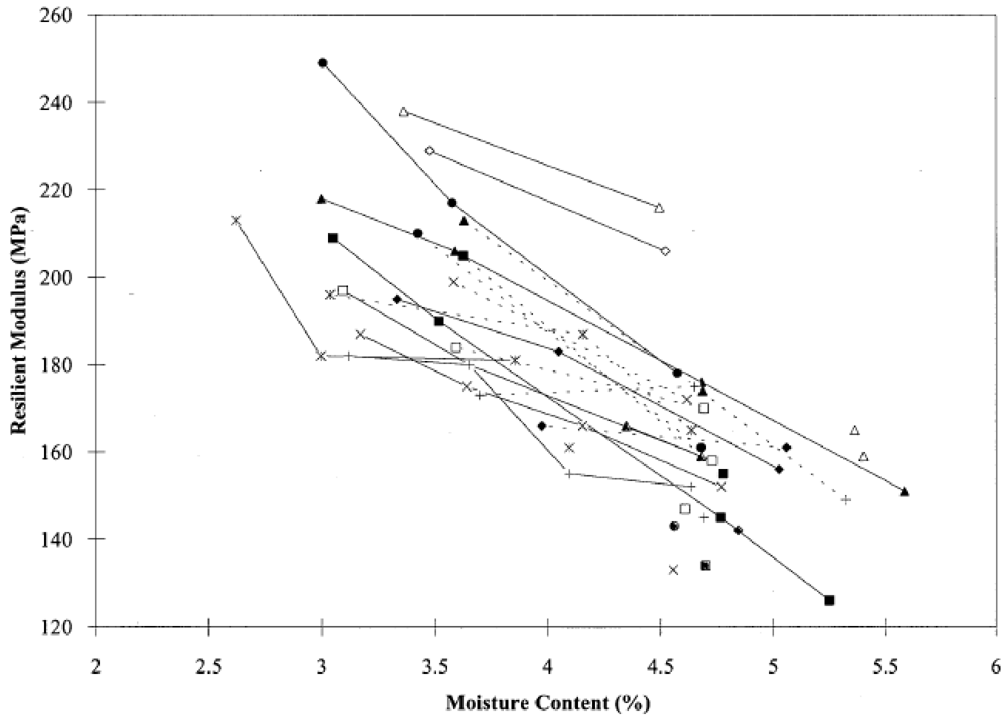

Figure 2. Relationship of moisture content and resilient modulus

As clearly seen from Figures 1 and 2, crushed rock base course are highly sensitive to moisture fluctuations where moisture ratios in excess of 60% result in reduced resilient modulus of the material (Buktus and Lee Goh 1997). This was also supported from test results from in-situ crushed rock base undertaken as part of the investigation. Following this, tests were undertaken to assess the use of cement to modify the behaviour of crushed rock base.

The cement treated crushed rock were tested for its performance against various cement content (0.5%, 1%, 2% and 3%), cement set time prior to compaction, curing time and a Hydration Test. The Hydration Test was designed by MRWA to assess whether part of the improvement of crushed rock base was due to factors other than the cementation process. The test involves an interference of the cementation process by regularly remixing the material prior to compaction. These properties and the conclusions are shown in Table 1 below

Table 1. MRWA Investigated Improvement Options

Properties	Cement content tested	Observation
Resilient modulus	0.5, 1, 2 and 3	• Increased performance generally (increased resilient modulus, strain rate and permanent strain) • Increased performance with increased moisture ratio
Cement set time	2	• Decrease performance with increasing set time
Curing Time	2	• Increased performance with curing time • Increased performance with increased moisture ratio
Hydration Test	2	• Reduced sensitivity to moisture

From the test results, it is concluded that the treatment of cement provided better performance against repeated load. However, tests undertaken by Lee Goh (1995) suggested that crushed rocks base course will behave as stabilised (bound) materials when more than 1% cement is applied because the measured Unconfined Compressive Strength (UCS) is more than 1 MPa.

Instead, the modification through treatment of cement less than 1% and the product of the Hydration Test was deemed as options to reduce the moisture sensitivity of the crushed rock base. These options were tested through trial pavements constructed on Reid Highway.

Detailed information regarding the trial pavements are reported in Harris and Lockwood (2009). In summary, the Reid Highway trial pavements concluded in the following observations pertinent to the two cement treatment options, i.e.

- Low cement options showed issues of homogenous distribution and permanency
- The hydrated cement treated crushed rock base (HCTCRB) initially showed good improvement against moisture sensitivity but were later exhibited issues with stabiliser permanency

With the measure of moisture a critical component to assess the integrity of pavements in Western Australia, this paper presents the mechanisms involved in the ingress of water and an alternative measurement method for moisture in pavements using the Tube Suction Test.

3. Unsaturated Flow and Sorptivity

In conventional soil mechanics, the permeability of materials are determined based on Darcy's law which assumes soils are in a fully saturated condition. However, pavements are dried back prior to seal and mostly unsaturated throughout their service life. The permeability under such conditions are known as unsaturated flow.

Unsaturated flow refers to the movement of moisture through porous materials where the water content is typically less than saturation and inhomogeneous (Hall and Djerbib 2006) and involves external and internal forces, i.e. gravity and capillary. Theoretically, the unsaturated flow theory builds upon the Darcy equation by introducing a dimensionless variable which represents the volumetric water content of the material which attenuates the permeability factor (Hall and Djerbib 2006). A methodology for measurement of unsaturated flow is the term known as Sorptivity.

The term Sorptivity (S) introduced to unsaturated flow theory was first used by Philip in 1957 to explain the absorption of water into a porous solid due to capillary suction (Hall and Djerbib 2006; Gonen and Yazicioglu 2006).

Sorpitivity is the rate of increase in water absorption against the square root of elapsed time. The cumulative volume water per unit inflow surface area as *i* can be represented as:

$$i = \frac{\Delta w}{\rho A}$$

where,	i	=	inflow volume (mm)
	Δw	=	change in weight (g)
	A	=	cross sectional area of test face (mm^2)
	ρ	=	density of water (assumed at 0.988 g ml^{-1})

Sorptivity, *S* is then defined as the gradient of the slope $i/t^{0.5}$ and its linearity represents the homogeneity of the specimen.

4. Dielectric Measurement and the Tube Suction Test

The Tube Suction Test is a moisture susceptibility test developed in the United States. Earlier versions of the Tube Suction Test was developed by the TxDOT to analyse the behaviour of ground-penetrating radar (GPR) signals of pavement materials (Guthrie and Scullion 2003) to formulate non-destructive methods for assessing in service roads. From these tests, it was noted that the dielectric permittivity, E_R, of materials was capable of characterising pavement materials. Through further funded research and a joint investigation between the Finnish National Road Administration and the Texas Transportation Institute (TTI), a standard Tube Suction Test was developed to assess the moisture susceptibility of granular materials. Further researches were then undertaken by Scullion et al. (2005), and Guthrie et al. (2001) on the moisture susceptibility of cement stabilised materials using the Tube Suction Test with promising results to ascertain the durability of the material.

Figure 3 Measurement of Dielectric Permittivity

Results of the Tube Suction Test is presented by the author in other publications (Yeo et al. 2011), however, the measurements as part of the Tube Suction Test are used to calculate the Sorptivity of the crushed rock base course.

5. Specimen Preparation and Testing Regime

Crushed rocks sourced from Holcim quarry in Western Australia which meets Main Roads Western Australia Specifications 501 for aggregates are used for this experiment. General Purpose (GP) cement is used for stabilisation. Specimens are compacted to a target modified dry density 2.35 t/m^3 at OMC. The crushed rock sourced is widely used in Western Australia as basecourse material.

In undertaking this research, specimens measuring 105mm in diameter x 115mm in height are prepared for cement content by weight ranging from 2% to 6% cement. The specimens are then cured for 7 days in 100% relative humidity and at a constant temperature before complete dryback for 1 day in an oven at 60^{o}C. Specimens are then wrapped with clear foil before being soaked in an enclosed water bath for 9 days with dielectric measurements undertaken every 24 hours using the Adek Percometer^{TM} as shown in Figure 3 below. Specimens are also weighed in every reading to determine the volume of water absorbed into the specimens.

5. Results and Discussion

The results for the inflow volumes calculated based on equation (1) are presented in Figures 3 below.

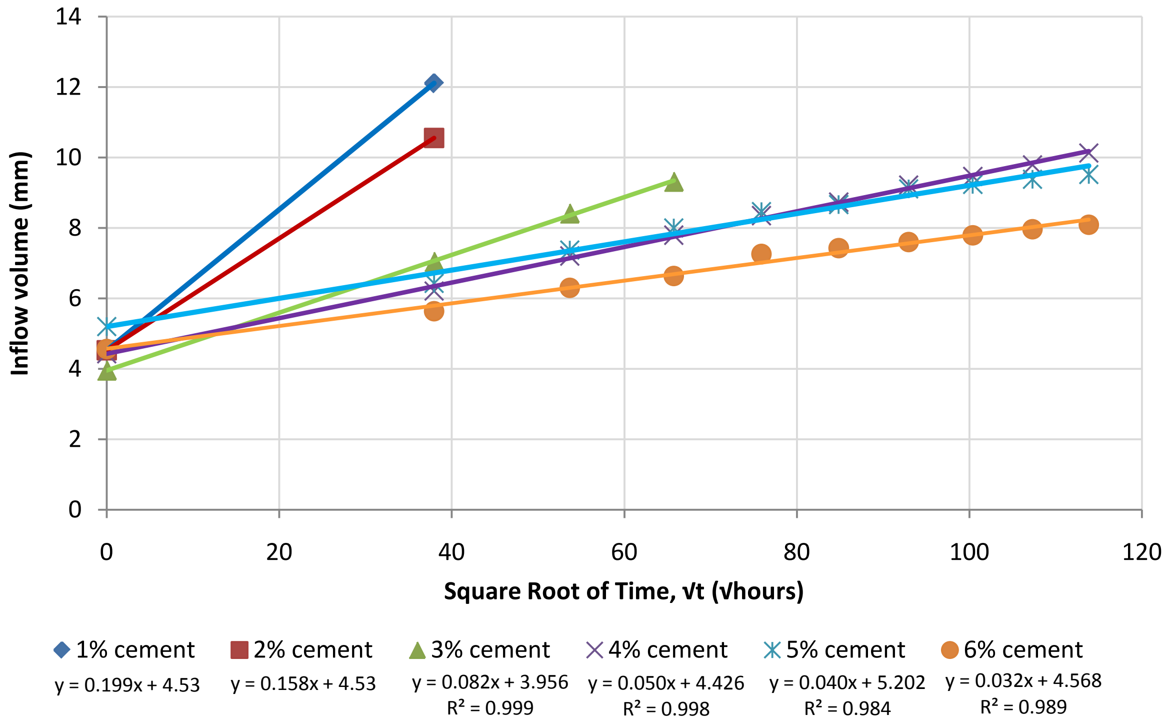

Figure 3. Sorpitivity of Crushed Rocks with 1% to 6% Cement Content

The r esults s hows t hat t he c ement t reated c rushed r ocks obe y the S orpitivity pr inciple, i .e. a l inear relationship is e vident between i and $t^{0.5}$. As ex pected, t he Sorpitivity r educes w ith i ncreasing c ement content allowing better resistance against moisture ingress. The results for 3% cement beyond 100 $min^{0.5}$ reaches saturation and is omitted in identifying the sorpitivity of the specimen. A marked decrease in the gradient, i.e. Sorpitivity is observed between 2% to 3%, which suggests that the susceptibility of cement treated c rushed r ock t o m oisture f luctuations r educes w hen 3% or m ore c ement ar e t reated t o t he material. F urthermore, bas ed on t he l east s quare r egression, R^2, t he m aterials t ested possess a h igh degree of homogeneity. The Sorptivity of the specimens are summarised in Table 2 below:

Table 2: Summary of test results

Cement Content (%)	Sorptivity ($mm/t^{0.5}$)	Homogeneity, R^2
1	0.199	-
2	0.158	-
3	0.082	0.999
4	0.050	0.998
5	0.040	0.984
6	0.032	0.989

7. Conclusion

Cement treated basecourse conforms to the unsaturated flow theory and therefore its susceptibility to moisture can be quantified by a measurement known as Sorptivity, i.e. the square root rate of inflow volume. The Sorptivity can be assessed by undertaking the Tube Suction Test which also provides database to test in-situ moisture content. The results suggest that with cement treatment of more than 3%, the material has a marked reduction in its susceptibility to moisture damage and fluctuation. Moreover, it also suggests that the primary defense of cemented basecourse is gained from its degree of impermeability. This notion would imply that the Hydrated Cement Treated Crushed Rock fabrication methodology used specifically in Western Australia may be highly susceptible to moisture damage.

The testing regime applied this investigation also provides an indication of the homogeneity of samples. The samples undertaken in this investigation showed high homogeneity manifested a high linear regression of $R^2 > 0.9$. It is recommended that further tests using cross sections should be undertaken and be tested with other materials.

8. References

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Mix Design of Cementitious Basecourse

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Synopsis: Selecting stabilisation mix designs for basecourse materials to provide adequate resistance against damage under traffic and environmental loading is important in maximizing the life of a pavement. Cement stabilised pavements are unique materials that they exist at the border between structural soil and conventional concrete. The pavement layer are typically roller-compacted and thus require sufficient water content to achieve compaction but at the same time also requires water sufficient for cement hydration and workability with a grader. A literature review combined with simple tests are undertaken to determine fit for purpose design mix, i.e. a compaction test to ascertain the optimum moisture content (OMC) and maximum modified dry density (MMDD) and unconfined compressive strength (UCS) test. The tests showed that the MMDD for the material at various cement content are a constant, and a fit for purpose design chart can be developed based on the water and cement content. The OMC for compaction of cement treated basecourse is the OMC of the parent material + 0.25% for every 1% in cement content. This relationship between water content and cement content runs parallel to the minimum water required for effective hydration to take place, i.e. a w/c ratio of 0.25.

Keywords: cement, basecourse, pavement, mix design, unconfined compressive strength.

1 Introduction

Stabilisation is a process of prescribing additives or binding agents to soil based materials to increase the performance of the materials for a specified purpose (Smith and Vorobieff 2007). Although with the variety of binding agents, cement is investigated in this paper due to its versatility for application as a stabilising agent for the majority types of soils (Auststab 2006) and its familiarity to the construction industry as shown in the excerpt from AustStab (2006) and Austroads (2006) in Figure 1 below. When cement is treated to soil, the stabilisation product is referred in general terms as soil-cement.

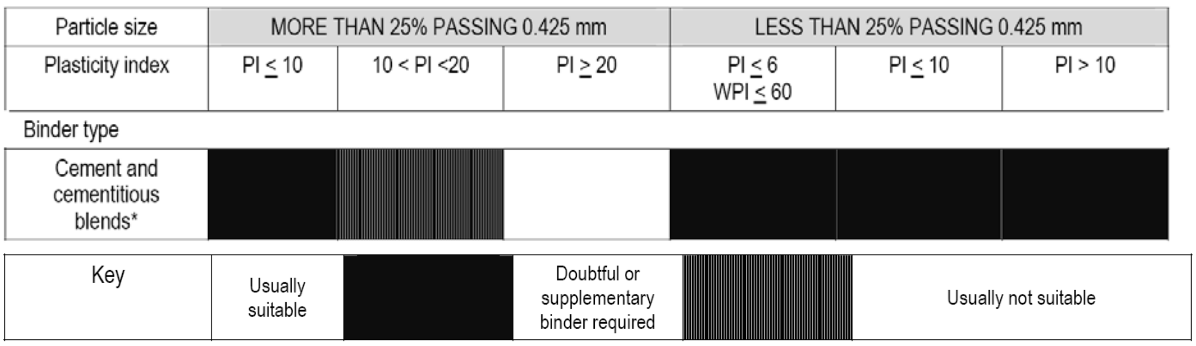

Figure 1. Suitability of Cement as a Stabilising Agent (AustStab 2005)

Soil-cement was first concocted in 1930s in a joint project between the South Carolina State Highway and the Portland Cement Association (PCA) (Scullion et al. 2005), who have since then been the leading experts in advancing studies in cement stabilisation. Among the application of soil-cement combinations, attention has been given to the stabilisation of pavement base course materials, typically composed of granular soil, as it forms the dominant structural layer in a pavement. The stabilisation of base course with cement is industrially known as Cement Treated Base (CTB) and is widely used in the United States, the Republic of South Africa and China (Cho et al. 2006).

In Australia, cement became a mainstream stabilising agent in eastern Australian states as early as 1950 through the establishment of a specialist contractor, leading to the construction of in situ stabilisation of local government roads in 1960s (Vorobieff 1998; Wilmot 1996). Motivated by sustainability of sourcing fit for purpose quality aggregates that meet performance criteria, deliver potential cost savings (Cho et al. 2006; Austroads 2010) and meet demands of continual increase of road networks in Australia (BITRE 2009), the use of cement stabilisation continues to present day and is even recognised as a potentially cost effective solution for rural road construction (Austroads 2010).

Cemented base course are a unique material in engineering as the material's behaviour spans between behaving as a "modified" unbound material to being a stiff material comparable to concrete. PCA (2005) have identified the production of diverse forms of soil-cement for different application and purposes by varying cement and water content. In comparison, Australia differentiates the varying percentage of cement content by its mechanical behaviour and degree of binding only, i.e. unbound, modified (semi-bound) and stabilised (bound).

Current Australian guidelines for stabilisation mix designs are provided by Austroads (2006). This guidelines emphasise mainly of the selection of binder type and provide approaches for assessing the structural properties of the mixture.

The strength and durability aspects of the pavement has been addressed by the author in other publications with this paper focusing on the challenges in proportioning materials to meet the design purpose of CTB in the context of Western Australia.

2 Considerations for Mix Design

Material used in Cement treated base (CTB) comprises of three elements, i.e. aggregate, cement and water. In Western Australia, the selection of aggregates is dictated by Specification 501 (MRWA 2011). The aggregates used in this study are crushed rocks sourced from Holcim Quarry in Gosnells, WA and comply with these requirements. General Purpose (GP) cement has been used for this study as studies undertaken by Butkus and Lee Goh (1997) and Butkus (2004) shows that the use of GP cement shows better performance compared to General Blend (GB) cement.

The proportioning of the three elements is crucial in order to provide a fit for purpose design. This paper provides a literature review of the roles and requirements of each elements along with simple tests to construct a material chart. These elements include:

- Water content
- Compaction
- Cement Content
- Compressive Strength

2.1 Water Content and Compaction

Unlike traditional concrete where water content is primarily driven by the workability of mixture and strength, water in the mix design of cement treated basecourse serves a third purpose of ensuring compaction of the basecourse is reached. The workability required for the mixture differs to that of traditional concrete since the material is placed via roller-compacting efforts instead of wet-forming Austroads (2009).

The gain in strength of a material is primarily a function of cement, but water plays a crucial role in ensuring that the hydration of cement is supplemented with the sufficient amount of water. Without sufficient water, the cement hydration process will not be fully activated (Thom 2010). The minimum water to cement ratio to allow hydration to occur is 0.22 to 0.25 (Hamory 1987; Thom 2010). At water to cement ratio in excess of 0.45, the hydration process is also overly diluted and creates porous matrices that are low in strength (Thom 2010).

Moreover, the increase in water content also reduces the strength properties of the material measured using the Unconfined Compressive Strength Test as shown in Figure 2 below (Butkus 1985). The results were commissioned by Main Roads Western Australia to assess the response of cement treated base course to changes in density and moisture ratio at compaction. Results show trends in unconfined compressive strength after 7 days, 28 days and 12 months of curing. The results show in Figure 2 shows that an increase in moisture content at compaction reduces the strength of the material.

Being predominantly soil in compound, the strength of the material is dependent on the compaction level achieved. As expected of the material, the unconfined compressive strength of the material increases with the density of the material.

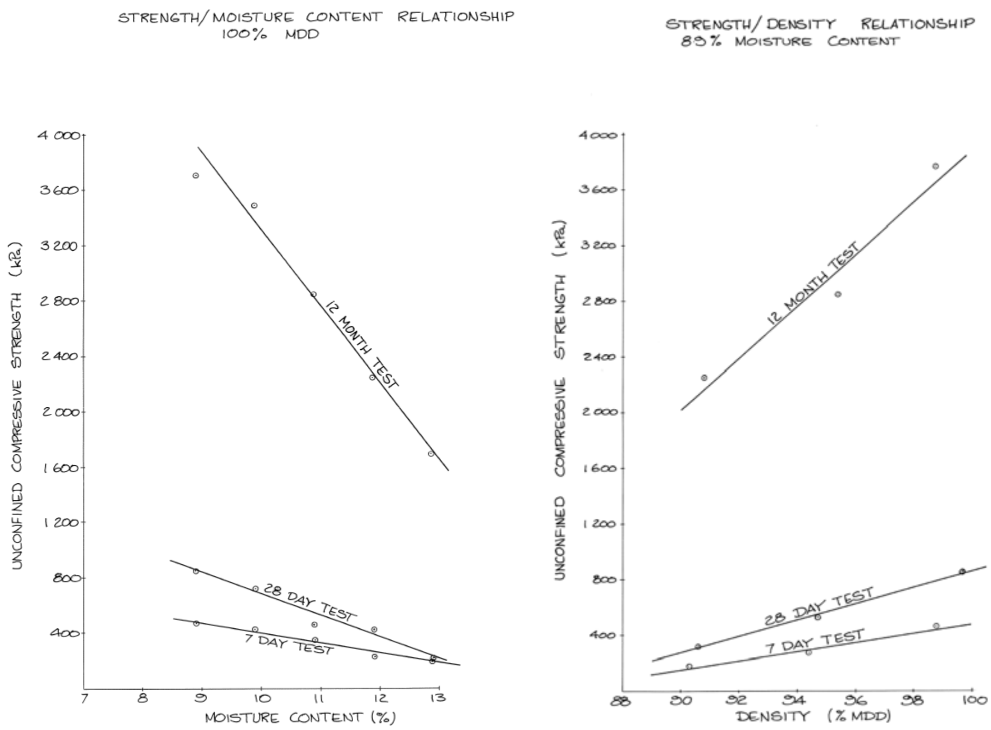

Figure 2. Relationship Between UCS vs Moisture Content and Density (Butkus 1985)

With the negative impact of moisture content and the positive impact of moisture content, the optimum moisture content corresponding to the maximum dry density is ideal for the preparation of cement treated basecourse. A compaction test is therefore undertaken as part of this study to determine the maximum modified dry density of the material in accordance with Test Method WA 133.1. The results of the compaction test area as shown below:

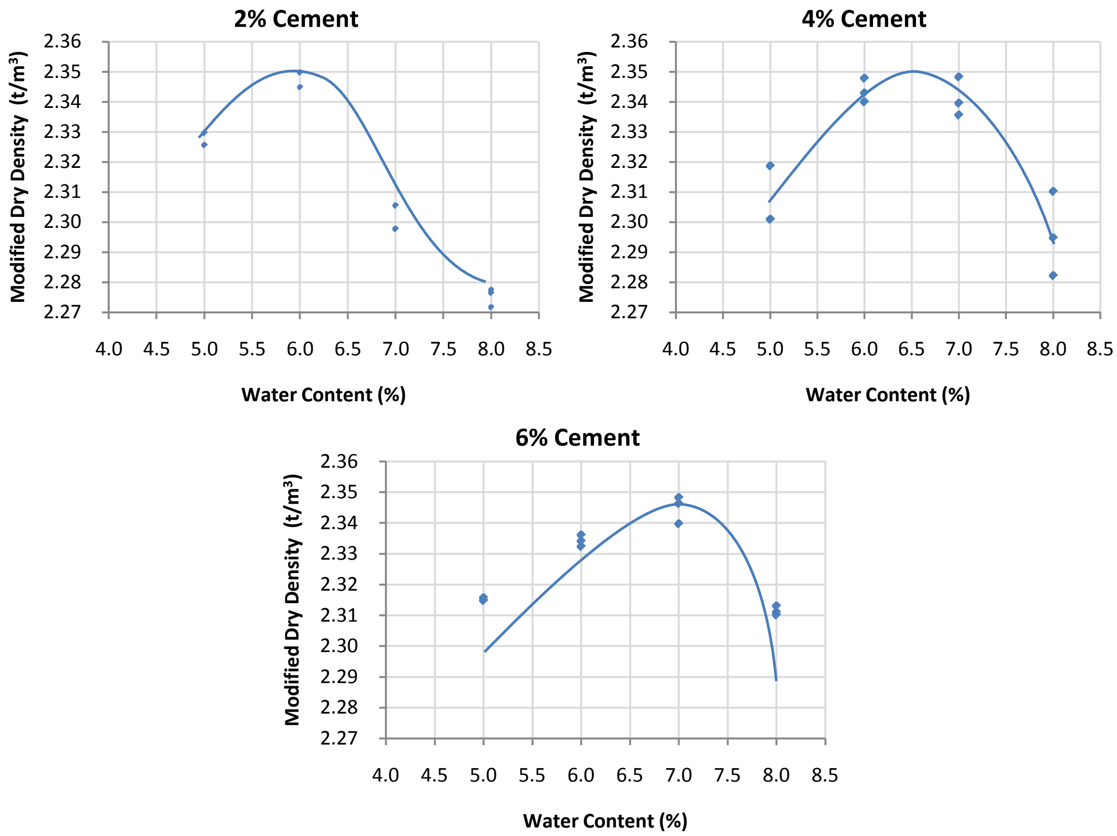

Figure 3. Compaction Test Results

As seen from the compaction test results, little variability in maximum modified dry density can be observed from varying cement content, where density can be estimated to remain constant at 2.35 t/m^3. However, the optimum moisture content corresponding to the tests show that the increasing cement content requires an increase in water content to achieve maximum compaction.

