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Effects of Aggregate Properties on Strength Characteristics of the Foamed Bitumen Mixture with Relation to Western Australian Roadway Construction

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DECLARATION

This thesis 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 this thesis contains no material previously published by any other person except where due acknowledgment has been made.

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

Refereed Journal Papers:

1. **Huan, Yue**, Siripun, Komsun, Jitsangiam, Peerapong and Nikraz, Hamid. 2010. "A Preliminary Study on Foamed Bitumen Stabilisation for Western Australian Pavements". Scientific Research and Essays. Vol. 5(23), pp. 3687-3700.
2. Jitsangiam, Peerapong and **Huan, Yue** and Siripun, Komsun and Leek, Colin and Nikraz, Hamid. 2012. "Effect of Binder Content and Active Filler Selection on Foamed Bitumen Mixtures: Western Australia Experience". International Journal of Pavement Research and Technology Vol. 5 (6): pp. 411-418.

Refereed Conference Papers:

1. **Huan, Yue** and Jitsangiam, Peerapong and Nikraz, Hamid. 2011. "Effects of Active Filler Selection on Foamed Bitumen Mixture in Western Australia". Applied Mechanics and Materials. 90-93: pp. 457-465.
2. **Huan, Yue** and Jitsangiam, Peerapong and Nikraz, Hamid and Grant, Rhys. 2012. "Mechanical characteristics of foamed bitumen mixtures in Western Australia". Advances in Transportation Geotechnics 2: pp. 309 -314.

3. **Huan, Yue**. and Jitsangiam, Peerapong. and Nikraz, Hamid. and Siripun, Komsun. 2012. "The Effects of Compaction Methods on Tensile Strength of Foamed Bitumen Mixture". Proceedings of the 11th Australia New Zealand Conference on Geomechanics (ANZ), Jul 15-18 2012, pp. 834-839. Melbourne, Vic.: Australian Geomechanics Society and New Zealand Geotechnical Society.
4. **Huan, Yue** and Jitsangiam, Peerapong and Siripun, Komsun and Nikraz, Hamid. 2012. "Effect of Aggregate Fine Contents on Foamed Bitumen Stabilisation", in the proceedings of the ISAP 2012 International Symposium on Heavy Duty Asphalt Pavements and Bridge Deck Pavements, May 23-25 2012. Nanjing, China: International Society for Asphalt Pavements (ISAP).

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ABSTRACT

Recently, pavement rehabilitation has become crucial in the context of growing political awareness of environmental issues, such as the finite availability of virgin natural resources, as well as the increasing costs associated with disposal of recycled waste materials. A viable solution to this issue is to recycle roadway materials within a pavement system upon the termination of their practical life cycle and allow them to be reused for the duration of their renewable life cycle. Many techniques have been utilised in this rehabilitation method, including effective cold-recycling with foamed bitumen. Foamed bitumen is produced by injecting small amounts of cold water and air into a hot bitumen mix, typically at about 180°C. The effect is that the water immediately turns to steam, resulting in an increase in volume of up to 15 times the original bitumen volume. This makes the bitumen much less viscous, increasing its workability for spraying and mixing with raw aggregate.

With the increased popularity of foamed bitumen stabilisation, it is first necessary to understand the basic characteristics of the materials stabilised with foamed bitumen, before exploring the myriad aspects which need to be optimised in order to fully refine a design approach. In Western Australia, the commonly used aggregates for road pavement construction are crushed rock base and crushed limestone, used as base course and sub-base course materials respectively. Basically, aggregate composition and gradation are two significant aggregate properties affecting the efficacy of foamed bitumen treatment.

This thesis aims to determine an effective mix design for foamed bitumen for use in Western Australian conditions, based on mechanical testing using typical Western Australian aggregates, while altering the percentage of crushed rock base and crushed limestone as well as the fine contents and grading curves within the aggregate mix. The main laboratory experimental work was undertaken in the Curtin University Laboratory under Western Australian conditions, where affecting factors

including mixing moisture content, compaction methods and curing methods as such were modified and constrained.

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Last, but not least, I wish to avail myself of this opportunity to express a special word of thanks to my beloved family and friends for their support, encouragement and understanding during my study overseas. In particular, I am extremely indebted to my wife, Xiaoxia for her unconditional assistance, encouragement, patience and endless support. The birth of our daughter, Ella brings me so much joy and happiness.