This can be potentially explained with the conglomeration of fines which affects the void ratio of the mix and the absorption of cement during the reaction process.

Another explanation of this is that the free water available within the voids are consumed by the hydration process of cement. By interpolating the results from the compaction tests, the increase in OMC is 0.25% for every 1% of cement as summarised in Table 1 below. This corresponds well with the minimum required water cement ratio as discussed earlier in this section.

Table 1. Relationship between Cement Content, OMC and W/C Ratio

Cement Content	1	2	3	4	5	6
Optimum Moisture Content (MMDD = 2.35 t/m^3)	5.75	6.00	6.25	6.5	6.75	7.00
W/C Ratio	0.17	0.33	0.48	0.62	0.74	0.85

2.2 Cement Content and Performance Measure by Unconfined Compressive Strength

The cement phase within CTB forms an interlocking matrix between the aggregates and binds the aggregates together. This process causes the development of tensile strength which in turn gives added flexural stiffness to pavements, minimising permanent deformation. The minimum practical cement content that should be treated to soil is to be 1% (Auststab) to ensure consistency of the material mix and ranges to typically 5%.

The Unconfined Compressive Strength is widely accepted as the classification criterion for cemented materials within the transportation industry because of its relative ease and speed to undertake (Vorobieff 2002). Although, it does not provide any input to design but have shown some relationship with various mechanical properties of CTB, the Unconfined Compressive Strength (UCS) provides an indicative measure of the normal stress and cohesive shear strength of the cement matrix, which is an expression of the degree of binding achieved from the mix design. Furthermore, the compressive strength is also used to categorise conventional concrete. Typical UCS and compressive values of these different categories are presented in Table 2 below.

Table 2. Compressive Strength Criteria of Different Classification of Cemented Basecourse

Classification	Testing Criteria	Source
Modified	0.7 MPa < UCS < 1.5 MPa	Austroads (2010)
Lightly Bound (Stabilised)	1.5 MPa < UCS < 3 MPa	Austroads (2010)
Bound (Stabilised)	UCS > 3 MPa	Austroads (2010)
Lean Mix	6 MPa < f_{cm} < 15MPa	DTMR (2009)
Conventional Concrete	f_{cm} > 20 MPa	Australian Standard (2009)

The UCS of CTB has been measured by this Author and published in other publications. The UCS results of CTB are repeated in Table 3 below for completeness with an indicaiton of the classification of the material based on Table 2:

Table 3. Unconfined Compressive Strength (UCS) of Tested Cement Treated Crushed Rock

Cement Content	2	3	4	5
UCS (MPa)	4.85	6.08	6.71	7.42
Classification	Heavily Bound	Heavily Bound	Heavily Bound to Lean Mix	Lean Mix

3 Mix Design Chart

The interaction of the various parameters discussed in this paper highlights the interdependency and the conflict of the mixes, resulting in proper manipulation of the mix design an arduous task. Figure 4 below i s b uilt up on t he f actors di scussed i n t his p aper an d pr esents t he different c ategories of materials achieved from varying cement content, water content and to a lesser extent the compaction effort. The Figure is adapted from Thom (2010) and PCA (2005).

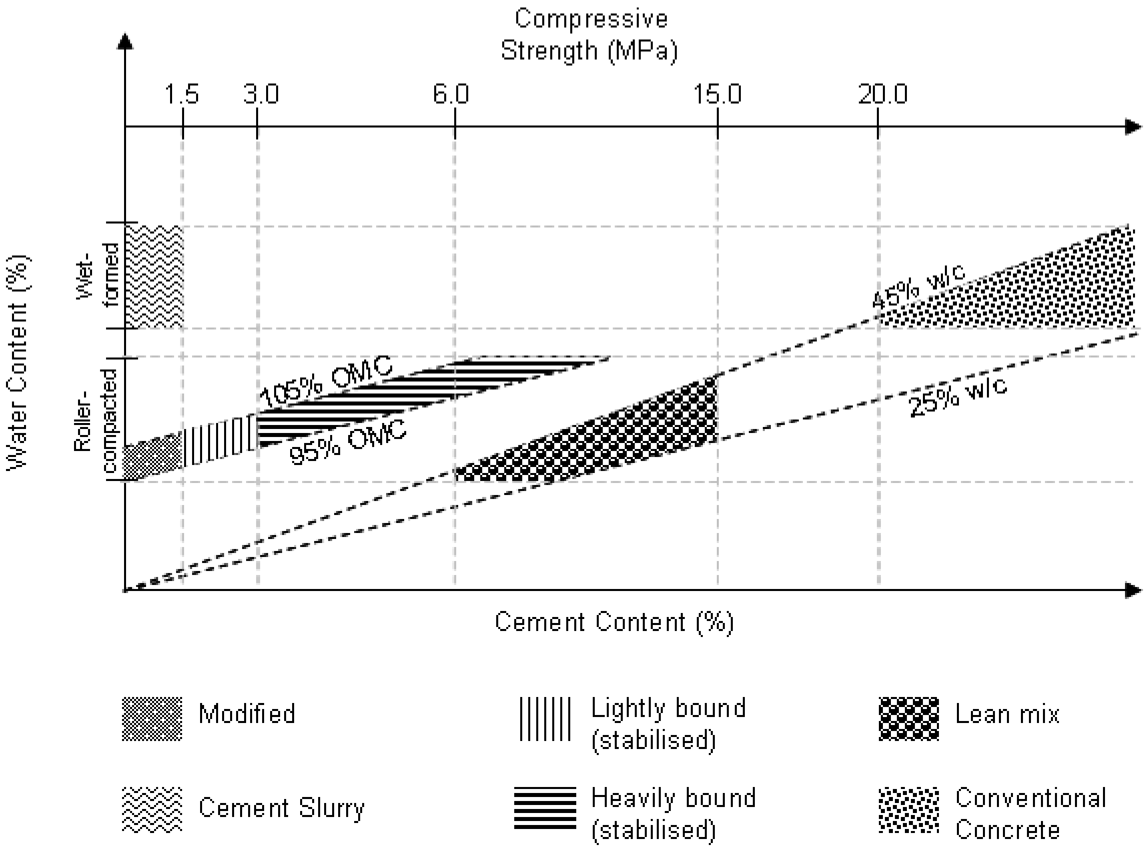

Figure 4. Mix Design Chart for Cemented Materials

The required water content to achieve compaction for the tested material means that the cement are overly saturated, porous and weak in strength, a specific trait that differs cement treated basecourse (CTB) f rom c onventional concrete. H owever, s ince the predominant r esistance of t he m aterial i s gained from the shear resistance of the aggregates, the cement matrix that forms around the aggregates provides added resistance to the

4 Conclusion

Several c onclusions c an be dr awn f rom t he s imple ana lysis and l iterature s tudy undertaken i n t his paper:

1. CTB are a unique material which oversaturates the material in terms of water required for cement hydration which will increase the porosity of the cement matrix and thus reducing strength. However, pavement shear strength are dictated by density and therefore the OMC is to be used for optimum performance and workability when using CTB.

2. The OMC for compaction of CTB is the OMC of the parent material + 0.25% for every 1% in cement content. This relationship runs parallel to the minimum water required for effective hydration to take place, i.e. a w/c ratio of 0.25. It is believed that is occurs due to the absorption of water by the cement paste for hydration and the reduction in fines due to the conglomeration of fines within the cement matrix.

3. The different mixes and use of soil-cement can be shown graphically as per Figure 4.

4. The cement treated crushed rock basecourse available in Western Australia falls within the highly stabilised to lean mix region.

5. Further work should be commissioned to determine the water content limits for roller compaction and wet forming. A slump test was initially planned as part of this paper but did not provide conclusive results. It is recommended that past experiences be drawn for this purpose.

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Erodability of Stabilised Pavements Using the Wheel Tracking Test

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Synopsis: An estimated total of $1.5billion dollars is spent on road rehabilitation in Australia per annum. This invokes a sustainable urgency to ensure reasonable service life is achieved for pavements. Cement treated basecourse provides a strong support to the pavement and is deemed as an alternate solution to reduce maintenance requirements of unsealed roads whilst minimising the generation of dust. When cemented basecourses are used for unsealed roads, Its primary purpose is often to maintain serviceability in lieu of sustaining heavy traffic loads, thus the vulnerability to erosion dictates its service life. In Australia, the study of erosion due to tyre loading on cemented pavements and its testing methodology thereof are very limited. The Cooper Wheel Tracking Test typically used for asphalt rutting testing is carried out to determine the Erodibility Index of cement treated crushed rocks. Results indicate that the increase in cement content increases the Erodibility Index. A proposed testing methodology for stabilised basecourse is ultimately derived from the investigation.

Keywords: unsealed pavements, stabilisation, dust control, erodibility, wheel tracking test.

1. Introduction to Unsealed Roads

Due to the geographical vastness of Australia and marginally low population densities in rural locations, unsealed roads form approximately 500,000 km, which represents 65% of roads in Australia (ARRB 1993). Furthermore, the development of the Australian commodity sector also extends the requirements of unsealed road networks to be developed to access remote areas.

The network of unsealed roads comprise of built up gravel roads, graded tracks or unformed roads on natural surface. Due to nature of these roads, more than $1 billion each year is spent on the construction and maintenance of unsealed roads (ARRB 1993), invoking a sustainable urgency to ensure reasonable service life is achieved for these pavements.

Defects requiring maintenance works of unsealed roads are generally categorised as either surface or structural. Structural defects involve failure of subgrades which result in permanent deformation of the road. On the other hand, surface defects include corrugations, potholes, slippery surface, rutting, ice formations, scouring, loose material and loss of surface material (ARRB 1993), which are generally localised on the surface of pavements and typically treated with re-grading works.

Furthermore, a critical issue with unsealed roads is the generation of dust. The generation of dust is a critical environmental issue, severely reduces visibility of trailing vehicles, increases wear and tear of vehicles and is detrimental to health.

2. Stabilisation of Unsealed Pavements with Cement

With the issues highlighted above, treatment of unsealed pavements in the form of stabilisation techniques is used typically to improve their serviceability. However, the stabilisation philosophy of unsealed pavement in the past had generally been to avoid the use of cement binders as it is not compatible with the maintenance regime typically applied for unsealed pavements. Cement stabilisation results in stiff bound surfaces which disallows routine grading and periodic shaping to be undertaken (ARRB 1993).

However in recent times, the use of cement and slag blend as a stabilisation option in rural Australia is gaining momentum due to its ability to minimise dust generation, reduce development of material sources and considerably decrease maintenance frequency on unsealed low traffic roads (Auststab 2009), potentially reducing their whole of life cost. In New South Wales, 5 unsealed pavement trial sections of various stabilising agents were constructed with promising results (Auststab 2009) as summarised in Table 1 below.

Table 1. AustStab Unsealed Pavement Trial

Road Name	Town	Reference Density (t/m^3)	Stabilisation Agent Tested
Barber	Griffith	2.2	Quicklime
Woodlands	Wombat	2.2	Cement/slag blend (70:30) and polymer based binder
Old Corowa	Jerilderie	2.05	Cement/slag (80:20)
Four Corners	Jerilderie	-	Quicklime
Back Mimosa	Temora	2.09	Quicklime

All stabilised unsealed pavements trialled showed adequate performance in wet weather conditions except the polymer based binder which became too slippery when wet. The cost per kilometre of stabilisation worked out to be $22,500 to $39,000 (AustStab 2009).

Since the issues associated to use of bound pavements for low volume roads, i.e. fatigue cracking is avoided, the performance criteria of stabilised unsealed pavements are therefore its durability and its propensity to generate dust, both manifested as the erodibility of the pavement.

3. Erodibility Index and the Wheel Tracking Test

De Beer (1989) undertook a comprehensive review of testing methods available at the time to assess pavement erodibility and durability (Scullion et al. 2005). In his study it was recommended that the use of South African Wheel Tracking Test (SAWTET) was deemed to be a more representative testing method for lightly cemented basecourse materials due to its ability to model in situ distress mechanisms experienced by thin sealed pavements (Guthrie et al. 2001) and more specifically unsealed pavements in the context of this paper. It was proposed that an Erodibility Index was to be used as an empirical quantification of the propensity of particulates of a surface to erode and is expressed as a depth of erosion caused by the SAWTET apparatus after 5000 passes (De Beer 1989).

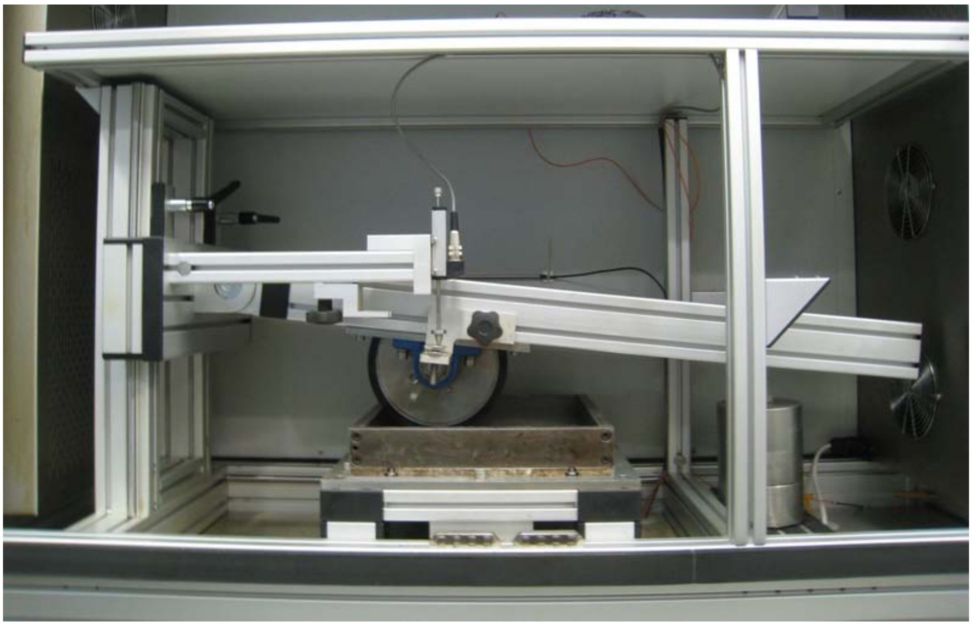

Figure 1. Cooper Wheel Tracking Test Device

Similar test setups emulating the concept of SAWTET also exist for asphalt testing to measure the rutting resistance of asphaltic seals. The Cooper Wheel Tracking Device is the most widely accepted asphalt tester in Australia (Austroads 2006) and is part of the repertoire of testing apparatus available at Curtin University's Pavement Research Group. The Wheel Tracker Test uses a reciprocating table which travels 230mm on linear bearings at a specified speed. The tests specimen is then placed on the bed with a rubber tyre wheel connected to a transducer resting onto the specimen. The typical setup is shown in Figure 1 above.

The Wheel Tracking Device is used to assess Erodibility Index of cement treated crushed rock available in Western Australia.

4. Sample Preparation and Testing Procedure

Crushed rocks sourced from Western Australia which meets Main Roads Western Australia Specifications 501 for aggregates are used for this experiment. The crushed rocks sourced are widely used in Western Australia as basecourse material. Cement Type General Purpose (GP) conforming to Australian Standard AS3697 is used for stabilisation.

The Cooper Compactor as shown in Figure 2 was utilised to create slab specimens measuring 305mm wide x 305mm long x 50mm deep. First the volume required to create the slabs were ascertained. The cement and aggregates were first dry mixed before water is added.

Specimens are compacted to a target modified dry density 2.35 t/m^3 at optimum moisture content.

The specimens were than spreaded evenly onto the mould and loaded onto the compactor which applied roller compacting actions at 3 pressure settings of 7 kPa, 12 kPa and 15 kPa for 10 times respectively. The specimens are weighed before and after compaction to ensure the target density is reached.

Figure 2. Cooper Compactor used to prepare slab specimens

The specimens are then wrapped cured for 7 days with a damp cloth in a sealed bag to promote the hydration of the specimen throughout the curing process. The specimen was then soaked for 12 hours prior to testing.

The Wheel Tracking Test was allowed to run up to 5000 passes with an applied total load of 700N comprising of a surcharge load of 180N and the wheel load of 520N. The average depth erosion at the centre 50mm span of the specimen is used to determine Erodibility Index.

5. Results and Analyses

A typical profile of the eroded surface at completion of 5000 passes for a 6% cement content slab is shown in Figure 3 below.

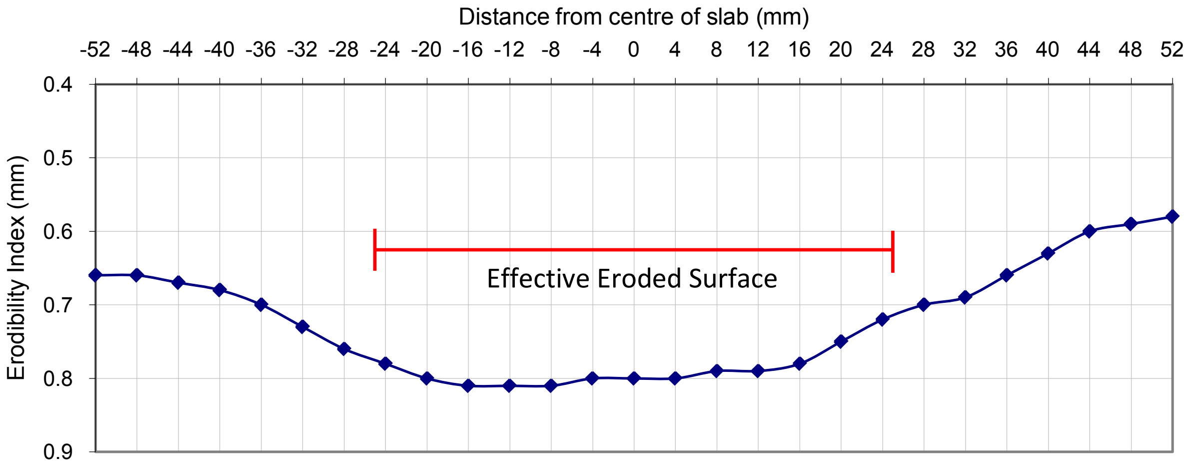

Figure 3. Typical profile of cement treated crushed rock slab after 5000 runs

As shown from the profile of the slab surface in Figure 3 above, where 0 mm represents the centre of the s lab, the m ost s evere er osion i s experienced on t he c entre of t he s lab. T his s upports t he methodology applied in this study whereby the Erodibility Index is determined from the centre 50mm of the slab.

The diminished erosion experienced towards the edge of the slabs is perceived to be caused by the deceleration of the wheel tracker. Surface inspections after each test were also undertaken to ensure that the readings are not distorted due to any deposits of large aggregates on the surface.

By taking the average of erodibility against number of runs for all specimens, Figure 4 showing the development of erosion can be created as shown below in Figure 4.

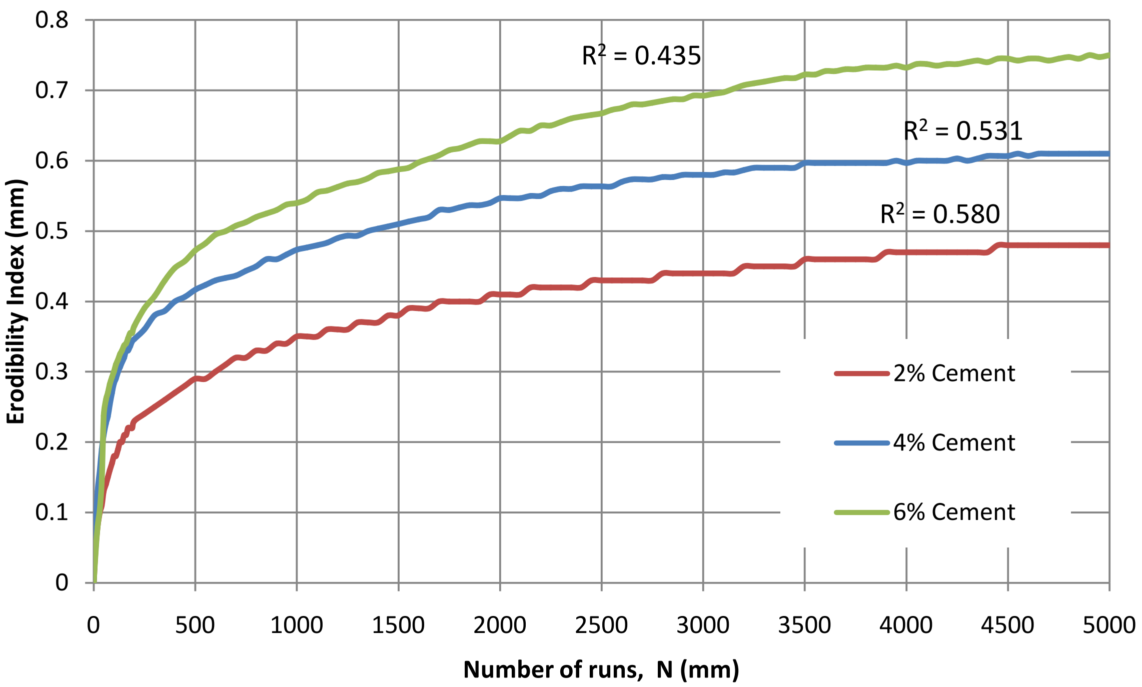

Figure 4. Erodibility Index vs. number of runs for various cement content

The Erodibility Index of the specimens, taken as the maximum eroded depth after 5000 passes, obeys the power law which is characterised with a sharp initial increase before achieving a resilient state with minor addition of erosion.

The Erodibility Index also shows a positive linear relationship with cement content. The Erodibility Index increases by approximately 0.1mm for an increase of 2% cement content. This suggests that the increase in cement content would result in faster surface deterioration of unsealed roads. An explanation of this observation can be traced to the change in water cement ratio. The water cement ratio from 2% cement content to 6% cement content decreases from 3 to 1.17, which potentially mean that the cement paste develops a higher propensity to migrate to the base of the slab during curing periods. As this occurs, less cement paste are being exposed on the surface. This is supported with visual observed of specimens prepared during the tests. The increased cement content showed a more pronounced concentration of cement paste on the surface of the slab. Figure 5 below shows a typical finished surfaced of a 6% cement content slab.

Figure 5. Typical Surface Depression After 5000 Runs

6. Limitations of the Erodibility Test

A cause for concern for the analysis undertaken is the relatively low least square regression achieved from the analysis, i.e. $R^2 \approx 0.5$. This clearly indicates that there is some variability with the results.

This is likely to be caused by the limitations of using the Cooper Wheel Tracking Test. The machine does not provide control measures to maintain the moisture of the specimens throughout the test unlike the South African Wheel Tracking Test used by De Beer (1989). Also, the temperature control system blew directly onto the specimens which caused expedited drying of specimens. As a result, specimens undergo significant fluctuations in moisture content, especially on the surface, throughout the test.

Furthermore, due to delays between testing and handling of specimens, the soaking period was varied ± 2 hours which resulted in a trend with Erodibility Index. Preliminary observations showed that the moisture condition at test significantly impacts the Erodibility Index. These observations however are still premature and will not be reported in this text.

7. Conclusions

The Cooper Wheel Tracking Test is capable of providing an indicative measurement of Erodibility Index for Cement Treated Basecourse. The Erodibility Index should be measured from the centre 100mm of the slab where the erosion is most critical. The Erodibility Index can be used as a design criteria for unsealed pavements or pavements with thin seals.

The Erodibility Index increases linearly with cement content and the development of erosion. This suggests that by reducing cement content a reduction of erosion and the generation of dust can potentially be realised.

However, there are limitations to the tests and modifications for the Cooper Wheel Tracking Test Device can be investigated to maintain moisture content throughout the test.

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CEMENT STABILISATION OF ROAD BASE COURSE: A CHRONOLOGICAL DEVELOPMENT IN WESTERN AUSTRALIA

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ABSTRACT

The use of cement stabilisation technologies was first pioneered in Australia as early as the 1950s. However, its use did not spread to Western Australia until the 1970s when an investigation was commissioned by Main Roads Western Australia (MRWA) to assess an alternative for bitumen stabilisation of base course. Cement stabilisation was identified to be a potential option as strength parameters of cement stabilised pavements were comparable to those of bitumen stabilised pavements. This paper presents the development of cement stabilisation technology from the laboratory investigation in 1970s, through to the construction of Leach Highway Trial Pavements, followed by the extensive laboratory testing by MRWA, and finally to the construction of Reid Highway Trial Pavements. In these investigations, cement has been assessed for its use as a stabilisation agent and subsequently a modifying agent and was found to be problematic, i. e. cement stabilised pavements exhibited issues of binder permanency, fatigue and shrinkage. As an immediate reaction to the observations of these investigations, MRWA prohibits the use of any appreciation in pavement strength from cement treatment when designing pavement thicknesses as covered in the current Engineering Road Note 9 (ERN9) 2010. A review of these events leading up to the publication of ERN9 2010 is presented along with testing regimes identified to close the knowledge gap.

1 INTRODUCTION

Cement became a mainstream stabilising agent in eastern Australian states as early as 1950 through the establishment of a specialist contractor, leading to the construction of in situ stabilisation of local government roads in the 1960s (Vorobieff 1998; Wilmot 1996). Its use continues to the present day and it is recognised as a potentially cost effective solution for rural road construction (Austroads 2010). Despite the myriad of documentation surrounding the development of stabilisation in eastern Australian states, literature on the development of cement stabilisation techniques in Western Australia that is available in the public arena is limited.

This paper therefore presents a chronological review of Main Roads Western Australia (MRWA) experiences based on literature on cement stabilisation in Western Australia which includes technical reports, reports on trial pavements and performance reviews. This is followed by a critical review of this development against the current design methodology adopted by MRWA.

2 CEMENT FOR STABILISATION

With the success of cement stabilisation in Eastern States, MRWA initiated extensive research works to assess its viability in Western Australia. This included laboratory investigations followed by the construction of Leach Highway Trial Pavements as discussed in subsequent sections.

2.1 INITIAL LABORATORY INVESTIGATION OF STABILISATION OPTIONS

The application of 3% bitumen for base course stabilisation was typically adopted in Western Australia (Hamory 1980) in the 1970s. However with increasing prices of bitumen, more economical options were sought. Subsequently in 1975, a detailed laboratory investigation of limestone stabilised with bitumen and cement in Western Australia was instigated by MRWA (Hamory 1977).

The investigation involved testing specimens treated with varying amounts of cement and bitumen ranging from 1% to 6% each. Specimens are prepared to 100% Modified Maximum Dry Density (MMDD) at 96% Optimum Moisture Content (OMC) and dried back to 70% OMC after compaction. The significant gain in strength and stiffness from relatively low quantities of cement treatment was noted from the study, where strength gained from 2% of cement exceeded that typically achieved for 3% of bitumen treatment (Hamory 1977; Hamory 1980). The results of this investigation are summarised in Table 1 below.

Table 1: Test results for 4 day soaked cement stabilised limestone

Cement Content (% by mass)	1%	2%	3%	4%	5%	6%
Peak Unconfined Compressive Stress (MPa)	0.755	1.702	2.642	1.860	3.760	4.940
Unconfined Compressive Modulus (MPa)	60	160	340	235	470	680
Tensile Strength (kPa)	68	151	252	329	593	569
Cohesion (kPa)	109	243	464	361	693	794
Internal Angle of Friction (^{o})	58	58	56	48	49	54
Western Australian Confined Compression Class Number	1.2	0.3	0.0	0.0	0.0	0.0

The observations from the laboratory study concluded that 2% cement mix shows some potential in being a superior road stabilising agent as strength parameters exceeded 3% of bitumen treatment. 3% of bitumen treatment typically achieved a Peak Unconfined Compressive Stress of 0.173 MPa and a Tensile Strength of 14 kPa (Hamory 1977). In addressing concerns with overly stiff pavements, field techniques were deemed a suitable option to mitigate issues with cracking. The 2% cement mix is thus assessed for its relationship to delay between mixing and compaction immediately after mixing and after 24 hours of mixing. Specimens are compacted to 100% OMC and tested immediately after curing without dry back. The results of this assessment are presented in Table 2 below.

Table 2: Strength parameters of cement stabilised limestone compacted at 0 and 24 hours delay.

Cement Content (% by mass)	2%							
Curing Time before Compaction (hours)	0				24			
Curing Time After Compaction (days)	0	7	14	21	0	7	14	21
Peak Unconfined Compressive Stress (MPa)	69	635	820	770	243	446	585	600
Unconfined Compressive Modulus (MPa)	5	56	80	57	23	28	34	27
Tensile Strength (kPa)	8	54	70	71	18	45	64	55
Cohesion (kPa)	11	89	115	116	32	67	92	87
Internal Angle of Friction (^{o})	54	58	59	68	61	56	55	58
Western Australian Confined Compression Class Number	3.6	1.4	1.0	1.0	2.4	1.8	1.6	1.4

The results show that a delay in compaction increases the initial strength of the material, which in turn suggests that field techniques may be employed to address cracking tendencies of stiff pavements (Hamory 1977). Due to the different specimen preparation methodology employed, the results of the tests also provide indication of the benefits of dryback practices. The results prompted a further need to assess in service conditions of cement stabilised limestone, which was subsequently realised through the construction of trial pavements on Leach Highway.

2.2 LEACH HIGHWAY TRIAL PAVEMENTS

Leading on from the preliminary laboratory investigation undertaken in 1975, trial pavements were constructed for the Leach Highway using 1% and 2% bitumen stabilised limestone base courses and a 2% cement stabilised limestone base course in 1977 (Hamory 1980). The pavement was designed based on the NAASRA 1977 pavement design procedures. Details of the trial pavements are as shown in Figure 1 below.

The trial pavements were assessed under comprehensive reports (Hamory 1980; Hamory 1981) for which the indirect tensile strength, field moisture and surface deflection were measured. The results of the comprehensive reports are presented in the subsections below.

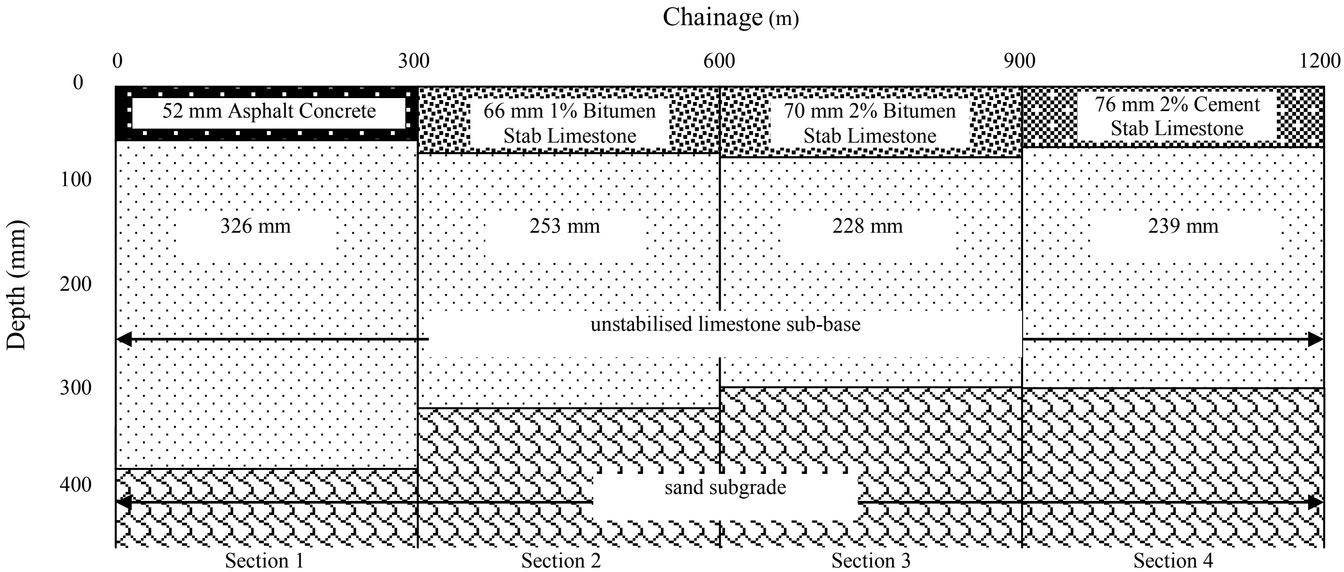

Figure 1: Profiles of Leach Highway trial sections measured in 1980/81 (Hamory 1981)

2.3 DEVELOPMENT OF TENSILE STRENGTH

Indirect tensile strength tests were undertaken from typically 25 random cores selected from the trial sections 2, 3 and 4 in each assessment period. These results are presented in Table 3 below (Hamory 1981). The asphalt concrete from trial Section 1 was not assessed periodically, but had its tensile strength measured on December 1977 to be 177kPa at 30°C and 39 kPa at 40°C

Table 3: Tensile Strength and Moisture Ratio of Leach Highway Trial Pavement

Date of Test	Section 2 (1% Bitumen Stabilised)		Section 3 (2% Bitumen Stabilised)		Section 4 (2% Cement Stabilised)	
	Tensile Strength (kPa)	Moisture Ratio (%)	Tensile Strength (kPa)	Moisture Ratio (%)	Tensile Strength (kPa)	Moisture Ratio (%)
December 1977	42	29	55	36	59	65
August 1978	72	20	113	23	-	-
November 1978	-	-	-	-	57	59
October 1979	36	34	52	39	26	66
December 1980	50	32	73	42	46	51

As the results indicate, a significant decrease in tensile strength of the all trial pavements was measured after 2 years in service (1979), which MRWA could not explain at the time (Hamory 1980). However, measurements undertaken in 1980 showed a rebound in tensile strength which attributed the change in the measurement to amendments in testing procedures (Hamory 1981). The tensile strength remained essentially stable after these measurements, as shown through ongoing monitoring (Hamory and Cocks 1988). Moreover, with the exception of section 3, tested specimens showed that by 1980 the tensile strength of more than 50% of the cores was lower than the minimum performance criterion at the time of 55kPa (Hamory 1980).

2.4 IMPLICATIONS TO DEFLECTION AND SERVICE LIFE

This lack of tensile strength was however dismissed, based on deflection monitoring using the Benkelman Beam. Deflections measured during the assessment of the trial pavements concluded that the deflections experienced by the four sections were similar, ranging from 0.06 to 0.10mm as shown in Table 4 below (Hamory 1981).

Table 4: ESAs and deflections of trial sections

Time of Measurement	Equivalent Standard Axle, ESA ($x10^6$)				Deflection (mm)			
	LH	Centre	RH	Total	Section 1	Section 2	Section 3	Section 4
30/10/1977	Construction Traffic Only				-	0.63	0.63	0.56
26/04/1978	0.08	0.09	0.04	0.21	0.42	0.48	0.50	0.54
15/05/1979	0.17	0.19	0.08	0.44	0.49	0.49	0.51	0.57
23/10/1979	0.26	0.29	0.12	0.67	0.51	0.45	0.54	0.55
15/04/1980	0.34	0.38	0.15	0.87	0.44	0.52	0.59	0.59
27/11/1980	0.42	0.47	0.19	1.08	0.47	0.43	0.42	0.47
30/04/1981	0.51	0.58	0.24	1.33	0.45	0.49	0.53	0.56

The results were then used to estimate the service life of the pavement based on NAASRA 1979 design guidelines, which provided the results presented in Table 5:

Table 5: Calculated pavement life

Section	NAASRA 1979 Deflection Design Line	Total ESAs in 1 Direction	Calculated Pavement Life (years)
1	2	$> 3 \times 10^7$	44
2	1	$> 3 \times 10^7$	44
3	1	$> 3 \times 10^7$	44
4	4	1.1×10^6	5

It was argued at the time that the compaction works during the construction of Section 4 were not completed until 24 hours after the addition of cement (Hamory 1980), which meant that a reduction of compressive modulus of 60% could have occurred (Hamory 1977) as seen in laboratory results presented in Table 2. This in turn implied that the NAASRA 1979 design line to be adopted could vary from 4 to 2, giving a calculated pavement life of approximately 20 years (Hamory 1980) rather than the 5 years calculated with Deflection Design Line 4. In comparing the design life and the deflections measured, the pavement was capable of providing adequate performance for the life of the pavement (Hamory 1980). It was later understood that the material had started to behave as unbound material due to extensive cracking (Hamory and Cocks 1988), a point discussed later in the subsequent section.

In validating the results, the Australian Road Research Board (ARRB) was also engaged to conduct test tracks to assess the four trial sections using a full scale test known as the Accelerated Loading Facility (ALF). The assessment ranked limestone stabilised with 2% cement as the best performing base course material (Sparks and Hamory 1980) especially in poorly drained areas (Hamory and Cocks 1988).

Further to these measurements, initial observations carried out during the construction of the trial pavements also pointed out that the cement treated limestone exhibited adequate strength with high CBR values that were double those of bitumen treated sections (Hamory 1980).

2.5 THE REALISATION OF FATIGUE CRACKING

The first appreciation of issues pertaining to materials with high stiffness was reported by Hamory (1977) during preparation of a cement treated limestone specimens for laboratory investigations. It was noted that a potential risk of cracking exists for cement stabilised limestone base course due to the stiff behaviour of the material. However, its correlation to fatigue phenomena was not yet established.

It was only in the mid 1980s did the concept of fatigue cracking of bound material was introduced to pavements in Western Australia. The concept was substantiated by the difficulty experienced when obtaining intact cores from cement stabilised base courses from Leach Highway trial pavements, an observation attributed to the development of extensive microcracks at the time (Cocks 1987; Hamory and Cocks 1988).

Moreover, measurements of trial pavements up to 1986 indicated that the calculated allowable ESAs for subgrade deformation based on NAASRA 1986 design charts is 0.75×10^6, which contradicts observed deflections and in service road conditions. The combined observations suggested that the design life calculations used were incompatible with the material in-situ behaviour of the material. The following conclusions were also drawn:

- the cement stabilised material had undergone extensive fatigue cracking and was then acting in "blocks" of unbound granular material, which when calculated as such would provide a more realistic allowable ESAs limit; and
- the subbase characteristics of Leach Highway were not compatible with the design equations used by NAASRA.

In 1987 through the documented work of Sales (1987) and Cocks (1987), the dependent relationship between the bound behaviour of materials and fatigue was established. The development of CIRCLY and the inclusion of a fatigue criterion in NAASRA 1986 design guidelines meant that a mechanistic analysis of pavements with bound layers was able to be assessed. Such an assessment was undertaken by MRWA to back-calculate the load-deflection relationship measured from the trial sections on Leach Highway. It was concluded from the report that the NAASRA methodology was "dubious" and did not provide any conclusive relationship (Sales 1987). Furthermore, Sales (1987) noted that the inconclusiveness is likely to be caused by the sand subgrade underlying Leach Highway providing adequate support for the stabilised layer "blocks".

In the same year, the *Pavement Design using Bound (Stabilised) Materials* guideline was developed by MRWA (Cocks 1987). In the guideline, the post cracking phase of pavements was introduced, and a suggestion that in scenarios when the failure criterion for fatigue is not specified, the base course shall be considered as unbound granular material for the pavement design.

3 CEMENT AS A MODIFICATION METHOD

Cement modification was not a new technology in Western Australia. Cement treatment of limestone base course was used in Western Australia as a modification method to reduce the moisture sensitivity of base course through the lowering of Plasticity Index (PI) and Linear Shrinkage (LS), for constructing floodways and other moisture sensitive structures.

For example, cement treatment of the gravel base course was undertaken as part of the Great Northern Highway at Sandfire (Hamory 1979). Samples collected from the constructed pavements showed that the PI was reduced by 44% in the windrow and 60% in pavement samples and the LS was reduced by 33% and 52% respectively. The difference was associated with the non-uniform distribution of cement treatment and the limitations of the testing methodology.

The use of cement as a modification method was later brought into the consideration due to the limitations of its use as a stabilisation agent. The development of cement modification is presented in the subsequent sections.

3.1 KWINANA FREEWAY AND CRUSHED ROCK BASE COURSE

In the early 1990s, the use of crushed rock conforming to the Specification 501 (MRWA 2011) as a base course material was increasing. However, during the construction of Kwinana Freeway, several sections were noted to have failed between Yangebup Road and Farrington Road, Welshpool Road and elsewhere (Watson 1995). The failures of the roads were associated to the sensitivity of the crushed rock base course to moisture. This prompted an urgent need to better understand the behaviour of crushed rock base course.

An extensive testing program was thus initiated by Main Roads Western Australia to analyse the response of crushed rocks to varying conditions of moisture, compaction, and modification techniques (Watson 1995). Among the investigated modification methods was the use of cement.

As part of the work by MRWA to investigate crushed rock base courses, the effects of density and moisture content on resilient modulus were assessed as a primary objective. This was achieved by testing samples at varying densities and moisture contents, which included specimens prepared to 100/80, 98/60, 98/50, 96/80 and 96/60 (dry density ratio/moisture ratio) for modified compactive effort to represent the in service conditions of base courses.

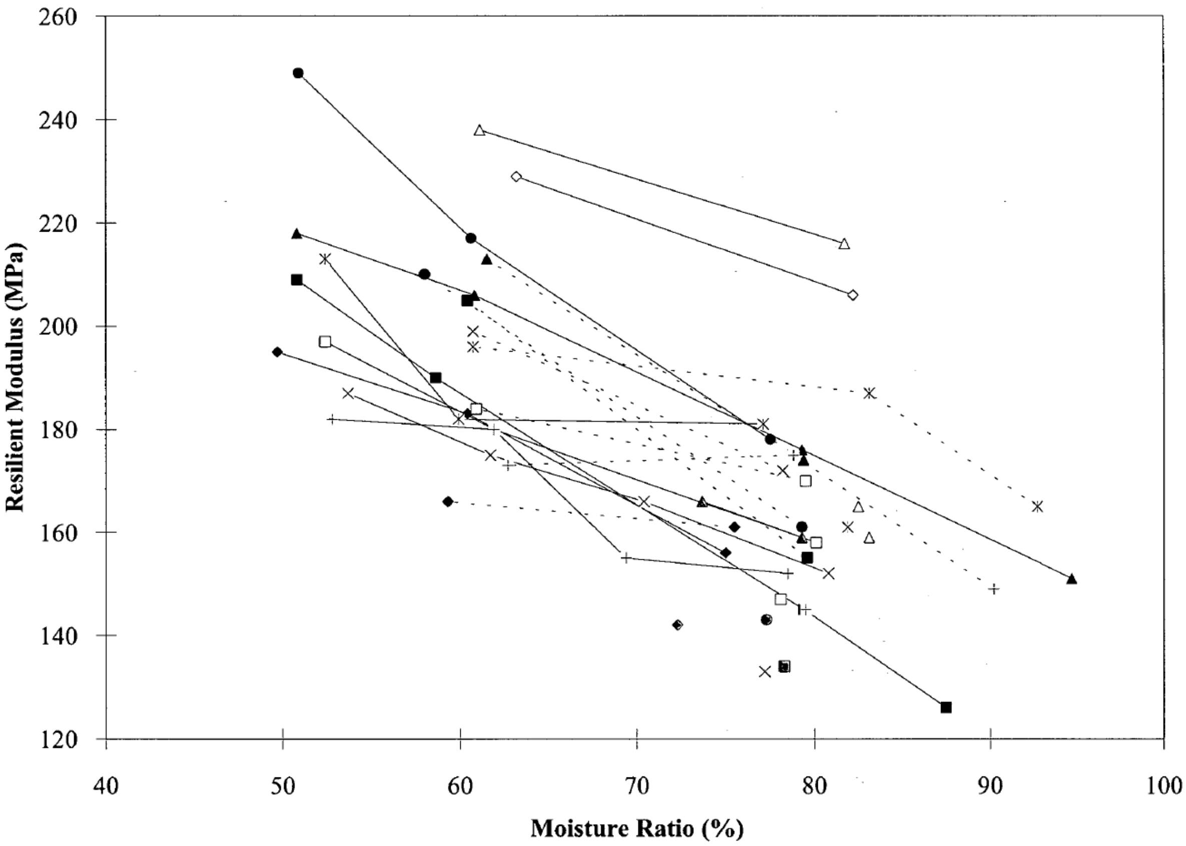

Figure 2: Relationship between resilient modulus and moisture ratio (Butkus and Lee Goh 1997)

As seen from the results in Figure 2, crushed rock base courses are highly sensitive to moisture ingress where moisture ratios (moisture content / optimum moisture content) in excess of 60% may result in poor performance due to a reduction in resilient modulus (Butkus and Lee Goh 1994). Moisture contents for materials are typically 5.9% by mass. This finding was also supported from results obtained from tests on in-situ crushed rock base courses. Following this, further tests were undertaken to assess the sensitivity of crushed rock base courses treated with cement.