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LIST OF ABBREVIATIONS/NOTATIONS

AASHTO	= American Association of State Highway and Transportation Officials
CRB	= Crushed Rock Base
CLS	= Crushed Limestone
CISR	= Cold In-Situ Recycling
ER	= Expansion Ratio
FB	= Foamed Bitumen
ITS	= Indirect Tensile Strength
ITM_R	= Indirect Tensile Resilient Modulus
MRWA	= Main Roads Western Australia
MDD	= Maximum Dry Density
MMC	= Mixing Moisture Content
MC	= Moisture Content
OMC	= Optimum Bitumen Content
PI	= Plasticity Index
PSD	= Particle Size Distribution
QDTMR	= Queensland Department of Transport Main Roads
UCS	= Unconfined Compressive Strength
$\tau 1/2$	= Half-Life

1. INTRODUCTION

This chapter aims to report on the background of the in-situ foamed bitumen stabilisation process and provide an overview of foamed bitumen mix design and performance, identifying deficiencies in current knowledge and potential opportunities for improving design certainty.

1.1. Background

The road network is vital for the transportation of people and goods to destinations, and sustaining a defined level of service requires adequate maintenance and for heavily trafficked roads, usually rehabilitation at some stage in the life of the pavement. Maintenance is defined as the process of maintaining the design function, whereas rehabilitation is the process of increasing a pavement's structural capacity due to the increased demand that develops during a pavement's life.

The overall aim of a road agency is to maintain more roads with less money, as well as improving environmental sustainability by minimising landfill waste and maximising the use of recycled and secondary aggregates. Western Australia has a road network of over 140,000km with an estimated replacement value of $27 billion, and the maintenance of these roads is a task which requires special attention (Main Roads Western Australia 2011). Eventually, heavily trafficked roads and pavement structures will reach the end of their design life and need rehabilitation, either prior to or after failure has occurred. Community expectations are that all areas of human endeavour should strive for improved sustainability; accordingly, road agencies aim to adopt more sustainable rehabilitation methods with reduced environmental impact. Increased landfill charges and the effect of the waste levy applied in many jurisdictions, as well as the shortage of virgin aggregates, are leading to the examination of the approaches to pavement engineering (Khweir 2007).

One method of pavement rehabilitation is cold in-situ recycling (CISR), whereby the existing pavement is combined with a stabilising agent on site at ambient temperature. The most common stabilising agents are foamed bitumen, bitumen emulsion, cement, lime, other pozzolanic materials or chemical additives.

Foamed bitumen is not a new concept – road agencies around the world have investigated or adopted this process since it was initially proposed by Csanyi in the mid-1950s at Iowa State University in North America, using steam to generate the foaming action (Csanyi 1957). In 1968, Mobil Oil Australia modified the original process by replacing steam with cold water, enabling foamed bitumen to be practically and widely used in the field. Incorporating the CISR process in road rehabilitation provides recognised cost and environmental benefits (Eller & Olson 2009).

Foamed bitumen is produced by injecting small amounts of cold water and air into hot bitumen, which is typically at about 180°C. The effect is that the water immediately turns to steam, resulting in an increase in volume of up to 15 times the original bitumen volume. This reduces bitumen viscosity, enabling the bitumen to be mixed with existing pavement materials at ambient temperature, hence eliminating the requirement for bitumen emulsifiers or cutbacks to achieve the reduced viscosity. As the foam collapses, most of the water is lost as steam. The resulting bitumen has properties similar to the original bitumen and is well dispersed through the aggregate in very small droplets. The bitumen droplets are attracted to and coat the finer particles, forming a uniform matrix that effectively binds the mixture of particles together (AustStab 2002).

1.2. Challenges in the Application of Foamed Bitumen Stabilisation

Numerous articles and reports have been published, recording, analysing and discussing various aspects of the foamed bitumen stabilisation method, and providing general information on how to understand and successfully use this process both in the field and in the laboratory. However, due to a lack of standardised mix designs, procedures and testing methods, different researchers and contractors usually form their own methods in terms of sample preparation, compaction effort, curing regime and mechanical testing programs. This results in confusion for those agencies who are interested in, but hesitant about adopting the foamed bitumen process.

The divergence in theory is demonstrated by the philosophy adopted by different agencies; for example, researchers in South Africa insist that the foamed bitumen mix remains in a granular state and only improves behavioural characteristics, rather than transforms the mixture to asphalt like form, where in Australia, researchers tend to believe that stabilised material becomes a fully bound material subject to fatigue failure (Academy, 2009; Vorobieff, 2005).