The cement treated crushed rock was tested for its performance as a function of various cement contents (0.5%, 1%, 2% and 3%), cement set time prior to compaction, curing time and a Hydration Test. The Hydration Test was designed by MRWA to assess whether part of the improvement of the crushed rock base was due to factors other than the cementation process (Butkus and Lee Goh 1994), e.g. (Watson 1995). The test involved an interference of the cementation process by regularly remixing the material prior to compaction. These properties and the conclusions from Butkus and Lee Goh (1994) are shown in Table 6 below

Table 6: Observations of cement modification on crushed rock base course properties

Property	Cement content (%)	Observation when Property Value is Increased
Resilient modulus	0.5, 1, 2 and 3	• Increased performance generally (increased resilient modulus, strain rate and permanent strain) • Reduced sensitivity to moisture
Cement set time	2	• Decreased performance
Curing Time	2	• Increased performance • Reduced sensitivity to moisture
Hydration Test	2	• Reduced sensitivity to moisture

From the test results shown in Table 6, it was concluded that treatment with cement generally provided better base course performance under repeated load and reduced sensitivity to moisture. However, Unconfined Compressive Strength (UCS) tests undertaken by Lee Goh (1994) suggested that crushed rock base courses will behave as stabilised (bound) materials when as little as 1% cement is applied, purely because the measured UCS is more than 1 MPa. This implied that the material will undergo fatigue damage and shrinkage cracking, characteristics unfavourable to the road construction strategy for Western Australia.

More significantly, the disturbed product of the hydration test led to the development of the Hydrated Cement Treated Crushed Rock Base (HCTCRB). HCTCRB is a modified material which is produced by remixing stockpiles of cement treated crushed rock to physically break the cementitious bonds. The 2% HCTCRB was deemed at the time a superior mix design as it exhibited an improved performance with respect to the influence of moisture ingress without the development of undesirable bound material behaviour.

As a conclusion to the program, modification of crushed rock base courses through treatment by cement less than 1% or by HCTCRB were deemed as potential options to reduce the moisture sensitivity of crushed rock base course. These options were tested through trial pavements constructed on Reid Highway.

3.2 REID HIGHWAY TRIAL PAVEMENTS

The Reid Highway Trial Pavements were constructed as an outcome of the laboratory work as discussed in the previous section. Their purpose was to investigate modified base course materials, with a specific focus on HCTCRB. The trial pavements consisted of 9 sections located between West Swan Road and Bennett Brook Bridge in Caversham, totalling approximately 860m in length (Butkus 2004; Harris and Lockwood 2009). Table 7 summarises the pavement details.

Table 7: Reid Highway trial sections base course material types and thicknesses

Section	Modified Base Course Material	Measured Depth (2009)
1	2% HCTCRB	123mm
2	2% Bitumen Stabilised Limestone	113mm
3	CRB	90mm
4	CRB	211mm
5	1% HCTCRB	210mm
6	2% HCTCRB	211mm
7	0.75% GGBFS Stabilised CRB	231mm
8	2% GGBFS Stabilised Limestone	182mm
9	LIMUD	214mm

GGBFS = Ground Granulated Blast Furnace Slag
HCTCRB = Hydrated Cement Treated Crushed Rock Base
LIMUD = Lime stabilised base course
CRB = Crushed rock base course

Detailed information regarding the trial pavements are reported at two major reporting periods by Butkus (2004) and Harris and Lockwood (2009). In summary, the Reid Highway trial pavements gave rise to the following summary observations pertinent to the two cement treatment options (Harris and Lockwood 2009):

- Section 7 showed that pavement with low cement contents suffered from non-homogenous distributions of cement and large initial deflections, before returning to the performance levels of untreated CRB
- HCTCRB Sections 1, 5 and 6 initially showed good improvement with respect to moisture sensitivity but later assessment showed that they returned to the performance levels of untreated CRB
- The binder contents of Sections 1, 5, 6 and 7 were noted to have “disappeared”, potentially due to carbonation as shown in the carbonation test results in Table 8 below
- The deficiency in later pavement performance for Sections 1,5, 6 and 7 could not be attributed to either loss of stabilisation or to fatigue cracking

- Transverse cracks were observed on the centreline and shoulders of HCTCRB sections and were believed to be shrinkage cracks caused by the presence of the relatively high cement content of 2%. Refer Figure 3 below.
- Limestone stabilised pavements as tested in Section 8 showed high curvature which suggested that limestone pavements are incompatible with cementitious treatment
- Thicker pavements generally performed better in terms of Benkelman beam curvature

Table 8: Results of carbonation test for cement treated sections

Section		1		5		6		7	
Base course Description and Measured Depth		2% HCTCRB 123mm		1% HCTCRB 210mm		2% HCTCRB 211mm		0.75% GGBFS Stabilised CRB 231 mm	
Sample Chainage		11570	11600	10520	10550	10430	10460	10330	10360
Test Solution	Phenolphthalein	N	N	Y	N	Y	Y	N	N
	Phenol Red	N	N	-	N	-	-	N	N
	HCl Acid	Y	Y	Y	Y	Y	Y	Y	Y
Carbonation Result		Full	Full	Partial	Full	Partial	Partial	Full	Full

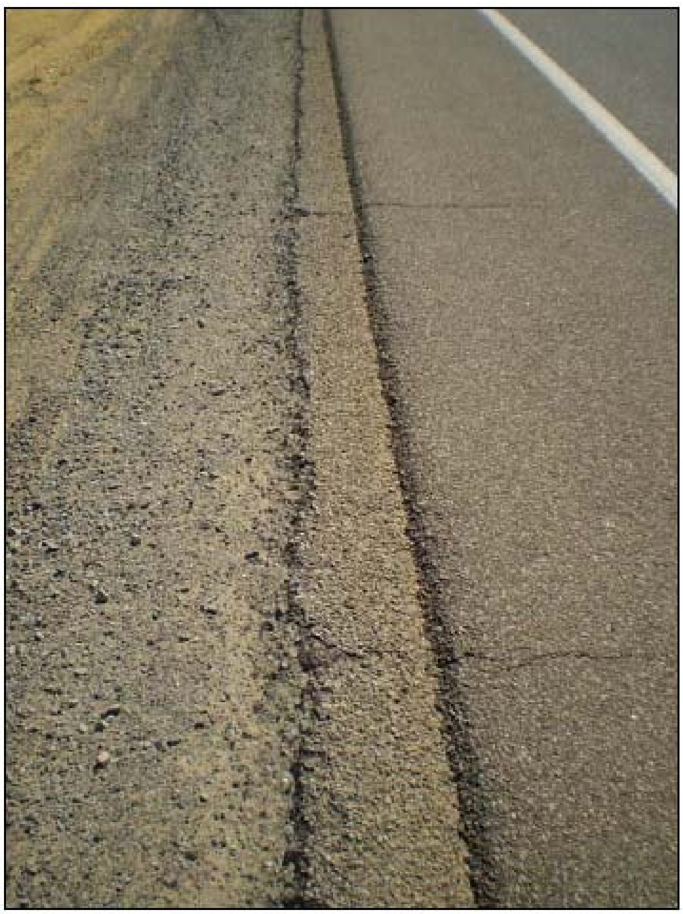

Figure 3: Transverse Cracking of Trial Pavements with HCTCRB Base Course

4 PAVEMENT DESIGN IN WESTERN AUSTRALIA - ENGINEERING ROAD NOTE 9

As a culmination of the laboratory tests and trial pavements, the current pavement design guideline released by Main Roads Western Australia, Engineering Road Note 9 (ERN9) - 2010 (MRWA 2010) states that:

Clause 1.1.8:

The pavement must not incorporate cemented materials.

Clause 1.1.8:

The pavement must not incorporate any modified granular material that satisfies one or more of the following criteria when tested at its in-service conditions: -

(a) 7-day unconfined compressive strength (UCS) of the material exceeds 1.0 MPa;
(b) 28-day UCS of the material exceeds 1.5 MPa; or
(c) Vertical modulus of the material exceeds 1500 MPa

Clause 4.2:

No reduction in thickness requirements can be made for pavements incorporating granular material modified with cement, lime, bitumen or other similar materials.

ERN9 asserts that the use of bound materials as a structural component is prohibited and modifications of unbound materials shall be limited to UCS values below those of bound materials. The guideline goes to the extent of requiring that even when modified materials are used, their strength gain is not to be considered as a structural improvement.

5 IMPLICATIONS AND DISCUSSION

5.1 DEFINING BOUND PAVEMENTS AND FATIGUE

The literature presented in this paper suggests that the current design clauses within ERN9 are immediate reactions to the observations from the trial pavements, with limited test data on the behaviour of bound materials.

The non-inclusion of cemented materials stems predominantly from UCS tests of 1% cement treated crushed rock samples by Lee Goh (1994) where materials exhibiting UCS values of more than 1.0 MPa were associated with development of fatigue cracking. Although this UCS limit complements the suggested bound behaviour definition provided by Austroads (2010), given the multitude of other factors that have yet to be understood, there is yet to be any conclusive evidence to suggest that fatigue and shrinkage become unmanageable issues at low cement contents.

Furthermore, the observations from the Reid Highway Trial Pavements which showed a reduction in stiffness, manifested as increased curvatures measured by the Benkelman Beam, were not conclusively associated with fatigue cracking. The curvatures are deemed to be either a result of fatigue cracking of the pavements or a loss of binder content. It is also important to note that the primary material investigated by the Reid Highway Trial Pavements is HCTCRB. There is some as to whether the issues related to fatigue cracking are relevant to the pre-disturbed cement matrix of HCTCRB.

With fatigue being the primary defining criterion for the limitation of the use of bound base courses as part of the road network in Western Australia, it may be prudent to reassess the defining criterion for bound material, especially when there is yet to be any recorded fatigue tests initiated by MRWA to confirm the UCS limits suggested in ERN9. Notwithstanding the fact that an overlap exists between the mechanistic behaviour of modified and stabilised materials, a more definitive delineation between modified and stabilised materials is required to efficiently design pavements rather than a blanket rule of non-inclusion.

5.2 BINDER PERMANENCY AND MOISTURE INGRESS

The prohibition of consideration of strength gain from base course modification to reduce the structural thickness of a pavement is related to the uncertainty associated with permanency of cement binder content. The "disappearance" of cement binder content of the modified pavements used in the past by MRWA is linked to the carbonation process of binders. Carbonation occurs as a result of the presence of carbon dioxide infused in water, where the hydrated cement paste undergoes a reversed chemical reaction and becomes dissolved into the water. When water enters pavements through groundwater intrusion, lateral seepage, or infiltration of surface water, carbonation occurs and the pavement returns to an unbound state. This carbonation and loss of permanency of binder content therefore limits the durability of the bound pavement.

Given that the driver for cement modification was to reduce the sensitivity of pavements to moisture content increases, having a non durable stabilisation effect means that stabilisation is not worthwhile. It would not only be detrimental to the integrity of the road, increasing the pavements' susceptibility to moisture content increases, but will also become a economic burden as the benefit from the cost spent for modification is not realised to its full potential.

The tests that have since been carried out by MRWA provide data on the structural performance of pavements after defined volumes of water have entered the pavements. The literature shows that those studies were focused on measuring the resilient moduli of materials as a function of various moisture ratios of pavement materials. The mechanism by which moisture may enter the pavement was not fully investigated. This presents as an opportunity for the design of laboratory tests to understand the mechanism of moisture ingress into a pavement base course and subsequently the rate of binder content carbonation.

5.3 SHRINKAGE AND TRANSVERSE CRACKING

Shrinkage cracking has been identified by MRWA as the cause of the observed transverse cracking on the centreline and shoulders of seals as per Figure 3. Although a plausible conclusion, again, no validation work has been undertaken to substantiate this. It is, however, recognised by the authors that there is limited technology available to confirm this conclusion.

It is reported in Harris and Lockwood (2009) that the cracking is found mostly on pavements in Section 6 - 2% HCTCRB, 211mm depth, at constant spacings of 2.5 to 3m. The consistent spacing of the cracks and their location do suggest that volumetric change may be the cause. Nevertheless, given the age of the pavement, the cracking is potentially caused more by fluctuations of moisture content arising from moisture intruding into the pavement from the edge of the pavement, rather than by shrinkage cracking from the cement hydration process.

Autogenous shrinkage, the process whereby water escapes from the pores within the cement, typically occurs within 10 to 15 days (Chakrabati and Kodikara 2005). With HCTCRB pavements, where the cement bonds are broken after 7 days of hydration, the influence of shrinkage on cracks generated after 9 years of service is debatable.

In retrospect, the suggestion by MRWA to increase the disturbance period for HCTCRB may hold some merit. However, further laboratory analysis should be undertaken to assess the product, considering that any appreciable modification may be destroyed from late re-mixing activities.

6 CONCLUSION

MRWA has been using cement as a stabilisation and modification binder since the early 1970s which can be summarised in Figure 4 below. However, there are still knowledge gaps with regards to the mechanistic behaviour of the material. Addressing these knowledge gaps is vital for the application of cement binder to road pavements in Western Australia.

In summary, there are a few key points that can be drawn from the literature study undertaken:

1) there is limited understanding of the fatigue behaviour of cement treated crushed rocks, and relying solely on UCS test results as a criterion without substantial laboratory or field results should be avoided

2) the mechanism of moisture ingress is critical in understanding the durability of cement modified and cement stabilised materials

3) limited studies on the effects of shrinkage and the effects of cement content on shrinkage are available.

These key points are being addressed by the authors through various research activities undertaken as part of doctoral research at Curtin University. This includes fatigue beam testing to assess fatigue behaviour of cement treated crushed rocks, Tube Suction Testing and the adoption of unsaturated soil flow theory to assess the mechanism of moisture ingress, and characteristic shrinkage tests through identification of pore structures.

The milestones for the development of cement treatment of road base course in Western Australia can also be summarised in Figure 4 below.

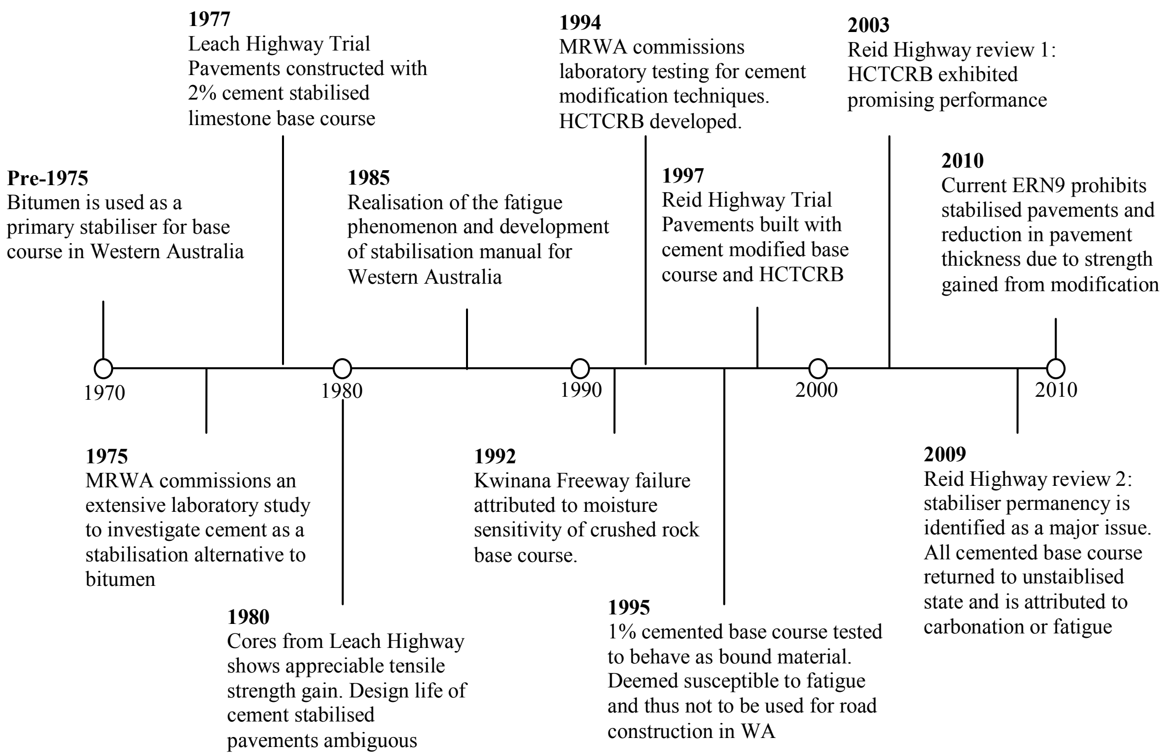

Figure 4: Milestone of cement treatment of base course in Western Australia

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APPENDIX B

Laboratory Data

APPENDIX B

B.1 Modified Proctor Compacation Test

CC (%)	2%	4%	6%	2% (ZAV)	4% (ZAV)	6% (ZAV)
WC (%)	MDD (t/m³)					
5	2.327	2.320	2.316			
5	2.330	2.303	2.316	2.450	2.476	2.497
5	2.327	2.303	2.317			
6	2.350	2.348	2.335			
6	2.350	2.343	2.337	2.391	2.415	2.435
6	2.345	2.340	2.323			
7	2.300	2.336	2.346			
7	2.300	2.340	2.331	2.334	2.357	2.376
7	2.307	2.348	2.348			
8	2.275	2.285	2.313			
8	2.279	2.297	2.314	2.280	2.302	2.319
8	2.280	2.312	2.312			
n	3	3	3			

MDD = modified dry density
CC = cement content
WC = water content
ZAV = zero-air-void

B.2 Unconfined Compressive Strength Test

CC (%)	2%	3%	4%	5%	CC (%)	2%	3%	4%	5%
ε	UCS (MPa)				ε	UCS (MPa)			
0.0000	0.0000	0.0000	0.0000	0.0000	0.0290	4.6419	5.4804	6.0400	6.1849
0.0010	0.0231	0.0277	0.0391	0.0391	0.0300	4.7611	5.5968	6.1968	6.3770
0.0020	0.0715	0.0732	0.0925	0.0925	0.0310	4.8690	5.6943	6.3339	6.5506
0.0030	0.1432	0.1384	0.1629	0.1629	0.0320	4.9653	5.7729	6.4510	6.7048
0.0040	0.2360	0.2242	0.2520	0.2520	0.0330	5.0497	5.8330	6.5479	6.8387
0.0050	0.3479	0.3312	0.3612	0.3612	0.0340	5.1221	5.8749	6.6247	6.9517
0.0060	0.4770	0.4593	0.4909	0.4909	0.0350	5.1822	5.8993	6.6818	7.0434
0.0070	0.6214	0.6082	0.6413	0.6413	0.0360	5.2300	5.9071	6.7197	7.1138
0.0080	0.7793	0.7768	0.8122	0.8122	0.0370	5.2655	5.8994	6.7391	7.1630
0.0090	0.9489	0.9641	1.0026	1.0026	0.0380	5.2886	5.8774	6.7412	7.1916
0.0100	1.1285	1.1684	1.2116	1.2116	0.0390	5.2996	5.8426	6.7270	7.2004
0.0110	1.3165	1.3880	1.4377	1.4377	0.0400	5.2986	5.7963	6.6980	7.1905
0.0120	1.5114	1.6210	1.6792	1.6792	0.0410	5.2858	5.7404	6.6557	7.1634
0.0130	1.7115	1.8651	1.9343	1.9343	0.0420	5.2615	5.6766	6.6019	7.1210
0.0140	1.9155	2.1182	2.2008	2.2008	0.0430	5.2261	5.6067	6.5385	7.0655
0.0150	2.1219	2.3779	2.4766	2.4766	0.0440	5.1799	5.5326	6.4674	6.9996
0.0160	2.3295	2.6419	2.7593	2.7593	0.0450	5.1234	5.4562	6.3908	6.9263
0.0170	2.5370	2.9076	3.0466	3.0356	0.0460	5.0573	5.3794	6.3108	6.8490
0.0180	2.7431	3.1729	3.3361	3.3149	0.0470	4.9820	5.3039	6.2297	6.7716
0.0190	2.9468	3.4354	3.6254	3.5979	0.0480	4.8983	5.2314	6.1497	6.6985
0.0200	3.1469	3.6927	3.9122	3.8828	0.0490	4.8069	5.1637	6.0731	6.6343
0.0210	3.3425	3.9429	4.1941	4.1673	0.0500	4.7085	5.1020	6.0020	6.5844
0.0220	3.5325	4.1839	4.4690	4.4493	0.0510	4.6041	5.0475	5.9386	6.5544
0.0230	3.7162	4.4138	4.7348	4.7268	0.0520	4.4945	5.0013	5.8850	6.5505
0.0240	3.8927	4.6310	4.9895	4.9976	0.0530	4.3806	4.9638	5.8429	
0.0250	4.0611	4.8340	5.2314	5.2598	0.0540	4.2637	4.9354	5.8140	
0.0260	4.2209	5.0214	5.4587	5.5114	0.0550	4.1446	4.9158	5.7999	
0.0270	4.3713	5.1922	5.6700	5.7506	0.0560	4.0247	4.9045	5.8015	
0.0280	4.5118	5.3454	5.8641	5.9756	n	3	3	3	3

CC = cement content
UCS = unconfined compressive strength
n = sample size
ε = strain

B.3 Indirect Tensile Strength Test

Cement content (%)	1%	3%	5%
7 - day Indirect Tensile Strength (MPa)	0.047	0.259	0.389
number of samples, n	3	3	3

Cement content (%)	2%	3%	4%	5%
28-day Indirect Tensile Strength (MPa)	0.466	0.692	0.806	1.111
sample sample, n	3	3	3	3

B.4 Flexural Bending Test

Cement content (%)	1%	2%	3%	4%	5%
Maximum breaking strain	0.0025	0.0015	0.0013	0.0015	0.0011
Maximum flexural stress (MPa)	0.4	0.8	1.3	1.9	2.2
Projected post failure strain	0.0034	0.0021	0.0019	0.0021	0.0018
sample sample, n	3	3	3	3	3

B.5 Fatigue Test Results

Not presented due to size of data.
Available upon request - contact author for further details