1.3. Opportunities offered by Foamed Bitumen Stabilisation

Australian society is becoming increasingly concerned about environmental issues including energy consumption and carbon emissions. A recent example of this is the proposal for and passing of a carbon tax bill, where revenue generated from the tax would fund research and construction projects with a greener focus. The road construction industry is also slowly embracing sustainability as evidenced by the use of recycled materials for road base and asphalt manufacture and warm mix asphalt technology. Foamed bitumen stabilisation has significant advantages in terms of environmental benefits, including minimal heat input, and reductions in transport requirements, material wastage, noise, dust and traffic disruptions. Whilst it has many advantages, the downside of the process is that it relies on bitumen, which is an expensive and finite resource requiring significant energy input. Optimising the bitumen content is essential in order to minimise the environmental footprint.

1.4. Key Objectives

The lack of a standardised mix design method in Australia gives this project significance. Uncertainty in laboratory mix design methods decreases confidence in the implementation of foamed bitumen stabilisation in the field. The development of an appropriate mix design under controlled laboratory conditions can also make necessary contributions to the pavement design process.

The purpose of this project is to expand knowledge of foamed bitumen mixture characteristics as well as the mix design procedure for Western Australia in particular, and to answer the question of how the mechanical strength of foamed bitumen mixture develops when the aggregate composition and gradation are altered.

Key objectives for the project are as follows:

- Develop a fundamental knowledge of foamed bitumen mixture as well as considerable mix design parameters.
- Evaluate current findings on foamed bitumen mix design by reviewing national and international methods.
- Conduct laboratory experimental trials on aggregate composition and gradation under Western Australian conditions.
- Determine an appropriate aggregate selection based on the results of the laboratory experiments.
- Provide recommendations for changes in existing methods in order to guide field construction.

1.5. Scope

In order to properly establish a foamed bitumen mix design, many aspects should be considered. Based on the aforementioned objectives, this research project has only considered the raw aggregate properties, such as composition and gradation. Other properties that may affect foamed bitumen mix design and which are considered to remain constant include optimum bitumen content, optimum active filler content and type, mixing moisture content, compaction methods and curing methods.

Locally manufactured Bitumen C170, and locally sourced 19mm crushed road base and 19mm limestone were used for preparation of the foamed bitumen mix specimens in the laboratory. A WLB10S foamed bitumen machine was used to produce foamed bitumen to be mixed with the aggregates. On completion of the foaming process, the samples were compacted and cured under the same conditions and further mechanical tests were undertaken.

1.6. Structure of the Thesis

Chapter 1 introduces a brief background to the study and presents some challenges and opportunities as well as statements of the key objectives.

Chapter 2 reviews the literature on international and national methods for foamed bitumen mix design.

Chapter 3 describes the research methodology and laboratory experimental design used to provide the required data.

Chapter 4 discusses the optimum aggregate composition for use in field conditions. In this chapter, different percentages of parent raw aggregate including locally sourced crushed rock base and crushed limestone and bitumen content are investigated.

Chapter 5 continues the research on the effects of aggregate gradation based on the parent aggregate determined in Chapter 4 in terms of the mechanical strength development of foamed bitumen mixtures.

Chapter 6 contains the final conclusion and recommended mix design considerations.

2. BACKGROUND AND LITERATURE REVIEW

2.1. Overview

This chapter explores common pavement types and their fundamental properties by reviewing the available literature. The pavement functional requirements are subsequently described in relation to pavement structures and the materials typically used, with a focus primarily on flexible pavement. Also mentioned are typical causes and types of pavement failure and various rehabilitation techniques including the foamed bitumen method, the characteristics of which will be explained in more detail.

The literature review concentrates on information from various national and international guidelines on foamed bitumen stabilisation mix design parameters. The Australian guidelines reviewed include the Austroads Technical Report and the Queensland Department of Transport and Main Roads (QDTMR), and the international guidelines are the Asphalt Academy’s Technical Guideline (South Africa) and the Full Depth Pavement Reclamation with Foamed Asphalt (California). It should be noted that some of the design parameters in the Austroads report are based on the QDTMR report, and the QDTMR report has been included as an additional reference to the Austroads report.

The review is split up into six individual topics which are the main mix design parameters required for foamed bitumen and are mentioned in most literatures. The topics include:

- Aggregate grading
- Foamed bitumen characteristics
- Secondary binder selection
- Mixing moisture content
- Laboratory compaction

- Laboratory curing regime

2.2. Pavement Types and Functions

There are two main types of engineered road structures commonly available today which can be classified as being either flexible or rigid. The term ‘flexible pavement’ can be applied to all pavement structures other than those described as rigid pavements (Austroads 2005).