B.6 Elastic Strain Limit Tets

		CC (%)	1%	2%	3%	4%	5%
με	N	∑N	Stiffness, S (MPa)				
10	10	10	19693.7	18445.2	19964.6	22625.0	18562.3
10	20	20	16774.5	14024.5	17146.5	16305.0	23055.5
10	30	30	12434.1	14556.6	16231.9	17752.3	21122.8
10	40	40	15499.9	14875.0	10379.0	16750.6	17234.9
10	50	50	9910.1	16191.9	12867.7	17220.5	15423.1
10	60	60	12425.1	12700.6	14485.4	21000.8	13416.1
10	70	70	14944.6	17020.9	12222.8	15897.7	17522.7
10	80	80	13961.8	14696.5	13454.6	18472.7	21127.6
10	90	90	17819.8	12258.5	13816.5	13860.3	15603.4
10	100	100	12353.2	17042.9	10990.0	20338.0	25192.2
10	110	110	14125.9	15506.9	15454.3	17194.3	18188.8
10	120	120	12668.6	12460.4	13867.6	12533.3	14553.2
10	130	130	11806.2	15411.9	14117.5	11508.6	23458.1
10	140	140	14258.8	15105.2	10379.0	16465.8	21027.3
10	150	150	13246.1	13398.9	14378.3	16284.8	19283.0
10	160	160	12497.7	11536.4	11038.4	18344.2	18042.0
10	170	170	11739.0	13744.3	11980.2	13663.9	15825.2
10	180	180	12236.6	14259.9	12711.9	19083.4	21277.9
10	190	190	13480.0	16117.7	14845.7	19122.0	18819.6
10	200	200	12573.9	18260.6	11381.6	20358.9	21972.9
10	210	210	10586.7	12886.8	10976.8	15114.0	24296.5
10	220	220	15149.7	17543.3	15645.6	14674.8	26045.4
10	230	230	13200.5	13307.3	12712.8	18758.2	26145.2
10	240	240	12006.6	12144.4	9962.4	14952.9	14362.4
10	250	250	12006.6	15666.0	13378.5	14235.4	16410.6
10	260	260	10821.2	15579.9	15939.9	17895.3	15898.6
10	270	270	16490.0	14854.9	15387.6	16607.3	18253.3
10	280	280	12403.2	15369.6	15439.4	15947.1	23149.3
10	290	290	12165.3	17173.7	10379.0	21436.8	14704.0
10	300	300	13037.2	14231.2	10982.8	13999.2	21779.3
10	310	310	18098.9	12490.2	14663.9	18695.1	15891.8
10	320	320	10564.9	11308.1	17596.7	15282.7	14971.9
10	330	330	13627.8	10960.0	19796.3	15232.4	18540.1
10	340	340	12771.3	22582.4	11997.8	17955.1	18467.5
10	350	350	13072.5	14301.7	12830.9	15761.5	26296.0
10	360	360	15268.2	17662.4	13997.4	14869.4	16414.5
10	370	370	10930.6	15007.3	11843.9	13582.6	19973.9
10	380	380	14091.8	12789.7	13197.5	14499.6	18112.2
10	390	390	11821.5	15007.3	13997.4	18605.3	26800.0
10	400	400	12343.8	15007.6	13997.4	20323.8	22236.9
10	410	410	13750.5	14408.4	17107.9	14698.6	13888.7
10	420	420	14385.9	13266.6	15397.1	17146.6	16410.6
10	430	430	15727.9	14199.4	10998.0	15265.2	17704.9
10	440	440	11791.1	12803.3	10152.0	20223.1	16885.9
10	450	450	15475.1	13068.4	18329.9	16392.3	17271.9
10	460	460	13933.8	16412.6	11843.9	15946.8	18970.4
10	470	470	14245.8	18680.0	12219.9	17539.6	17885.5
10	480	480	9854.3	14859.9	12830.9	15441.6	17028.1
10	490	490	12589.8	11884.0	18329.9	15608.1	19460.0
10	500	500	13393.6	17224.8	9998.1	18994.4	18442.6
10	510	510	12683.7	9466.3	14663.9	15434.4	25251.9
10	520	520	14754.6	15824.4	9998.1	14632.4	14052.0
10	530	530	15210.7	18475.2	17596.7	14196.4	16074.6
10	540	540	10757.6	12232.3	13997.4	17876.4	20693.0
10	550	550	16926.0	14974.8	11997.8	19675.8	14659.4
10	560	560	8824.1	13216.2	13535.9	16048.6	14052.0
10	570	570	10992.1	10775.2	17596.7	20055.1	19115.7
10	580	580	18341.3	12363.3	7332.0	16423.9	19504.7
10	590	590	14270.1	15488.6	12830.9	11326.7	18074.7
10	600	600	14477.5	14459.5	17596.7	16126.0	21388.3
15	10	610	17568.1	10406.5	11815.3	15892.1	20291.4
15	20	620	11253.1	9754.5	8438.5	14550.9	16482.2
15	30	630	7784.5	8726.0	8297.1	12820.0	21043.2
15	40	640	8974.4	9803.9	10633.7	13617.8	14754.1
15	50	650	8848.9	9382.9	9832.9	18514.7	16994.5
15	60	660	10583.7	9376.2	12582.4	10580.3	13956.0
15	70	670	14852.9	9014.8	9533.9	12239.0	13884.1
15	80	680	8173.1	8754.2	10529.9	11613.3	14975.8

με	N	∑N	Stiffness, S (MPa)				
15	90	690	9744.3	8879.2	10343.9	10556.8	16321.3
15	100	700	10214.7	9724.5	7891.0	13884.3	18681.6
15	110	710	9977.6	10987.8	9929.5	15397.4	13429.9
15	120	720	9419.3	8477.6	8397.3	13791.0	15858.2
15	130	730	7033.2	8782.9	9242.2	15482.5	14471.5
15	140	740	9043.1	9267.4	8829.7	16870.5	11943.0
15	150	750	7927.7	9210.5	10185.4	15106.6	16233.1
15	160	760	9609.4	7856.2	8398.2	12379.7	17202.6
15	170	770	8848.9	10620.4	8930.0	13527.9	14393.3
15	180	780	10573.6	11600.1	13953.2	13552.4	15986.6
15	190	790	13564.7	10734.5	7288.9	15327.9	16236.7
15	200	800	11237.1	8801.0	7976.3	12234.3	14701.0
15	210	810	9077.9	8324.5	10191.6	13556.1	17838.3
15	220	820	7414.1	9916.2	7113.1	12935.9	15189.1
15	230	830	9661.2	10543.6	7916.6	14840.0	20744.6
15	240	840	8897.6	8215.1	12063.5	13488.9	16071.6
15	250	850	9459.8	8905.3	11440.7	14352.4	14393.3
15	260	860	13514.0	9656.1	8896.8	11335.1	13667.7
15	270	870	8526.3	10335.9	9839.8	15719.2	15381.7
15	280	880	7952.3	10506.2	12928.1	13415.8	13568.3
15	290	890	9621.8	7454.6	8231.8	12342.0	11894.5
15	300	900	8122.6	9935.7	8759.3	14840.0	17288.3
15	310	910	6065.8	8409.1	10998.0	12034.3	16691.8
15	320	920	10558.0	9415.5	12219.9	11926.8	16162.8
15	330	930	7790.1	10391.5	9426.8	12825.7	13892.9
15	340	940	7660.9	10211.7	6873.7	14729.1	12035.3
15	350	950	12042.4	9885.5	7332.0	13289.5	11673.0
15	360	960	8633.2	10669.2	14663.9	13675.2	20309.7
15	370	970	9846.5	7794.0	10351.0	11004.7	16290.3
15	380	980	10834.7	8774.3	7332.0	13886.9	18632.0
15	390	990	9699.5	7671.6	7855.7	11119.6	15918.5
15	400	1000	8974.4	10258.6	19063.1	13129.7	17781.1
15	410	1010	11211.3	8803.6	8248.5	14703.5	13340.1
15	420	1020	12234.9	7806.9	5175.5	12370.3	17965.9
15	430	1030	8594.4	9136.2	6469.4	15508.0	15461.4
15	440	1040	8162.5	10390.3	9776.0	13598.1	15218.5
15	450	1050	8251.2	10119.7	7763.3	15523.5	16277.7
15	460	1060	7477.7	6758.2	11731.1	11820.5	15215.2
15	470	1070	9956.2	13289.6	9057.1	12012.3	12734.2
15	480	1080	12231.1	7768.4	11731.1	12989.1	14841.4
15	490	1090	7407.5	10192.3	6469.4	14332.8	12310.1
15	500	1100	8425.0	11006.7	11644.9	13187.7	13921.4
15	510	1110	8974.4	11186.7	16820.4	12145.9	16463.1
15	520	1120	12426.9	10829.7	9898.2	14298.5	15674.8
15	530	1130	7915.4	8365.9	9057.1	11320.4	15486.4
15	540	1140	11436.3	10788.2	11644.9	14027.4	15926.5
15	550	1150	9151.1	8900.2	8248.5	13288.4	19533.8
15	560	1160	11996.0	9643.4	7763.3	14323.9	15674.0
15	570	1170	9506.7	9769.7	7855.7	15302.4	16595.0
15	580	1180	6618.7	8725.0	7332.0	13381.1	13061.7
15	590	1190	8509.7	10030.6	10998.0	14274.9	18878.2
15	600	1200	14235.5	8377.8	12372.7	12759.9	17916.8
20	10	1210	9651.8	7324.2	8864.0	14830.0	14011.2
20	20	1220	5520.7	6410.1	8084.4	14724.7	13202.2
20	30	1230	4987.2	7613.5	6373.6	12816.2	15382.8
20	40	1240	7682.8	7614.0	8067.8	14057.2	16209.6
20	50	1250	7286.0	7603.9	6407.6	12760.9	14446.1
20	60	1260	7103.9	7153.6	7412.0	13349.3	13897.4
20	70	1270	5483.9	7151.8	5444.9	12475.4	12842.4
20	80	1280	7132.9	5627.9	5474.3	13515.8	13785.8
20	90	1290	9403.9	8582.9	7753.7	13697.0	12963.3
20	100	1300	5990.7	8405.2	6280.6	12023.6	13649.3
20	110	1310	5929.5	8316.0	5308.5	11000.7	14234.0
20	120	1320	6553.5	6725.1	6111.4	12510.2	13776.6
20	130	1330	8182.8	6135.2	6886.0	12043.7	14762.9
20	140	1340	5579.9	6957.8	5541.0	10688.8	14281.7
20	150	1350	7438.9	6513.6	6662.2	14225.3	14231.0
20	160	1360	9254.8	6521.0	8377.3	15244.8	14340.5
20	170	1370	6051.2	6912.1	9149.8	12729.2	14895.0
20	180	1380	6319.0	7160.0	8142.7	12749.2	12355.7
20	190	1390	7654.0	6647.6	6073.7	11694.2	12651.5
20	200	1400	5856.0	7637.4	7055.0	12003.1	14922.2
20	210	1410	9108.4	7222.3	6565.9	12915.0	13016.4

με	N	∑N	Stiffness, S (MPa)				
20	220	1420	6558.4	7360.9	6037.2	10556.6	13442.5
20	230	1430	6028.7	6993.1	7274.0	13128.1	14030.7
20	240	1440	6013.6	8049.0	5886.4	11209.5	16816.7
20	250	1450	8019.1	6842.9	9060.2	11661.7	16178.5
20	260	1460	6892.3	8471.3	9148.0	11142.2	17958.5
20	270	1470	6667.2	6661.8	7230.0	13311.0	14708.0
20	280	1480	5832.2	7362.4	8141.8	12837.5	14088.4
20	290	1490	7718.7	6717.8	8428.2	13075.9	16082.2
20	300	1500	6565.6	6050.6	9060.2	10201.6	11103.2
20	310	1510	7688.3	6630.2	5738.1	13228.4	14569.2
20	320	1520	8725.2	8102.4	6284.5	12409.3	15123.9
20	330	1530	6920.5	6260.3	7998.5	11043.2	14375.1
20	340	1540	6486.7	9251.6	8798.4	13878.7	15502.5
20	350	1550	6981.6	6983.1	5499.0	11559.2	15091.9
20	360	1560	7170.0	6109.1	5998.9	14024.1	15626.6
20	370	1570	6330.6	6124.7	8998.3	12341.0	14942.6
20	380	1580	7089.8	6574.9	12097.7	13600.1	13702.8
20	390	1590	6763.7	6518.0	6694.4	13189.2	15602.8
20	400	1600	6205.7	7676.2	9426.8	12886.8	13997.0
20	410	1610	5303.6	6740.9	8379.4	12731.0	16225.9
20	420	1620	7115.9	6745.3	6694.4	14091.7	12793.9
20	430	1630	6008.5	9772.2	7038.7	14345.3	13322.4
20	440	1640	7780.6	7499.7	6694.4	11722.6	12992.3
20	450	1650	6391.1	6335.8	6998.7	12024.5	12953.9
20	460	1660	5549.0	6659.3	8379.4	13621.7	14922.2
20	470	1670	7342.7	6173.7	7332.0	11152.1	16328.5
20	480	1680	9000.1	7038.6	5998.9	13586.9	14570.1
20	490	1690	7656.4	6623.7	10474.2	13538.5	16485.9
20	500	1700	8150.6	6169.3	6598.8	11346.2	12722.6
20	510	1710	5506.2	6710.1	5499.0	14618.6	14759.2
20	520	1720	6654.4	8383.8	6694.4	13148.4	12887.9
20	530	1730	5148.6	7412.2	6284.5	11840.1	13705.4
20	540	1740	7364.1	6648.9	8103.8	13361.9	14419.6
20	550	1750	6925.5	5992.7	7698.6	12583.4	15152.8
20	560	1760	6930.1	7167.8	5738.1	10567.2	14570.1
20	570	1770	6802.2	7644.2	5237.1	13861.9	14787.0
20	580	1780	6427.7	7843.7	7332.0	12966.4	12914.3
20	590	1790	7018.6	8846.4	8379.4	12684.8	14913.3
20	600	1800	6778.9	8321.9	8607.1	13225.2	14642.4
25	10	1810	6373.5	6370.1	7720.2	17961.4	16047.5
25	20	1820	4191.4	5368.9	5550.8	12403.8	14355.1
25	30	1830	7217.8	6649.1	4839.8	11852.5	13640.6
25	40	1840	7265.0	6418.7	5260.1	12115.5	12707.0
25	50	1850	4971.3	5879.6	6336.1	11067.7	12808.9
25	60	1860	5942.7	6007.5	4121.0	10624.1	13580.9
25	70	1870	6979.7	5875.1	4953.6	11114.8	12914.3
25	80	1880	4601.7	4987.8	7136.0	12982.9	13421.0
25	90	1890	5813.4	6531.9	6017.2	12396.7	14146.8
25	100	1900	5666.9	6725.3	4889.6	11685.5	13736.0
25	110	1910	5987.7	7567.8	6582.4	11204.0	17015.9
25	120	1920	6871.3	7574.6	6453.0	11154.0	13470.9
25	130	1930	5947.2	5949.4	6513.1	12154.6	13636.0
25	140	1940	5243.6	6360.0	5550.8	12675.1	13464.3
25	150	1950	5829.8	7171.0	5146.3	13331.1	12266.6
25	160	1960	5582.9	7755.7	6989.8	13088.6	13424.9
25	170	1970	5551.8	6119.8	4904.6	11794.6	12916.5
25	180	1980	6406.9	7202.2	6209.3	11743.9	13619.2
25	190	1990	6331.1	5384.2	6717.4	11162.8	14626.3
25	200	2000	7410.2	6927.3	4670.2	11751.8	14335.9
25	210	2010	5641.0	7450.5	5651.0	11290.0	13166.4
25	220	2020	8724.3	5571.0	5242.4	12384.4	13196.7
25	230	2030	6099.5	6485.8	7688.0	12133.0	13295.5
25	240	2040	5547.5	7092.5	6294.5	11846.9	12483.8
25	250	2050	4431.3	5265.2	3958.1	11777.2	12345.5
25	260	2060	5709.7	7773.3	5083.2	12498.4	14152.0
25	270	2070	6912.6	5044.1	7631.8	12604.1	12471.2
25	280	2080	6706.2	6302.6	7393.6	12789.9	12770.0
25	290	2090	4934.5	5068.3	7117.7	11805.0	12739.7
25	300	2100	6837.5	5868.9	6804.0	11589.2	12345.5
25	310	2110	6123.8	6959.8	5279.0	12571.4	13710.3
25	320	2120	6064.3	6483.8	7070.1	12338.0	11392.0
25	330	2130	4661.4	6362.7	5702.6	12591.8	14759.1
25	340	2140	7956.9	7008.2	5279.0	13036.5	12768.0

με	N	∑N	Stiffness, S (MPa)				
25	350	2150	4737.4	6020.1	5702.6	13100.3	13367.6
25	360	2160	4274.6	5799.0	6517.3	13142.2	11489.9
25	370	2170	5624.5	6390.6	6517.3	12730.2	13499.4
25	380	2180	4353.7	5359.5	7038.7	12416.8	13708.7
25	390	2190	5903.1	6005.1	6517.3	12883.9	12123.3
25	400	2200	6612.5	7962.8	5922.0	12270.0	13763.5
25	410	2210	4524.2	5986.2	5922.0	10648.1	13625.4
25	420	2220	4934.5	6497.8	5922.0	12123.3	12825.0
25	430	2230	6946.9	6623.9	8460.0	11358.8	15166.9
25	440	2240	7116.5	5335.6	8460.0	12832.6	13763.5
25	450	2250	6886.8	5228.9	6284.5	11683.9	16287.4
25	460	2260	5440.0	7875.1	7095.5	12162.9	14185.3
25	470	2270	4194.2	6816.2	4888.0	11499.8	14232.9
25	480	2280	6335.6	6501.0	4399.2	12636.4	15056.3
25	490	2290	6313.8	6366.2	7038.7	11768.5	14052.0
25	500	2300	5963.9	5610.1	4230.0	12429.5	12582.4
25	510	2310	6298.7	5941.6	3927.8	11347.7	12567.7
25	520	2320	6637.0	5487.7	5922.0	12741.9	13876.0
25	530	2330	4635.6	6081.8	4888.0	12693.9	15609.6
25	540	2340	5159.7	6750.7	4073.3	11596.1	14764.1
25	550	2350	4113.9	6439.3	7332.0	10465.7	13419.5
25	560	2360	4036.1	5772.3	5702.6	11843.3	15325.7
25	570	2370	5520.7	6136.9	7038.7	12728.6	13434.4
25	580	2380	5366.4	5701.5	4713.4	12996.8	13980.7
25	590	2390	5858.2	5589.5	9306.0	11418.8	12668.0
25	600	2400	4938.4	6546.0	6517.3	11719.0	15281.4
30	10	2410	4694.1	6640.2	5144.8	14543.5	13798.2
30	20	2420	4576.5	5991.8	4415.5	10946.9	13629.0
30	30	2430	4157.4	5986.9	5358.0	11463.3	13957.4
30	40	2440	5154.3	5076.2	4557.8	9596.5	15706.7
30	50	2450	4011.3	5261.6	4411.0	9920.7	14055.6
30	60	2460	4998.5	4609.6	6050.1	10379.8	14520.2
30	70	2470	4399.2	6564.8	4895.5	11290.7	13697.8
30	80	2480	3720.1	5429.2	7142.1	10841.3	13093.8
30	90	2490	4682.6	5891.2	4430.7	10801.2	14388.1
30	100	2500	5367.2	4815.1	4065.7	11125.0	12795.4
30	110	2510	4398.7	6232.0	4082.2	10023.2	13079.5
30	120	2520	4551.4	5802.3	5456.7	11098.8	13818.7
30	130	2530	5093.1	7291.3	6019.7	12373.7	13785.8
30	140	2540	4846.7	4542.0	4280.6	9921.5	12690.5
30	150	2550	5436.0	5437.4	5866.5	11002.3	14544.0
30	160	2560	5196.2	5871.3	4550.6	9599.2	16068.5
30	170	2570	6325.7	5901.8	4350.4	10784.7	14201.2
30	180	2580	4538.1	6565.3	4832.1	10494.7	14157.7
30	190	2590	4743.8	5477.8	3960.9	10148.4	13558.7
30	200	2600	6599.4	5295.3	4282.2	10664.5	14413.4
30	210	2610	4545.9	5903.9	5275.5	10240.6	15122.1
30	220	2620	3914.6	6240.5	4036.1	9736.5	14283.4
30	230	2630	5127.4	6197.1	5095.8	10017.9	13567.7
30	240	2640	5518.8	5701.8	4148.5	9921.4	11895.2
30	250	2650	6216.0	4413.1	4211.2	10144.9	13981.7
30	260	2660	5601.7	4745.6	4211.8	10627.6	13401.6
30	270	2670	4424.0	6894.1	5622.6	12666.1	12494.3
30	280	2680	5101.8	4833.8	4689.6	9971.9	14413.4
30	290	2690	4548.0	4680.3	5281.2	11100.6	12702.3
30	300	2700	4058.2	6232.6	5690.0	11492.7	12869.2
30	310	2710	4048.4	5013.1	5132.4	10952.1	13365.7
30	320	2720	5025.0	5585.9	3999.3	10648.3	15742.4
30	330	2730	5126.3	5429.1	5676.4	11274.9	14068.5
30	340	2740	4516.3	6519.6	5676.4	9332.3	15702.5
30	350	2750	4875.5	5530.2	5499.0	9917.4	13963.6
30	360	2760	3630.2	6002.3	6385.9	10062.0	13567.7
30	370	2770	5578.2	6258.5	4257.3	11001.6	13974.5
30	380	2780	3863.5	6128.7	5499.0	11752.2	12519.8
30	390	2790	4992.5	6101.7	4399.2	11032.5	13789.8
30	400	2800	4595.1	6072.2	5499.0	10649.6	13927.3
30	410	2810	4828.5	7036.1	5332.3	10472.6	14023.6
30	420	2820	5269.0	5504.7	4399.2	10607.1	14439.2
30	430	2830	4422.9	5394.5	5822.4	10538.9	13177.5
30	440	2840	4634.5	5114.2	3436.9	9963.7	15215.2
30	450	2850	4753.0	6118.0	4550.9	11481.4	14586.7
30	460	2860	4363.8	5903.6	5332.3	10747.9	13148.9
30	470	2870	4408.2	4966.6	4811.6	10661.4	13821.1

με	N	∑N	Stiffness, S (MPa)				
30	480	2880	5299.3	6163.5	6721.0	10910.6	14188.0
30	490	2890	4529.4	5025.0	4257.3	12249.6	13715.4
30	500	2900	4990.0	6048.4	8514.5	11563.4	14258.7
30	510	2910	4233.1	5715.5	5499.0	10112.5	12104.1
30	520	2920	4742.2	4882.1	5676.4	10867.8	13439.2
30	530	2930	5460.5	5185.7	7561.1	11118.9	11926.1
30	540	2940	5210.1	6271.5	3881.6	10063.4	12441.2
30	550	2950	4894.4	5283.5	4811.6	10885.4	14549.2
30	560	2960	4941.0	5442.9	4966.8	10256.6	13965.1
30	570	2970	5192.0	4446.9	5998.9	9651.5	14818.6
30	580	2980	4623.7	4878.9	7805.0	10757.4	15009.1
30	590	2990	4424.5	6989.1	6873.7	10324.7	14900.1
30	600	3000	3842.4	5780.2	5132.4	11452.0	14068.5
35	10	3010	3737.1	5640.2	3064.6	11416.9	13288.8
35	20	3020	3398.1	4776.3	4142.9	10962.7	10848.5
35	30	3030	3323.4	4814.8	4331.9	11047.7	12855.3
35	40	3040	3246.0	4416.1	5912.7	10924.7	12995.7
35	50	3050	2867.8	5080.0	5452.7	10425.4	12227.7
35	60	3060	3534.9	4472.0	4828.5	10995.8	12298.1
35	70	3070	4253.0	5270.1	4061.6	10822.6	14648.5
35	80	3080	3280.9	4667.8	6455.7	9908.7	12556.2
35	90	3090	3586.8	5725.2	4118.1	10811.1	14365.6
35	100	3100	4889.8	4394.0	4177.9	11860.2	12808.7
35	110	3110	6383.6	4675.7	3935.4	11327.6	12695.9
35	120	3120	3722.1	5624.8	4898.8	10897.0	12424.0
35	130	3130	4532.4	5888.2	5900.1	9673.8	12563.8
35	140	3140	3567.2	4939.9	5341.8	10032.6	12831.7
35	150	3150	4171.3	5010.9	4521.2	10634.4	14520.1
35	160	3160	3981.8	5024.0	3751.5	10007.9	13073.3
35	170	3170	3827.7	5192.2	4582.5	11217.7	14362.4
35	180	3180	3497.1	4424.9	4509.6	12243.3	12914.3
35	190	3190	3826.0	5514.6	5589.7	10208.0	13718.3
35	200	3200	4123.4	3917.5	4069.8	10184.9	12421.5
35	210	3210	4300.7	5739.2	5352.1	9868.4	12714.5
35	220	3220	3432.2	4146.6	5669.0	10211.3	13487.3
35	230	3230	3582.4	5476.7	4120.6	10069.3	12523.7
35	240	3240	5153.6	6538.3	5729.1	11412.6	13243.2
35	250	3250	3208.1	4842.5	4564.8	11394.6	12728.1
35	260	3260	5139.7	4618.6	5147.2	10686.3	12976.4
35	270	3270	5287.3	4424.8	3736.1	11421.5	13023.6
35	280	3280	3637.4	5328.5	4928.5	10362.1	11964.2
35	290	3290	3582.4	5017.5	3963.7	11326.2	12414.7
35	300	3300	4098.6	4971.5	6290.4	11143.5	12640.9
35	310	3310	3789.5	5136.1	4630.7	11235.2	13258.1
35	320	3320	5466.8	4544.4	4277.0	9420.2	12560.9
35	330	3330	3878.7	4974.6	4630.7	11207.3	11012.8
35	340	3340	5390.2	4846.5	3666.0	11024.2	11990.0
35	350	3350	5493.2	6027.3	3566.9	10873.2	13421.9
35	360	3360	4060.5	4972.1	4051.9	10707.5	12206.7
35	370	3370	4068.0	4789.8	3055.0	9977.8	11898.6
35	380	3380	4471.3	5891.8	4755.9	10793.0	13005.7
35	390	3390	6166.5	4836.6	4755.9	11776.1	13237.6
35	400	3400	4640.2	5013.2	3566.9	11533.1	13053.7
35	410	3410	5156.6	5473.4	4291.9	9512.0	12863.6
35	420	3420	3429.3	6678.1	4755.9	10930.4	12256.4
35	430	3430	5355.9	4858.7	7133.8	10606.8	12525.1
35	440	3440	4480.2	4160.1	4888.0	10302.4	13432.2
35	450	3450	2949.1	4570.3	5499.0	10474.2	12728.1
35	460	3460	4505.4	5091.0	4630.7	9676.2	12382.4
35	470	3470	3865.0	4694.4	4399.2	11584.8	12862.2
35	480	3480	3246.0	5836.9	4755.9	10745.0	13411.6
35	490	3490	7315.9	5274.0	7332.0	10145.2	13264.5
35	500	3500	4370.5	5022.5	5499.0	11650.1	12814.0
35	510	3510	4676.9	6175.5	3948.0	10323.6	11727.3
35	520	3520	4554.4	4034.8	3849.3	10648.3	14669.1
35	530	3530	4470.2	4398.9	3566.9	11005.3	13517.4
35	540	3540	5426.6	5385.8	4630.7	11377.1	12525.1
35	550	3550	5253.4	5125.9	3384.0	11025.5	12782.2
35	560	3560	4471.3	5057.5	4277.0	10470.9	13634.4
35	570	3570	4237.9	4514.3	4277.0	9567.7	12434.0
35	580	3580	4424.1	5405.4	5076.0	10392.6	13982.2
35	590	3590	4157.2	6311.3	4051.9	10517.0	11965.7
35	600	3600	3194.0	5820.2	3384.0	10313.1	11962.8

με	N	∑N	Stiffness, S (MPa)				
40	10	3610	4176.7	4387.4	3992.9	12117.6	14595.8
40	20	3620	3890.2	4191.0	4211.3	10422.6	12820.5
40	30	3630	4447.1	4121.5	4609.5	10364.3	13766.3
40	40	3640	3507.4	4440.5	6156.7	12363.3	13702.4
40	50	3650	3913.6	5825.0	4155.9	10780.7	12473.8
40	60	3660	3875.7	4480.4	4440.0	10284.5	12538.2
40	70	3670	2720.8	3978.6	3862.8	9894.5	12386.1
40	80	3680	3458.3	4543.2	4683.2	10345.1	12032.2
40	90	3690	3405.8	4161.1	4263.0	10344.8	12302.7
40	100	3700	3763.1	5032.3	4618.1	9324.7	11184.7
40	110	3710	2606.4	4505.6	5250.3	10562.6	13762.5
40	120	3720	4399.9	4445.8	4862.1	10070.5	12498.6
40	130	3730	4819.5	4088.1	4870.6	10327.2	12051.8
40	140	3740	5090.3	6301.9	4028.2	10554.1	13224.1
40	150	3750	3675.9	4359.0	4450.5	9893.6	12322.4
40	160	3760	4202.4	4580.9	4668.1	10266.4	12452.1
40	170	3770	3839.3	4798.9	3806.7	10736.4	12948.1
40	180	3780	3623.4	5005.5	4165.8	10409.4	12094.3
40	190	3790	3412.1	4296.8	4084.6	10023.2	11876.6
40	200	3800	3770.5	4330.1	4155.7	10386.1	12275.3
40	210	3810	2367.0	4733.0	4053.5	10704.9	13135.1
40	220	3820	4016.2	4504.6	4358.5	10419.7	12977.8
40	230	3830	3431.8	4263.0	4251.1	9562.3	12538.2
40	240	3840	3067.3	4780.6	6835.2	10358.2	14663.9
40	250	3850	3715.9	4336.4	4055.4	9141.7	12729.5
40	260	3860	3621.2	4910.2	4650.5	10455.6	13115.2
40	270	3870	4215.9	5123.3	3975.7	9919.3	13309.4
40	280	3880	3292.4	5594.3	3858.1	10891.3	12275.3
40	290	3890	3516.8	4443.1	4456.2	9545.3	14356.0
40	300	3900	3290.0	4282.9	4980.8	10673.8	12701.5
40	310	3910	3599.9	4169.7	6974.3	10205.2	13079.2
40	320	3920	3770.5	4555.9	4999.1	12184.9	11754.1
40	330	3930	3661.6	4230.0	4603.8	11476.1	11932.7
40	340	3940	3763.0	5034.1	5626.9	10139.1	12189.3
40	350	3950	4079.3	4583.5	4680.0	10268.1	14687.1
40	360	3960	4055.1	5668.7	4713.4	9765.3	12263.4
40	370	3970	3400.4	5291.3	5115.3	9551.9	13114.5
40	380	3980	3203.6	4133.0	5760.8	9384.1	12283.0
40	390	3990	3121.1	5032.5	5626.9	10158.8	12594.8
40	400	4000	3564.1	4362.3	4189.7	10237.1	14641.7
40	410	4010	3067.1	4006.1	4828.4	9993.3	13679.9
40	420	4020	3389.3	4694.9	5760.8	10310.7	12924.8
40	430	4030	4129.8	4815.0	4092.3	10858.7	13230.6
40	440	4040	3485.3	5325.3	6216.2	10353.0	12046.5
40	450	4050	3516.3	4582.9	5376.8	9473.5	12122.0
40	460	4060	4819.7	4551.0	5259.9	10714.8	12344.4
40	470	4070	2758.8	4036.1	6552.0	9971.6	12067.1
40	480	4080	3215.7	4639.4	5626.9	10327.9	12980.6
40	490	4090	4098.4	4836.0	5115.3	10211.0	12027.5
40	500	4100	2943.1	3949.8	5626.9	9650.1	12039.5
40	510	4110	3294.2	5649.1	6284.5	10234.5	12174.8
40	520	4120	3369.1	5140.0	5626.9	10069.0	12323.9
40	530	4130	3628.9	4100.4	5626.9	10127.0	11864.0
40	540	4140	3613.6	4373.6	4189.7	10884.6	11463.7
40	550	4150	4149.3	5304.4	4291.9	10524.1	12847.9
40	560	4160	4372.6	4826.5	5364.9	10604.8	12816.6
40	570	4170	3117.6	4330.1	4713.4	9885.6	12816.4
40	580	4180	3520.2	4842.3	4303.5	9965.3	13365.2
40	590	4190	3635.2	4224.1	6974.3	10208.0	12896.3
40	600	4200	4565.9	4975.6	4603.8	10005.7	11296.8
45	10	4210	3181.3	5812.7	4450.8	12915.5	12694.8
45	20	4220	3405.6	4323.7	5102.4	9712.0	11354.0
45	30	4230	4647.8	4076.1	4003.0	10092.2	12840.5
45	40	4240	5150.0	3610.7	4884.3	10048.0	12712.9
45	50	4250	3178.5	4942.5	4644.5	10338.1	13031.8
45	60	4260	2927.9	5248.8	4934.7	9022.5	13860.1
45	70	4270	3336.3	3773.1	3786.5	9837.6	13078.5
45	80	4280	2502.1	4742.8	4177.4	10544.7	12388.6
45	90	4290	2294.0	3622.4	4809.9	9042.5	12174.5
45	100	4300	3670.5	4111.8	4974.0	9775.0	13347.0
45	110	4310	3632.5	4506.0	6093.3	9446.4	12636.1
45	120	4320	5221.5	3758.6	6084.3	9928.4	12369.7
45	130	4330	2989.4	4235.2	3928.0	9079.6	11992.2

με	N	∑N	Stiffness, S (MPa)				
45	140	4340	3290.2	4623.6	4783.7	9973.1	11877.3
45	150	4350	3064.3	3889.6	4181.6	9999.1	13465.8
45	160	4360	3255.8	3425.2	4711.1	10353.8	12681.0
45	170	4370	3708.8	3902.0	4272.5	9865.8	12754.7
45	180	4380	2869.7	3696.1	5199.0	10175.9	11800.4
45	190	4390	2889.1	3800.2	4873.5	9786.6	12763.4
45	200	4400	3246.1	3805.8	4333.2	10087.9	11563.6
45	210	4410	3492.5	4572.5	3928.6	10294.7	13421.7
45	220	4420	3005.2	5518.2	3919.0	10632.2	13424.0
45	230	4430	3009.3	4542.1	4784.3	9962.3	12888.6
45	240	4440	2813.3	3715.3	4211.5	9705.1	12864.7
45	250	4450	3236.3	5196.9	5849.3	9923.2	11426.8
45	260	4460	2984.6	4459.5	4512.9	10086.8	12476.3
45	270	4470	4382.2	4053.7	4581.2	9813.3	12625.1
45	280	4480	3405.7	4153.5	3909.2	9243.0	12482.2
45	290	4490	3686.4	4206.1	4410.5	9239.2	12994.7
45	300	4500	3522.2	3967.9	4853.6	9180.4	12070.4
45	310	4510	3420.8	4897.1	5616.0	9425.8	12523.6
45	320	4520	2914.9	3915.7	7488.0	9761.6	13186.8
45	330	4530	3709.9	4566.2	5957.2	9572.0	13345.6
45	340	4540	3418.0	3782.1	6694.4	9613.9	13188.0
45	350	4550	3657.5	4335.3	4680.0	9819.6	12954.2
45	360	4560	3048.7	3993.3	5499.0	9507.5	12829.6
45	370	4570	2497.5	4465.1	6216.2	9767.8	12993.5
45	380	4580	2895.9	4131.8	5148.0	9946.0	12953.1
45	390	4590	3676.8	3830.8	5386.8	9811.1	12753.6
45	400	4600	2963.5	3840.4	4680.0	10280.7	12070.4
45	410	4610	3160.8	4935.5	5040.7	9698.3	12607.6
45	420	4620	3751.0	4138.2	6216.2	9903.4	12447.1
45	430	4630	3507.7	4106.9	5040.7	9333.3	12640.9
45	440	4640	3635.7	4155.9	5148.0	10980.5	13147.7
45	450	4650	3405.7	4172.4	5616.0	10529.0	12103.6
45	460	4660	3014.2	4264.2	5606.8	10764.5	12599.6
45	470	4670	2724.4	4153.6	5718.9	8973.8	11634.5
45	480	4680	3373.6	3769.7	6084.0	9549.7	12259.5
45	490	4690	3255.8	3188.2	5175.5	10007.9	12251.0
45	500	4700	3477.9	3870.2	6216.2	9735.0	12967.6
45	510	4710	3744.5	3588.3	5259.9	9557.9	13091.5
45	520	4720	3009.3	4507.1	6415.5	9467.3	13420.5
45	530	4730	2692.7	4205.6	5386.8	9681.6	13232.3
45	540	4740	4310.6	3581.9	5148.0	9442.3	12461.0
45	550	4750	3009.3	4945.0	5957.2	9622.8	12573.3
45	560	4760	4264.7	4647.7	4680.0	9992.8	12294.5
45	570	4770	4390.8	4372.2	5499.0	9742.1	13124.8
45	580	4780	3140.5	4165.3	5040.7	9938.4	13555.6
45	590	4790	2517.2	4188.6	5040.7	9715.4	12245.1
45	600	4800	2915.2	6615.6	5616.0	9325.5	12715.9
50	10	4810	2760.1	4366.9	2823.5	9720.2	12178.9
50	20	4820	2978.9	3783.4	3950.6	9470.3	12452.1
50	30	4830	2993.7	3605.3	3844.3	9811.0	12583.5
50	40	4840	3013.3	3814.7	3685.8	9807.6	12737.4
50	50	4850	2641.4	5063.1	5457.8	9719.3	12696.8
50	60	4860	3206.2	3892.3	3496.2	10029.1	12112.5
50	70	4870	2429.2	3904.0	3509.8	9490.9	12449.6
50	80	4880	3026.6	3436.9	4014.3	9172.4	11875.6
50	90	4890	2857.2	4145.0	3254.1	10106.5	12818.8
50	100	4900	2772.4	3841.8	2812.7	9063.6	12196.0
50	110	4910	3068.9	3454.7	3315.4	9709.8	12387.1
50	120	4920	2870.0	3212.0	3051.1	10152.8	11695.0
50	130	4930	2195.1	4204.0	3409.9	9083.1	11455.4
50	140	4940	2755.4	4015.5	3938.0	9079.4	12049.5
50	150	4950	2738.5	3531.1	3496.2	9154.0	12336.6
50	160	4960	2953.2	3305.3	2487.2	9520.7	11927.7
50	170	4970	2906.4	3379.6	3074.5	10125.0	12213.7
50	180	4980	3451.2	3367.8	2616.8	9501.4	12268.3
50	190	4990	3169.5	3977.6	3379.3	8957.9	12345.5
50	200	5000	2342.2	4319.1	3398.1	9451.3	12985.4
50	210	5010	2840.9	4049.4	3324.6	8955.7	11819.8
50	220	5020	3292.9	4641.5	4558.7	9261.0	11803.0
50	230	5030	3436.7	3771.7	2895.2	9730.8	12586.5
50	240	5040	3306.0	3484.1	3552.3	9131.6	11904.3
50	250	5050	3136.2	3567.3	2874.1	8815.0	12556.7
50	260	5060	2912.8	3998.6	3389.3	9222.3	11919.9

με	N	∑N	Stiffness, S (MPa)				
50	270	5070	3061.9	3661.9	3688.8	9494.7	12344.4
50	280	5080	3708.9	3570.9	3736.7	9330.9	12092.7
50	290	5090	3129.3	4221.7	4193.4	9329.6	12825.6
50	300	5100	3339.6	3899.6	4144.3	9903.0	12414.1
50	310	5110	3455.7	3453.2	4230.0	9785.6	12949.7
50	320	5120	3191.1	3692.6	5076.0	9262.1	12861.1
50	330	5130	2474.7	3851.4	4744.2	9379.4	11771.5
50	340	5140	1920.4	3728.6	4480.6	9441.0	12449.6
50	350	5150	2628.0	3464.1	4653.0	9486.1	11577.7
50	360	5160	3212.1	2854.0	4653.0	9477.8	11991.4
50	370	5170	2767.8	3730.7	2961.0	9672.8	12559.8
50	380	5180	3132.5	3690.9	2961.0	9281.9	11988.5
50	390	5190	2850.5	3603.3	4653.0	8948.3	13773.9
50	400	5200	3733.2	3747.1	5395.2	9189.1	11880.4
50	410	5210	3072.5	3677.2	3735.2	9693.4	11866.6
50	420	5220	3269.9	3437.0	3735.2	9422.5	12200.9
50	430	5230	2990.4	3597.9	3320.1	10436.9	11326.9
50	440	5240	3572.7	4598.8	4073.3	8646.5	12346.5
50	450	5250	2877.0	3262.7	3258.7	9189.9	12667.0
50	460	5260	3636.9	4121.5	4980.2	9061.8	12883.5
50	470	5270	3050.1	3349.6	2749.5	10089.1	13111.0
50	480	5280	4341.6	4056.0	3320.1	9413.3	13994.1
50	490	5290	2645.1	3605.2	4073.3	9492.7	12124.9
50	500	5300	2936.0	3921.1	3881.6	9570.9	12322.9
50	510	5310	2642.5	4519.4	5076.0	9629.5	11452.5
50	520	5320	2586.0	3624.0	3999.3	9363.2	12261.8
50	530	5330	3620.8	4126.0	4073.3	9772.8	11977.6
50	540	5340	3835.0	3182.3	3807.0	9267.7	12707.3
50	550	5350	3196.7	3589.0	4653.0	10019.6	13056.8
50	560	5360	2617.8	3670.4	3735.2	9689.6	12673.3
50	570	5370	2763.0	3211.1	3320.1	8972.0	12772.1
50	580	5380	3079.5	4057.4	3599.3	9083.3	12023.9
50	590	5390	2421.3	3712.4	4073.3	8796.3	13044.6
50	600	5400	3151.1	3782.7	3807.0	9239.6	11796.3
55	10	5410	3603.3	4734.9	4473.7	9817.9	12364.2
55	20	5420	3926.7	3788.0	3792.4	9008.1	11780.6
55	30	5430	3040.8	3373.5	3728.1	9810.3	12529.4
55	40	5440	3008.2	3136.2	3245.3	9344.4	11239.5
55	50	5450	2739.2	3418.7	3792.4	9249.6	11652.7
55	60	5460	2765.3	3307.2	3792.4	9343.5	11728.9
55	70	5470	2513.8	3517.5	4171.6	8683.8	12122.6
55	80	5480	3190.2	3891.2	3413.2	8882.9	13319.2
55	90	5490	2816.0	3545.5	3728.1	8636.5	12113.8
55	100	5500	2814.4	3221.0	2524.1	9861.3	12294.5
55	110	5510	3384.6	3306.9	4171.6	8705.2	12309.9
55	120	5520	2840.2	2775.8	4550.9	8949.5	12256.9
55	130	5530	2895.2	3610.2	3413.2	8474.7	12339.1
55	140	5540	2837.9	3088.9	3473.0	8999.2	12426.4
55	150	5550	3037.0	3574.5	3792.4	9229.7	11595.7
55	160	5560	2653.0	3675.4	4171.6	9303.3	11949.9
55	170	5570	3198.0	4542.9	3792.4	9030.5	11885.3
55	180	5580	3205.9	4149.8	3355.3	9958.0	11445.7
55	190	5590	2612.3	3291.1	4930.1	9527.5	12798.6
55	200	5600	3053.7	3785.7	3413.2	8603.8	11847.3
55	210	5610	2863.8	3045.2	3792.4	9754.6	12004.2
55	220	5620	2814.4	3219.9	3792.4	9254.6	12949.5
55	230	5630	2262.6	3307.2	3087.1	8723.5	11627.6
55	240	5640	3059.0	3662.8	2701.3	9271.4	11724.1
55	250	5650	2428.4	2925.2	5219.4	8712.3	12819.4
55	260	5660	2428.4	3235.6	4171.6	9370.7	11427.5
55	270	5670	2454.4	3306.2	3087.1	8952.1	11099.4
55	280	5680	2910.2	3680.5	3033.9	8979.2	12024.7
55	290	5690	2931.2	3408.4	4032.6	8510.2	12322.9
55	300	5700	2401.7	3075.3	3413.2	9239.8	11974.4
55	310	5710	2663.6	3336.4	4930.1	9064.6	11705.4
55	320	5720	2421.1	3032.9	3858.9	9261.3	12169.4
55	330	5730	3142.6	3688.6	4930.1	9293.2	13077.3
55	340	5740	2840.2	3230.3	4100.9	8982.6	11674.4
55	350	5750	3885.4	4028.6	4327.1	9542.4	11741.2
55	360	5760	2433.3	3104.3	4171.6	9043.7	12195.9
55	370	5770	4549.7	3434.6	3355.3	8955.9	10642.8
55	380	5780	2584.2	3307.7	2982.5	8571.8	11502.5
55	390	5790	3139.0	3132.3	4171.6	8830.5	12214.9

με	N	∑N	Stiffness, S (MPa)				
55	400	5800	2791.6	3246.3	4171.6	8592.9	11705.4
55	410	5810	2646.6	3201.3	3858.9	8653.3	11338.3
55	420	5820	2423.7	3996.0	3355.3	9200.2	11464.6
55	430	5830	2788.9	3668.8	3033.9	8781.5	11902.4
55	440	5840	2939.7	3250.4	3413.2	9031.7	11621.7
55	450	5850	2638.3	3688.9	3355.3	9118.6	11382.6
55	460	5860	2886.3	3654.9	3473.0	8859.6	11933.8
55	470	5870	2688.0	4052.8	3355.3	9218.1	11329.4
55	480	5880	3271.5	3391.9	3473.0	8934.6	11239.2
55	490	5890	2960.8	2786.1	3087.1	9090.0	12245.3
55	500	5900	2830.2	3137.6	4100.9	9443.4	12334.8
55	510	5910	3491.1	3925.8	3355.3	10285.9	12533.7
55	520	5920	3265.9	3327.2	4244.8	9749.0	12323.0
55	530	5930	3612.4	3731.8	3858.9	8613.2	12010.1
55	540	5940	3143.8	3149.9	2932.8	9226.1	11904.7
55	550	5950	3718.1	3316.5	5219.4	8820.3	11962.7
55	560	5960	2791.6	3691.9	4630.7	9624.2	11410.8
55	570	5970	3413.0	3454.4	4846.6	9333.6	12724.0
55	580	5980	2651.3	3388.0	4550.9	9445.1	11551.7
55	590	5990	2665.3	3481.6	5586.3	8919.7	11070.2
55	600	6000	3417.2	3179.4	4171.6	9329.7	12057.9
60	10	6010	2692.7	3595.0	3098.8	9717.7	11867.1
60	20	6020	2816.3	3469.5	2927.1	9010.9	12844.3
60	30	6030	3051.1	3954.4	3374.3	8926.6	11984.7
60	40	6040	2049.3	3237.0	3255.6	8524.6	12015.0
60	50	6050	2991.1	3226.3	3146.8	8523.2	12155.6
60	60	6060	2487.1	3350.9	3614.0	9214.4	12148.6
60	70	6070	2962.2	3467.1	4273.9	8933.4	12457.9
60	80	6080	3031.3	3699.9	3106.8	9284.6	12588.6
60	90	6090	2159.2	2966.4	3136.1	9420.8	12007.6
60	100	6100	2248.3	3351.5	3387.5	9018.5	11932.2
60	110	6110	2752.4	3014.5	3968.7	9291.1	13300.1
60	120	6120	2695.8	3112.6	3095.8	8695.2	12388.1
60	130	6130	2168.3	3674.6	3317.6	8738.1	12617.4
60	140	6140	2240.0	3254.3	3395.9	8649.6	10978.0
60	150	6150	2373.3	3393.1	2912.9	9091.4	11855.9
60	160	6160	2148.7	3107.6	3829.3	9714.0	12230.3
60	170	6170	2614.5	3625.8	3686.7	8820.7	12674.9
60	180	6180	1989.9	3031.4	3509.3	9457.2	12648.9
60	190	6190	2160.8	3226.5	4003.9	8475.3	12920.2
60	200	6200	2640.9	3255.7	2742.1	9139.0	12657.6
60	210	6210	2920.2	3677.3	3876.6	8819.8	12647.8
60	220	6220	2598.5	3922.1	2592.1	8344.5	12763.4
60	230	6230	2138.9	3390.9	4009.7	8357.7	11833.5
60	240	6240	1975.1	2890.9	3044.4	9020.6	12481.5
60	250	6250	2730.2	4246.0	3296.9	8756.5	12918.3
60	260	6260	3154.3	3207.2	3481.2	9466.2	12652.7
60	270	6270	2618.5	3137.5	3495.0	8839.6	12105.5
60	280	6280	3257.3	3602.8	3810.3	8511.2	12035.9
60	290	6290	2356.7	3104.5	3967.7	8799.4	12022.7
60	300	6300	2283.5	3483.6	4354.0	8951.3	12674.9
60	310	6310	2288.8	3133.9	3722.4	8766.6	12461.6
60	320	6320	3308.3	3605.4	3780.5	8725.3	12442.6
60	330	6330	2181.2	3316.0	4467.9	8752.8	12585.3
60	340	6340	2925.5	3160.7	3283.0	9391.2	12176.7
60	350	6350	2179.3	3344.2	3722.4	9130.4	12197.4
60	360	6360	2703.3	3448.5	3491.4	8620.5	12105.5
60	370	6370	2487.3	3258.4	3491.4	8537.7	13114.1
60	380	6380	3864.8	3024.0	3436.9	8443.8	12526.4
60	390	6390	2433.7	3109.6	3142.3	9561.6	13268.4
60	400	6400	2192.5	3495.2	3902.5	8450.6	12762.0
60	410	6410	2479.2	3905.3	3840.6	8710.8	12744.2
60	420	6420	2430.2	3855.1	4327.1	8381.4	13431.8
60	430	6430	2492.2	3914.6	5676.4	9125.2	12211.3
60	440	6440	2101.9	3187.8	3840.6	8782.9	12624.6
60	450	6450	1955.1	3406.6	3142.3	9046.9	12539.8
60	460	6460	2619.1	3170.5	3902.5	8667.4	11826.1
60	470	6470	3392.7	4984.7	3436.9	8857.4	12770.4
60	480	6480	2404.7	2792.6	3436.9	8845.1	12584.5
60	490	6490	1937.2	3760.8	3491.4	8781.7	12564.2
60	500	6500	2825.2	3797.9	3093.2	9004.0	12196.0
60	510	6510	2398.2	3349.0	3780.5	8710.2	13020.7
60	520	6520	2323.6	3010.1	4189.7	9356.1	11905.9

με	N	∑N	Stiffness, S (MPa)				
60	530	6530	2300.6	3094.7	3722.4	9523.7	12949.3
60	540	6540	2591.3	3157.9	3093.2	7984.9	13175.4
60	550	6550	2274.4	3563.8	4612.0	8487.0	12016.0
60	560	6560	2505.1	3091.9	2793.1	8377.5	12034.0
60	570	6570	3373.5	2879.6	4189.7	8181.8	12191.2
60	580	6580	2541.2	3636.7	3840.6	8809.7	12087.2
60	590	6590	2398.2	4187.7	3999.3	8075.0	12074.0
60	600	6600	2742.9	3513.5	3547.7	8913.2	11555.9
65	10	6610	2277.1	3255.3	2707.2	8103.0	11955.9
65	20	6620	2382.3	3421.0	2711.8	7799.5	11927.7
65	30	6630	2978.9	3564.1	2911.2	7356.8	12883.2
65	40	6640	2665.3	2982.7	4144.2	7357.7	11614.9
65	50	6650	2592.8	2837.8	4027.4	7762.7	11701.7
65	60	6660	2527.0	2941.9	2550.2	7622.2	11647.5
65	70	6670	2365.7	2828.2	3234.7	8086.4	12993.5
65	80	6680	2133.6	3274.1	2869.0	7580.5	12371.4
65	90	6690	2530.8	3258.4	3234.7	7846.5	11625.1
65	100	6700	3181.3	3189.8	3234.7	7667.3	11623.5
65	110	6710	2703.8	2931.0	2587.8	7604.2	11481.3
65	120	6720	2365.8	3426.3	2869.0	7847.6	12514.1
65	130	6730	3073.7	2889.7	2550.2	7955.5	12148.2
65	140	6740	2714.3	3216.2	3881.6	7374.2	11287.7
65	150	6750	2831.8	2603.2	2869.0	7460.1	11669.5
65	160	6760	2402.6	3219.0	3283.0	7585.5	11762.5
65	170	6770	2771.8	3569.1	3825.4	8426.7	12583.6
65	180	6780	2392.6	3033.5	2911.2	8464.1	12045.1
65	190	6790	2831.8	2730.6	5252.8	7856.0	13410.5
65	200	6800	2946.1	2892.7	2828.0	8059.7	11813.8
65	210	6810	1895.0	2890.2	3558.2	7587.7	11870.6
65	220	6820	2195.0	2731.8	2869.0	7531.9	12036.1
65	230	6830	2703.8	3432.2	2869.0	7248.0	11566.4
65	240	6840	2687.7	3194.4	2911.2	7584.9	11416.2
65	250	6850	2730.7	3341.7	2869.0	7931.1	12307.3
65	260	6860	2426.8	3528.7	2788.2	8067.7	11139.7
65	270	6870	3583.9	2888.4	2869.0	7640.5	11892.1
65	280	6880	3701.8	3457.2	3506.6	7676.9	11528.7
65	290	6890	2649.4	2882.0	2788.2	8166.8	11543.3
65	300	6900	1901.9	3001.9	2869.0	7791.0	11720.5
65	310	6910	2656.4	2975.0	2550.2	7883.3	12164.8
65	320	6920	2492.1	2728.0	3407.8	7891.0	12631.0
65	330	6930	2613.5	3669.7	2478.4	7727.7	11817.1
65	340	6940	2056.5	2833.4	2911.2	7836.0	11859.4
65	350	6950	2858.9	2730.6	3234.7	7864.8	11777.9
65	360	6960	2428.0	2799.6	4462.9	7582.4	12116.7
65	370	6970	2872.0	3788.2	2231.5	7851.2	12556.0
65	380	6980	2687.7	2988.6	3234.7	7924.4	11870.6
65	390	6990	2684.3	3490.5	3187.8	7661.0	12032.1
65	400	7000	2687.7	3019.7	2911.2	7812.5	12753.6
65	410	7010	2438.5	2976.8	3234.7	7382.0	11557.3
65	420	7020	1883.3	3115.0	4205.1	8209.5	11492.5
65	430	7030	2318.7	3301.3	4528.6	7335.4	12533.7
65	440	7040	3047.6	3398.2	2749.5	8131.9	12953.8
65	450	7050	1854.8	3082.2	3558.2	8108.7	11370.9
65	460	7060	3126.7	2887.5	3939.6	8237.1	11298.0
65	470	7070	2014.2	3304.3	2911.2	8088.0	11767.2
65	480	7080	2473.2	2866.3	3187.8	7326.3	11476.7
65	490	7090	2247.4	3420.3	3558.2	7342.2	12059.3
65	500	7100	2804.0	3358.6	2954.7	7974.9	11943.3
65	510	7110	2086.7	2969.7	3234.7	7850.0	11389.1
65	520	7120	3539.1	3034.3	4781.7	7908.9	11901.7
65	530	7130	2186.0	3134.7	2911.2	7796.0	12012.5
65	540	7140	3104.1	3536.0	2513.8	7375.5	11542.5
65	550	7150	3306.7	3499.2	2869.0	7845.3	11481.3
65	560	7160	2073.1	2711.3	2869.0	7797.5	12127.5
65	570	7170	3080.2	3302.0	3881.6	7792.4	12466.4
65	580	7180	2211.1	2535.4	3558.2	8096.4	11882.6
65	590	7190	1993.8	3083.2	4462.9	8401.5	11049.5
65	600	7200	2387.1	3206.3	2869.0	7673.5	11946.7
70	10	7210	2165.2	3178.0	3233.1	8267.3	12742.2
70	20	7220	2055.5	3062.1	2552.7	7983.8	11825.9
70	30	7230	2153.8	2485.7	3382.9	7672.8	11965.0
70	40	7240	2314.1	2580.7	2940.4	8646.2	12041.3
70	50	7250	2367.1	2674.9	2826.6	7797.0	12208.0

με	N	∑N	Stiffness, S (MPa)				
70	60	7260	1963.5	2407.9	2832.8	8341.2	12710.8
70	70	7270	1978.3	2871.5	2971.1	7987.4	12885.0
70	80	7280	1957.9	2592.9	2684.5	7937.8	11879.0
70	90	7290	1964.5	2509.9	3552.7	8304.5	12712.3
70	100	7300	2111.3	2894.5	2705.0	8190.1	12053.3
70	110	7310	2359.8	2412.5	3296.2	8152.4	11736.4
70	120	7320	2464.0	3008.9	3137.7	7903.5	12485.5
70	130	7330	2521.3	2604.9	3041.1	8182.5	12044.0
70	140	7340	2174.3	2420.8	2347.0	8787.6	11193.7
70	150	7350	2760.8	3004.3	2830.4	8044.0	11990.0
70	160	7360	1960.4	2588.1	3192.6	7617.5	11127.4
70	170	7370	2656.9	2884.4	2915.7	7899.5	12340.4
70	180	7380	1557.4	2553.6	3303.5	8029.7	12700.8
70	190	7390	2376.0	2627.3	2669.8	8606.1	12339.0
70	200	7400	2052.9	2526.1	2652.7	8029.4	11951.3
70	210	7410	2378.5	2279.0	3021.2	7722.6	12053.3
70	220	7420	2427.9	2725.9	2793.7	8142.4	12476.0
70	230	7430	2394.9	2534.1	2938.8	8110.7	12193.9
70	240	7440	2442.1	2962.9	2798.1	8323.2	12408.3
70	250	7450	2121.5	3000.2	2794.8	8121.3	12510.5
70	260	7460	3350.3	2394.9	3629.6	7622.8	12105.9
70	270	7470	1666.8	2428.4	2366.9	7879.1	12046.7
70	280	7480	2502.5	2319.9	3151.7	7383.3	12162.1
70	290	7490	2632.8	2781.4	2639.1	7898.6	12288.3
70	300	7500	2458.2	2873.2	3281.8	8217.5	12769.0
70	310	7510	2343.8	2583.2	2377.9	8864.8	11917.2
70	320	7520	1827.9	2556.3	2410.5	8007.8	12421.5
70	330	7530	2320.2	2797.4	3615.8	7435.2	11594.3
70	340	7540	2636.9	3034.0	4218.4	7596.2	11652.2
70	350	7550	2155.2	2080.8	3055.0	8104.3	12492.9
70	360	7560	2219.9	2711.2	3013.1	8255.7	12102.5
70	370	7570	2432.0	2967.7	4161.4	7532.5	12298.1
70	380	7580	2251.6	2676.2	2972.4	7834.3	11965.0
70	390	7590	2334.4	2286.6	4161.4	7947.0	11974.9
70	400	7600	1971.3	2405.2	3269.7	8039.6	11860.4
70	410	7610	2262.8	2530.2	2346.2	7739.0	11985.7
70	420	7620	2062.9	3178.1	2410.5	7479.6	11575.8
70	430	7630	1755.2	2790.6	3360.5	7540.0	11954.8
70	440	7640	2039.6	2852.3	3269.7	8006.8	12105.9
70	450	7650	1958.9	2794.9	3314.5	7581.9	11658.3
70	460	7660	2418.8	3247.9	3013.1	7932.2	12433.5
70	470	7670	2007.8	2755.6	4105.9	7537.2	12037.8
70	480	7680	2268.0	2092.3	2675.2	8014.5	12180.2
70	490	7690	2567.4	2765.6	2711.8	7826.4	12426.0
70	500	7700	3279.2	2691.4	3269.7	7909.2	11884.4
70	510	7710	1993.0	2515.7	3013.1	7920.2	12322.7
70	520	7720	1926.5	2637.8	3360.5	7839.0	11574.6
70	530	7730	2167.7	2459.0	2972.4	7543.2	11572.4
70	540	7740	1947.7	2843.2	3269.7	7605.4	12084.0
70	550	7750	2959.4	2477.6	3427.9	7873.2	11594.3
70	560	7760	2366.9	2923.3	3615.8	8395.7	11481.4
70	570	7770	2602.1	2926.4	2675.2	7862.1	11754.9
70	580	7780	2486.7	2856.6	2410.5	8043.6	13268.9
70	590	7790	2097.7	2934.5	2856.6	7934.2	12102.0
70	600	7800	2233.4	3135.4	2675.2	7599.3	11888.0
75	10	7810	2410.6	2812.4	3611.3	8870.1	13397.3
75	20	7820	3046.3	3615.8	3102.0	8260.2	12815.4
75	30	7830	2305.4	3287.5	3384.0	8238.5	11642.9
75	40	7840	1895.5	3373.0	3619.6	7712.8	12261.5
75	50	7850	2258.9	2394.9	3487.2	8056.1	12328.3
75	60	7860	2154.7	3202.1	3299.4	7405.8	11686.8
75	70	7870	3043.7	3055.9	3948.0	7821.1	11682.3
75	80	7880	2298.0	2946.3	3341.1	8024.2	11269.8
75	90	7890	2458.6	2997.0	4794.0	8094.5	11753.1
75	100	7900	2525.9	2626.2	2784.3	7403.2	11761.6
75	110	7910	2044.2	2568.9	2505.9	7603.4	12036.2
75	120	7920	1602.4	3165.3	2784.3	7930.1	11205.5
75	130	7930	3428.6	2463.7	3666.0	7522.2	11621.6
75	140	7940	2241.3	3149.8	3898.0	8029.4	11850.0
75	150	7950	1644.8	2919.0	2715.5	7477.4	12471.8
75	160	7960	2712.9	2782.2	3849.3	8697.5	11973.8
75	170	7970	3290.3	3518.6	2987.1	7781.9	11559.3
75	180	7980	2330.0	3120.1	3574.3	7531.7	11622.9

με	N	∑N	Stiffness, S (MPa)				
75	190	7990	2052.2	3071.2	3427.9	7638.6	11051.3
75	200	8000	2485.0	2737.0	3142.3	8282.8	11909.8
75	210	8010	2608.6	2873.4	3384.0	7318.8	10802.5
75	220	8020	2458.3	3184.5	3142.3	7392.3	11789.3
75	230	8030	1919.4	2781.1	3102.0	7548.3	11569.2
75	240	8040	2104.5	2820.8	4284.9	7804.5	11813.6
75	250	8050	2017.1	2801.1	3849.3	7260.6	11489.2
75	260	8060	2334.4	2522.5	2650.1	8081.2	11409.5
75	270	8070	2089.5	2781.5	3102.0	7681.5	11796.7
75	280	8080	2206.9	3016.6	2784.3	7517.8	10755.1
75	290	8090	2344.7	3261.2	4454.9	7971.7	11700.2
75	300	8100	1773.7	2574.7	3619.6	7570.2	11189.9
75	310	8110	2298.0	2570.6	2915.1	8080.6	12010.7
75	320	8120	2224.5	3064.7	3102.0	7427.6	12728.1
75	330	8130	2264.7	3540.6	3666.0	7689.1	11709.6
75	340	8140	2195.5	2676.3	2846.5	7235.7	11412.2
75	350	8150	2840.6	2708.6	3666.0	7935.2	11732.2
75	360	8160	2185.2	2838.5	3102.0	7675.2	11150.3
75	370	8170	2312.9	2569.2	3666.0	7306.1	11467.2
75	380	8180	2270.9	2881.6	3849.3	7720.0	11265.8
75	390	8190	1884.3	2792.9	4399.2	7639.6	12811.0
75	400	8200	2806.0	2981.2	3619.6	7573.3	11147.9
75	410	8210	2519.1	2861.7	3530.2	7851.6	11629.6
75	420	8220	2775.3	2766.0	3427.9	7471.0	11566.7
75	430	8230	2456.3	2561.5	2820.0	8444.1	11734.3
75	440	8240	2290.4	3273.5	3384.0	7669.8	11574.5
75	450	8250	1931.4	2761.8	3384.0	7556.1	11130.2
75	460	8260	2062.0	2531.7	2538.0	7128.7	11409.5
75	470	8270	1644.8	2399.8	2784.3	7592.8	11546.0
75	480	8280	2193.6	2665.9	3530.2	7857.2	10850.6
75	490	8290	1931.2	2741.0	3299.4	8156.6	11470.1
75	500	8300	2446.2	3046.8	2505.9	8316.6	11211.2
75	510	8310	2341.9	3234.8	3341.1	7233.8	11462.6
75	520	8320	1919.4	2832.1	2987.1	7482.9	11285.9
75	530	8330	2509.2	2732.2	3849.3	7826.9	11476.5
75	540	8340	2784.3	3113.8	3427.9	7608.1	11490.6
75	550	8350	2776.2	2558.3	2950.7	7600.7	11129.1
75	560	8360	2946.5	2813.4	2820.0	8400.6	11921.6
75	570	8370	2566.4	3783.5	2820.0	7204.5	11304.9
75	580	8380	1979.2	2598.8	2856.6	7491.9	11326.9
75	590	8390	2053.9	2859.3	3384.0	7041.9	10796.0
75	600	8400	2220.9	2650.5	3142.3	7065.7	11549.3
80	10	8410	1961.7	2456.9	3613.0	6907.6	8537.8
80	20	8420	1709.1	2518.4	3794.1	6535.9	9008.9
80	30	8430	1782.3	2380.3	3650.3	6725.9	8881.5
80	40	8440	2097.3	2629.4	3431.6	6635.1	8585.8
80	50	8450	1809.0	2775.0	3713.0	6166.2	8591.7
80	60	8460	2156.5	2225.5	3436.4	6457.1	9670.1
80	70	8470	2311.7	2358.5	3498.2	6567.1	9041.7
80	80	8480	2046.3	2411.6	3459.9	6541.5	8583.3
80	90	8490	1912.4	2188.4	3366.6	6689.3	9089.9
80	100	8500	1542.5	2710.4	3647.3	6439.8	9502.4
80	110	8510	2162.9	2950.1	3221.8	6355.8	9039.5
80	120	8520	1950.9	2438.7	3126.5	6407.6	8677.0
80	130	8530	2050.3	2514.2	3806.0	6567.6	8779.7
80	140	8540	2029.0	2171.8	3012.3	6665.9	8829.0
80	150	8550	2451.3	2393.3	3209.5	6237.9	8322.9
80	160	8560	2461.1	2712.0	3209.3	6358.2	8319.4
80	170	8570	1950.5	2131.2	3771.6	6415.0	8464.1
80	180	8580	2068.0	2341.4	3066.2	6283.2	8622.2
80	190	8590	2277.9	2636.9	3491.6	6327.8	8927.1
80	200	8600	1712.2	2851.5	3284.0	6454.1	8960.4
80	210	8610	1889.5	2527.4	3459.9	6683.2	8409.5
80	220	8620	1619.0	2161.8	3047.4	6335.7	8203.8
80	230	8630	2241.0	2598.6	4143.4	6345.7	8770.9
80	240	8640	2355.6	2546.4	3129.4	6652.2	8540.2
80	250	8650	1908.6	2672.1	2958.9	6487.6	8713.7
80	260	8660	1874.0	2263.9	3221.8	6823.6	8786.4
80	270	8670	1539.9	2576.5	3333.3	6522.3	8378.5
80	280	8680	1959.3	2512.6	3852.3	6560.5	8865.0
80	290	8690	1891.7	3044.1	3607.3	6958.5	9046.1
80	300	8700	2210.1	2076.9	3513.6	6100.4	8058.9
80	310	8710	2993.3	2860.8	3280.6	6432.8	8091.3

με	N	∑N	Stiffness, S (MPa)				
80	320	8720	1984.6	2668.0	3892.3	6353.4	7985.1
80	330	8730	1986.7	2143.1	3645.1	6527.9	8502.3
80	340	8740	1569.9	2449.6	2964.3	6238.0	8671.0
80	350	8750	2104.7	2540.4	3613.9	7453.5	8120.2
80	360	8760	2225.2	2591.1	3401.0	6302.6	9202.7
80	370	8770	1537.3	2243.3	3800.6	7241.0	8851.0
80	380	8780	1561.7	2375.1	2985.8	6550.3	8476.9
80	390	8790	1852.1	2501.0	4043.3	6361.4	8501.0
80	400	8800	1693.3	2494.1	3244.9	6521.5	8509.0
80	410	8810	1617.2	2762.9	3229.8	6354.4	8310.0
80	420	8820	2300.6	2178.1	3186.2	6124.9	8495.3
80	430	8830	1968.2	2511.6	3996.8	6437.0	8292.0
80	440	8840	1782.5	2610.4	3199.2	6354.6	9061.7
80	450	8850	1977.7	2068.0	3370.1	6670.8	8116.6
80	460	8860	1918.3	2424.5	3104.7	6548.7	8401.0
80	470	8870	2667.0	2872.1	2779.8	6291.6	8073.9
80	480	8880	1687.6	2346.5	3478.6	6241.2	8215.9
80	490	8890	2718.9	2088.5	3634.8	6637.1	9076.9
80	500	8900	1945.1	2446.7	2821.3	6694.4	8120.2
80	510	8910	2420.7	2489.7	2964.3	7070.4	8680.4
80	520	8920	1905.0	2980.2	3107.6	6807.8	8344.2
80	530	8930	2854.2	2726.8	2815.3	6213.3	8515.7
80	540	8940	2566.3	2168.2	3247.7	6552.0	8300.8
80	550	8950	1807.6	2409.6	3090.0	7225.2	8621.8
80	560	8960	1888.2	2648.7	3373.0	6719.7	8490.8
80	570	8970	1794.2	2565.1	3361.7	6283.2	8014.6
80	580	8980	2031.2	2160.6	3199.0	6655.8	8449.8
80	590	8990	1812.1	2636.5	3351.9	6791.7	8874.9
80	600	9000	1882.5	2429.0	3396.6	6451.5	8174.1
85	10	9010	1690.1	1651.6	3621.8	6859.6	9192.1
85	20	9020	2431.8	1638.7	3873.4	6465.4	9221.2
85	30	9030	2179.0	2261.8	3747.0	6551.9	8971.9
85	40	9040	1474.5	1705.9	3449.8	6356.7	9713.4
85	50	9050	2160.6	1855.0	3332.4	6805.0	9003.0
85	60	9060	1576.6	1715.0	3504.2	6195.7	8702.2
85	70	9070	2107.8	1986.9	3889.6	6502.5	8758.1
85	80	9080	2693.5	1683.5	3779.8	6125.1	8390.1
85	90	9090	1579.3	2390.5	3563.8	6220.0	9167.3
85	100	9100	2066.1	1525.3	3792.9	6584.1	8699.7
85	110	9110	2613.6	2237.1	3384.4	6401.2	9449.3
85	120	9120	1695.2	1943.4	3898.1	6184.9	9025.0
85	130	9130	1942.4	1930.8	3514.7	6344.8	8751.8
85	140	9140	1945.8	1978.6	3374.1	6049.0	8360.7
85	150	9150	2079.1	1713.7	3368.4	6449.3	8612.6
85	160	9160	1927.6	1811.0	3314.7	6287.3	8653.2
85	170	9170	2209.1	1609.7	3113.5	6070.6	8343.6
85	180	9180	1922.7	2109.9	3646.5	6207.1	9103.4
85	190	9190	2050.8	1713.6	4043.1	6441.9	9126.5
85	200	9200	1954.0	1796.6	3243.9	6024.1	8558.4
85	210	9210	2148.6	2103.2	3491.3	6116.0	8729.8
85	220	9220	2438.7	2203.0	3597.8	6168.5	8409.4
85	230	9230	2046.4	2062.5	3155.3	5932.0	8603.1
85	240	9240	2036.2	1934.8	3548.4	6120.5	8511.4
85	250	9250	2071.1	1960.1	3407.8	6225.6	9124.9
85	260	9260	2229.9	1545.5	3304.1	6586.3	9026.1
85	270	9270	1808.4	2120.5	3074.5	6588.9	8861.2
85	280	9280	1453.0	1886.1	3461.4	5906.7	8361.5
85	290	9290	2333.7	2123.1	3106.4	5761.7	8701.9
85	300	9300	1788.0	1695.6	3459.3	6016.8	8209.7
85	310	9310	1704.5	2451.9	3148.1	5906.5	8883.0
85	320	9320	3088.7	2106.6	3575.4	5911.1	8946.5
85	330	9330	1458.1	1545.9	2962.7	6010.0	8360.7
85	340	9340	2171.1	2025.5	3963.9	6085.9	8037.8
85	350	9350	2152.9	1854.0	3659.0	6193.4	8178.3
85	360	9360	2277.6	1784.8	3428.9	6112.1	8354.9
85	370	9370	2064.2	1645.4	3969.3	6047.6	8489.9
85	380	9380	1711.4	1930.2	3401.0	5932.3	8454.0
85	390	9390	1948.0	2233.2	3499.5	6466.5	8029.0
85	400	9400	2054.0	2047.5	3748.5	6628.1	8500.7
85	410	9410	1839.2	1960.0	3278.7	5960.0	7895.8
85	420	9420	1737.3	2014.4	3756.1	6017.7	8130.1
85	430	9430	2167.9	1992.7	3307.0	6084.0	8313.9
85	440	9440	2065.6	1464.4	3164.9	5768.5	8684.6

με	N	∑N	Stiffness, S (MPa)				
85	450	9450	2318.7	1865.1	3631.6	5886.0	8657.0
85	460	9460	1932.2	2036.8	3565.4	6073.9	7783.5
85	470	9470	2797.7	1629.5	3069.7	5907.0	8148.2
85	480	9480	1961.3	2438.9	3749.2	6063.2	8035.8
85	490	9490	1924.4	2110.9	3128.0	6032.8	8532.4
85	500	9500	2438.1	2014.1	3374.1	5729.0	8153.3
85	510	9510	1917.9	1958.8	3509.4	6097.6	8226.1
85	520	9520	2791.2	2394.3	3322.2	6552.9	8446.4
85	530	9530	1684.6	2000.6	3299.2	5744.5	8454.6
85	540	9540	2087.3	1945.1	3408.6	5840.6	8617.3
85	550	9550	2297.8	1775.3	3747.0	6262.8	8681.1
85	560	9560	1947.5	2225.7	3300.3	6147.0	9539.6
85	570	9570	2678.5	1864.8	3521.8	6362.8	8150.0
85	580	9580	2184.6	1694.9	3259.8	6028.0	8607.2
85	590	9590	1926.5	1699.9	3243.1	6047.5	8253.7
85	600	9600	1695.0	2061.5	3223.0	5890.5	8557.4
90	10	9610	1585.0	2406.2	2365.2	5896.1	7432.7
90	20	9620	1594.2	2241.5	2778.4	5568.7	7946.5
90	30	9630	1643.1	1608.3	1852.3	5376.3	7688.7
90	40	9640	1773.6	1947.0	2083.8	5116.6	7147.5
90	50	9650	1723.0	1774.5	2040.9	5572.7	7454.6
90	60	9660	1335.8	2087.4	2315.4	5385.7	7184.7
90	70	9670	1605.1	2083.2	2083.8	5470.5	7452.7
90	80	9680	1744.4	2010.6	2062.1	5573.8	7647.7
90	90	9690	1827.4	1849.3	2106.0	5463.3	7221.2
90	100	9700	1525.0	1990.3	3042.0	5104.0	7299.2
90	110	9710	2047.5	2210.4	2315.4	5220.4	7452.7
90	120	9720	1686.4	2086.2	2444.0	5392.8	7378.3
90	130	9730	1450.8	2537.1	2947.9	5349.3	7884.8
90	140	9740	1666.5	2090.2	2365.2	5525.1	7414.8
90	150	9750	1894.4	1849.2	2267.6	4842.3	7304.5
90	160	9760	1986.7	2066.2	2083.8	5148.4	7343.8
90	170	9770	1910.3	2126.0	2340.0	5658.9	8080.9
90	180	9780	1747.6	2474.3	2778.4	5132.3	7844.9
90	190	9790	1509.0	2172.4	3241.5	5263.9	7188.4
90	200	9800	1638.2	1841.4	2315.4	5179.0	6946.3
90	210	9810	1797.6	1628.7	2244.5	5135.0	7493.2
90	220	9820	1890.3	2139.2	1620.8	5197.2	7262.1
90	230	9830	2040.0	1769.3	1814.1	5209.9	7764.4
90	240	9840	1894.9	2023.0	3010.0	5519.9	6909.1
90	250	9850	1528.2	2010.4	2778.4	5536.5	7266.4
90	260	9860	1843.7	2021.5	1833.0	5128.0	7021.0
90	270	9870	1614.4	1914.5	2494.4	4869.2	7063.1
90	280	9880	1759.1	2329.5	2083.8	4781.0	7067.8
90	290	9890	1664.7	2321.6	2520.4	4746.0	6707.7
90	300	9900	2072.9	2130.5	1814.1	5066.9	7647.6
90	310	9910	2082.0	2384.2	2546.9	4881.4	6954.3
90	320	9920	1671.6	2097.9	1872.0	5283.0	7920.1
90	330	9930	1978.2	1917.7	2778.4	5701.7	7107.7
90	340	9940	1819.5	2241.8	2106.0	4657.1	7105.0
90	350	9950	1752.6	1835.5	1833.0	4914.3	6834.8
90	360	9960	1597.1	1959.9	2494.4	4908.4	7652.6
90	370	9970	1910.8	1836.8	2749.5	5019.9	6756.8
90	380	9980	2080.1	1927.7	2574.0	5185.1	6744.9
90	390	9990	1499.9	1900.8	2340.0	5070.2	6873.7
90	400	10000	1809.4	1926.5	3074.7	5039.9	7099.3
90	410	10010	2029.6	2379.6	2315.4	5710.9	6941.2
90	420	10020	1885.6	1843.8	2062.1	4845.5	6889.1
90	430	10030	2186.1	2172.3	2340.0	4975.8	6492.5
90	440	10040	1880.7	2106.0	2340.0	5216.2	6641.8
90	450	10050	1360.9	1885.1	2083.8	5152.0	7066.5
90	460	10060	2212.2	1677.4	2267.6	5295.7	7025.9
90	470	10070	1807.3	1752.5	2520.4	5290.6	7027.9
90	480	10080	1526.2	2875.9	2106.0	4898.9	6609.3
90	490	10090	1513.5	2076.1	2574.0	4968.7	7179.4
90	500	10100	1521.3	1959.2	1852.3	5062.1	6907.1
90	510	10110	1523.1	1919.2	2574.0	5251.3	6902.3
90	520	10120	1686.1	1606.5	2546.9	5126.1	6959.6
90	530	10130	2001.3	1549.3	2546.9	5336.9	7500.7
90	540	10140	1600.6	1662.7	2546.9	4668.5	6796.7
90	550	10150	2031.3	2472.8	1852.3	5903.7	6420.3
90	560	10160	2525.9	1925.6	2340.0	5724.3	6649.6
90	570	10170	1907.0	2169.1	2083.8	4746.7	7023.9

με	N	∑N	Stiffness, S (MPa)				
90	580	10180	1441.8	2230.2	2340.0	4963.2	7337.7
90	590	10190	2218.7	2015.1	2128.6	5617.8	6911.7
90	600	10200	1515.1	2261.4	1999.6	4974.7	7182.9
100	10	10210	1833.3	1313.6	2094.8	5096.3	8065.7
100	20	10220	1517.5	1437.1	2075.1	5011.6	7806.9
100	30	10230	1912.4	1436.1	2036.7	5239.0	7789.3
100	40	10240	1763.0	1439.5	2466.8	4775.4	7131.2
100	50	10250	1229.9	1253.1	2304.3	5185.4	7198.4
100	60	10260	2117.4	1812.9	1439.0	4899.3	7318.5
100	70	10270	1585.9	1778.1	2075.1	4719.5	7038.0
100	80	10280	1378.1	1306.8	2094.8	5124.1	6938.2
100	90	10290	1437.2	1504.3	2036.7	5545.2	7024.5
100	100	10300	1834.7	1372.8	2055.7	4886.5	6448.1
100	110	10310	1779.1	1589.2	2538.0	4799.6	7092.5
100	120	10320	1519.6	1545.0	3351.8	4998.2	6683.6
100	130	10330	1679.0	1327.4	2513.8	4884.7	7024.4
100	140	10340	2649.6	1489.4	3112.6	4882.1	7013.0
100	150	10350	1843.9	1481.5	2490.1	4586.6	6995.1
100	160	10360	1636.1	1499.0	2055.7	5198.5	6510.7
100	170	10370	1633.7	1392.1	2723.3	4449.1	6586.6
100	180	10380	1727.1	1468.3	2513.8	4672.6	6815.6
100	190	10390	1797.2	1614.2	2490.1	4821.0	6887.4
100	200	10400	1506.5	1728.0	2199.6	4610.6	6786.4
100	210	10410	1368.2	1231.9	1885.4	5015.0	6751.6
100	220	10420	1768.2	1199.3	2055.7	4804.3	6747.8
100	230	10430	1374.3	1698.7	1480.5	4704.9	6603.8
100	240	10440	1712.0	1618.5	1867.6	4867.0	6361.3
100	250	10450	2246.1	1398.5	1885.4	4741.9	6874.9
100	260	10460	2056.4	1468.5	2261.3	4961.5	6517.6
100	270	10470	1701.6	1840.7	2075.1	4413.0	6631.7
100	280	10480	1491.5	1679.6	2304.3	4368.0	6482.0
100	290	10490	1355.3	1380.9	2304.3	4689.3	6662.6
100	300	10500	1720.7	1661.4	1692.0	4423.0	6406.5
100	310	10510	1637.1	1389.1	2075.1	4987.8	6747.8
100	320	10520	2041.2	1274.4	2115.0	4889.0	6554.2
100	330	10530	1933.7	1141.6	2326.5	4828.9	6442.0
100	340	10540	1727.7	1386.6	2932.8	4737.0	6512.1
100	350	10550	1728.8	1249.4	2776.2	4592.9	6468.8
100	360	10560	1539.8	1393.1	2094.8	4822.3	6547.3
100	370	10570	1446.0	1281.7	2538.0	4956.4	6406.6
100	380	10580	1762.2	1412.3	3142.3	4717.4	6570.1
100	390	10590	1762.2	1266.7	2326.5	4392.9	6296.7
100	400	10600	1662.8	1372.7	2774.3	4832.1	6252.7
100	410	10610	1729.2	1463.8	1452.6	4937.7	6588.0
100	420	10620	1873.6	1502.6	2075.1	4677.6	6153.0
100	430	10630	2493.1	1613.3	2075.1	4558.1	6076.8
100	440	10640	1652.6	1290.1	1644.6	5018.2	6572.0
100	450	10650	1545.7	1504.0	2282.6	4608.6	6279.9
100	460	10660	2502.6	1534.1	2036.7	4698.2	6414.8
100	470	10670	1565.8	1498.4	2282.6	4772.7	6325.8
100	480	10680	1660.6	1516.0	2466.8	4590.6	6903.9
100	490	10690	1642.4	1175.0	1885.4	4989.0	7486.1
100	500	10700	1439.7	1530.5	2490.1	4772.2	6302.5
100	510	10710	1852.6	1379.3	2075.1	4474.8	5934.6
100	520	10720	2348.9	1237.4	2513.8	4630.7	6223.0
100	530	10730	1964.5	1379.5	1614.4	4479.5	6617.9
100	540	10740	1542.9	1512.3	1885.4	4909.6	6092.7
100	550	10750	1769.9	1631.6	2094.8	5058.0	6579.5
100	560	10760	1539.8	1661.1	2538.0	4631.1	6070.6
100	570	10770	1960.2	1711.6	2282.6	4683.8	5921.1
100	580	10780	1824.6	1616.6	2326.5	4601.2	6558.8
100	590	10790	1868.7	1482.2	3527.6	4395.3	6407.1
100	600	10800	2092.3	1393.6	3055.0	4483.3	6456.7
		n	3	3	2	3	2

με = applied microstrain

N = cycle

∑N = cumulative cycle

n = sample size

B.7 Tube Suction Test

CC (%)	0%	1%		2%		3%	
Days	DV	DV	m (g)	DV	m (g)	DV	m (g)
0	3.32	3.55	2326.6	3.85	2401.7	4.76	2459.0
1	7.10	8.47	2430.2	6.39	2491.9	5.37	2519.3
2	7.25	8.50	2431.1	7.49	2493.7	5.85	2531.0
3	7.31	8.52	2430.4	7.62	2494.5	7.77	2538.7
4	7.53	8.59	2430.1	7.86	2495.0	7.87	2540.2
5	7.53	8.76	2429.8	7.96	2495.1	8.22	2541.9
6	7.81	8.79	2430.8	8.01	2495.5	8.27	2541.1
7	7.91	8.85	2430.9	8.12	2496.0	8.35	2542.1
8	8.09	8.64	2430.7	8.23	2496.3	8.27	2542.1
9	8.14	8.70	2430.1	8.26	2496.5	8.35	2541.9
UCS (MPa)	-	3.61		5.63		9.28	
UCS_{Dry} (MPa)	-	7.94		9.62		15.33	
n	3	3		6		3	

CC (%)	4%		5%		6%	
Days	DV	m (g)	DV	m (g)	DV	m (g)
0	4.49	2394.4	5.09	2383.1	4.98	2417.9
1	4.95	2457.2	5.70	2440.2	5.50	2466.1
2	6.09	2467.9	5.93	2448.5	5.69	2471.8
3	6.83	2473.4	6.13	2453.8	5.71	2474.7
4	6.85	2476.5	6.74	2457.7	5.76	2480.0
5	7.03	2478.7	7.23	2459.9	6.01	2481.4
6	7.27	2481.7	7.67	2463.2	6.14	2482.9
7	7.60	2482.8	8.06	2464.2	6.23	2484.5
8	7.90	2484.2	8.30	2465.4	6.42	2486.0
9	8.05	2486.0	8.47	2466.4	6.47	2487.1
UCS (MPa)	11.06		13.07		16.08	
UCS_{Dry} (MPa)	15.84		15.98		19.57	
n	7		4		3	

CC = cement content
DV = dielectric value
m = mass
UCS = unconfined compressive strength at end of TST
UCS_{Dry} = unconfined compressive strength of dry control specimen
n = sample size

B.8 Nitrogen Adsorption Test Results

24 hours cured specimens									
1%		2%		3%		4%		5%	
w (Å)	V (cm^3/g. Å)	w (Å)	V (cm^3/g. Å)	w (Å)	V (cm^3/g. Å)	w (Å)	V (cm^3/g. Å)	w (Å)	V (cm^3/g. Å)
396.6	4.99E-06	385.1	4.49E-06	381.3	4.47E-06	386.8	1.39E-05	390.9	1.55E-05
210.2	1.09E-05	209.7	1.10E-05	211.1	1.69E-05	223.6	3.36E-05	221.7	3.87E-05
139.4	1.15E-05	139.3	1.16E-05	140.0	1.86E-05	140.7	3.47E-05	140.4	4.24E-05
103.2	1.02E-05	103.2	1.17E-05	103.6	2.18E-05	103.2	3.40E-05	103.0	4.33E-05
81.2	8.96E-06	81.1	1.27E-05	81.4	2.83E-05	81.4	3.54E-05	81.2	4.91E-05
66.4	7.22E-06	66.3	1.36E-05	66.6	3.06E-05	66.6	3.88E-05	66.5	5.86E-05
55.7	5.70E-06	55.7	1.37E-05	56.0	3.02E-05	56.0	4.25E-05	55.9	6.82E-05
47.7	2.76E-06	47.6	1.38E-05	47.9	3.01E-05	47.9	4.24E-05	47.8	7.82E-05
41.3	1.39E-06	41.2	1.45E-05	41.6	3.13E-05	41.5	4.11E-05	41.5	8.14E-05
36.1	2.95E-07	36.0	1.45E-05	36.4	3.05E-05	36.4	4.06E-05	36.3	8.48E-05
31.8	8.37E-07	31.7	1.69E-05	32.0	3.32E-05	32.0	3.97E-05	32.0	8.53E-05
28.0	2.85E-06	28.0	2.00E-05	28.3	3.58E-05	28.3	3.89E-05	28.2	8.79E-05
24.8	3.84E-06	24.7	2.33E-05	25.1	3.56E-05			25.0	8.57E-05
21.9	3.53E-06	21.8	2.27E-05	22.2	3.61E-05			22.1	7.31E-05
19.2	2.95E-06	19.2	2.29E-05	19.5	3.44E-05			19.5	5.59E-05

7 days cured secimens									
1%		2%		3%		4%		5%	
w (Å)	V (cm^3/g. Å)	w (Å)	V (cm^3/g. Å)	w (Å)	V (cm^3/g. Å)	w (Å)	V (cm^3/g. Å)	w (Å)	V (cm^3/g. Å)
414.2	2.56E-05	417.4	9.83E-05	354.5	4.63E-05	376.1	4.30E-05	398.7	4.81E-05
225.0	3.15E-05	224.3	1.02E-04	213.8	6.08E-05	219.7	6.81E-05	215.0	8.67E-05
146.5	2.52E-05	146.2	8.03E-05	143.3	4.22E-05	145.1	4.35E-05	143.2	5.78E-05
103.5	2.30E-05	103.3	7.55E-05	102.5	3.90E-05	103.3	3.82E-05	102.4	5.51E-05
80.5	1.98E-05	80.3	6.46E-05	80.0	3.48E-05	80.5	3.19E-05	80.0	4.99E-05
65.9	1.63E-05	65.8	5.45E-05	65.7	3.12E-05	66.0	2.63E-05	65.6	4.52E-05
55.3	1.25E-05	55.3	4.71E-05	55.3	2.72E-05	55.5	2.03E-05	55.2	4.13E-05
47.3	9.70E-06	47.2	4.41E-05	47.3	2.41E-05	47.5	1.51E-05	47.2	3.69E-05
41.0	7.43E-06	40.9	4.43E-05	40.9	2.24E-05	41.1	1.09E-05	40.9	3.39E-05
35.8	6.34E-06	35.7	4.04E-05	35.8	2.15E-05	36.0	8.53E-06	35.7	3.25E-05
31.5	6.43E-06	31.4	4.14E-05	31.4	2.14E-05	31.6	8.17E-06	31.4	3.07E-05
27.8	6.80E-06	27.7	4.27E-05	27.7	2.14E-05	27.9	6.31E-06	27.7	2.83E-05
24.5	6.28E-06	24.5	3.65E-05	24.5	1.91E-05	24.7	2.93E-06	24.5	2.40E-05
21.7	3.87E-06	21.6	2.76E-05	21.6	1.40E-05	21.8	1.74E-06	21.6	1.61E-05
19.0	2.01E-06	18.9	1.63E-05	19.0	1.01E-05			18.9	7.58E-06

w = pore width
V = pore volume

B.9 Linear Shrinkage Test

LS (%)	1.84	1.56	0.04	-0.08	-0.16
n	1	1	1	1	1

LS = Linear shrinkage
n = sample size

B.10 Wheel Tracking Test

CC	2%	4%	6%	CC	2%	4%	6%	CC	2%	4%	6%
Runs	Δ (mm)			Runs	Δ (mm)			Runs	Δ (mm)		
1	0.000	0.000	0.000	1150	0.360	0.483	0.558	3100	0.440	0.583	0.698
10	0.050	0.077	0.045	1200	0.360	0.490	0.563	3150	0.440	0.583	0.703
20	0.080	0.130	0.080	1250	0.360	0.493	0.568	3200	0.450	0.587	0.708
30	0.100	0.160	0.105	1300	0.370	0.493	0.570	3250	0.450	0.590	0.710
40	0.110	0.190	0.145	1350	0.370	0.500	0.575	3300	0.450	0.590	0.713
50	0.130	0.210	0.238	1400	0.370	0.503	0.583	3350	0.450	0.590	0.715
60	0.140	0.227	0.260	1450	0.380	0.507	0.585	3400	0.450	0.590	0.718
70	0.150	0.237	0.270	1500	0.380	0.510	0.588	3450	0.450	0.590	0.718
80	0.160	0.253	0.283	1550	0.390	0.513	0.590	3500	0.460	0.597	0.723
90	0.170	0.270	0.293	1600	0.390	0.517	0.598	3550	0.460	0.597	0.723
100	0.180	0.283	0.300	1650	0.390	0.520	0.603	3600	0.460	0.597	0.728
110	0.180	0.290	0.310	1700	0.400	0.530	0.608	3650	0.460	0.597	0.728
120	0.190	0.300	0.318	1750	0.400	0.530	0.615	3700	0.460	0.597	0.730
130	0.200	0.307	0.325	1800	0.400	0.533	0.618	3750	0.460	0.597	0.730
140	0.200	0.313	0.330	1850	0.400	0.537	0.623	3800	0.460	0.597	0.733
150	0.210	0.320	0.338	1900	0.400	0.537	0.628	3850	0.460	0.597	0.733
160	0.210	0.330	0.340	1950	0.410	0.540	0.628	3900	0.470	0.597	0.733
170	0.220	0.330	0.348	2000	0.410	0.547	0.628	3950	0.470	0.600	0.735
180	0.220	0.337	0.355	2050	0.410	0.547	0.635	4000	0.470	0.597	0.733
190	0.220	0.343	0.355	2100	0.410	0.547	0.643	4050	0.470	0.600	0.738
200	0.230	0.347	0.365	2150	0.420	0.550	0.643	4100	0.470	0.600	0.738
250	0.240	0.360	0.390	2200	0.420	0.550	0.650	4150	0.470	0.600	0.735
300	0.250	0.380	0.408	2250	0.420	0.557	0.650	4200	0.470	0.600	0.738
350	0.260	0.387	0.430	2300	0.420	0.560	0.655	4250	0.470	0.603	0.738
400	0.270	0.400	0.448	2350	0.420	0.560	0.660	4300	0.470	0.600	0.740
450	0.280	0.407	0.458	2400	0.420	0.563	0.663	4350	0.470	0.603	0.743
500	0.290	0.417	0.473	2450	0.430	0.563	0.665	4400	0.470	0.607	0.740
550	0.290	0.423	0.483	2500	0.430	0.563	0.668	4450	0.480	0.607	0.745
600	0.300	0.430	0.495	2550	0.430	0.563	0.673	4500	0.480	0.607	0.745
650	0.310	0.433	0.500	2600	0.430	0.570	0.675	4550	0.480	0.610	0.743
700	0.320	0.437	0.508	2650	0.430	0.573	0.680	4600	0.480	0.607	0.745
750	0.320	0.443	0.513	2700	0.430	0.573	0.680	4650	0.480	0.610	0.745
800	0.330	0.450	0.520	2750	0.430	0.573	0.683	4700	0.480	0.610	0.743
850	0.330	0.460	0.525	2800	0.440	0.577	0.685	4750	0.480	0.610	0.745
900	0.340	0.460	0.530	2850	0.440	0.577	0.688	4800	0.480	0.610	0.748
950	0.340	0.467	0.538	2900	0.440	0.580	0.688	4850	0.480	0.610	0.745
1000	0.350	0.473	0.540	2950	0.440	0.580	0.693	4900	0.480	0.610	0.750
1050	0.350	0.477	0.545	3000	0.440	0.580	0.693	4950	0.480	0.610	0.748
1100	0.350	0.480	0.555	3050	0.440	0.580	0.695	5000	0.480	0.610	0.750
								n	3	4	4

CC = cement content
Δ = erodibility index
n = sample size