2.2.1. Rigid Pavement

Rigid pavement or concrete pavement systems usually comprise a layer of Portland cement concrete which is typically supported by a sub-base layer on top of the sub-grade (or natural soil) (Nikraz 1998). Figure 2-1 illustrates the layout of a typical rigid pavement. The concrete may be either reinforced or unreinforced, depending on the design controls for shrinkage cracking. The high modulus of elasticity and rigidity of concrete, when compared with other road making materials, provide a concrete pavement with a reasonable degree of flexural or beam strength. Due to high construction costs and timing, this pavement type is not common in Western Australia.

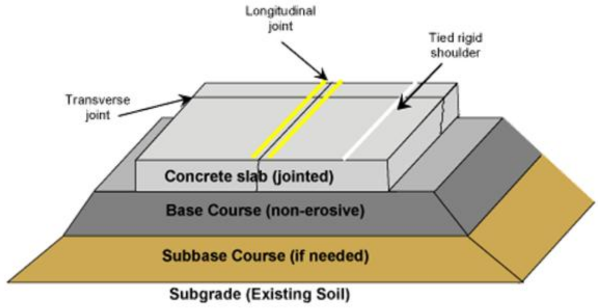

Figure 2-1: A typical section of rigid pavement

2.2.2. Flexible Pavement

A flexible pavement may consist of unbound granular layers or a combination of both bound and unbound granular layers. Basically, flexible pavements include a wearing surface, base and sub-base sitting on top of a soil foundation (sub-grade) as shown diagrammatically in Figure 2-2 (Austroads 2005).

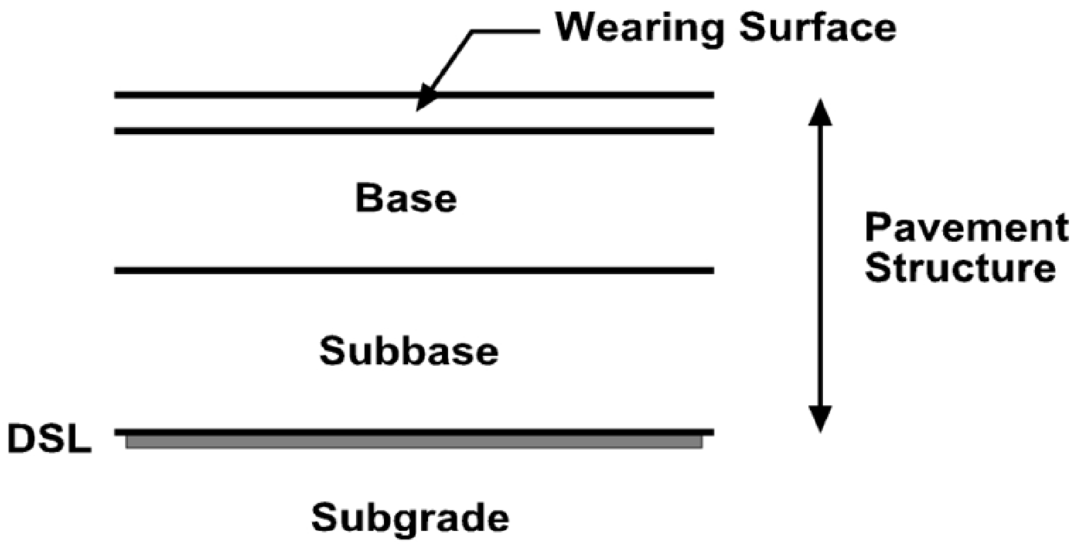

Figure 2-2: Flexible pavement structure (Austroads 2005)

Wearing or surface course: generally applied to the granular base to withstand abrasion from traffic, waterproof the pavement and prevent dust generation as well as providing a safe and functional riding surface (Austroads 2005). It usually consists of one or more layers of bituminous surfacing, either a sprayed seal or asphalt.

Base Course: the main load carrying course in the pavement (Austroads 2005). It may consist of one or more layers of fine crushed rock, natural gravels, broken stone, stabilised or improved material asphalt or other material. This layer is the main focus of this research project.

Sub-base Course: laid between the sub-grade and the base course. Its purpose is to make up the additional pavement thickness required over the sub-grade, to prevent intrusion of the sub-grade into the base course and/or to provide a working surface over a weak sub-grade upon which the remainder of the pavement can be constructed.

Due to this layer being lower in the pavement structure, it is often (but not necessarily always) made up of lower quality material than the base course.

Sub-grade: a foundation of natural earth, or possibly compacted selected fill soil made as part of the earthworks operation.

2.2.3. Pavement Functions

The basic function of the pavement is to withstand loads under different seasonal environmental conditions without deforming or cracking. The function of the different layers within a flexible pavement structure is to spread the load on the surface and reduce the intensity with depth (Mallick and El-Korchi 2009). Figure 2-3 demonstrates how the two types of pavements withstand the loading distribution.

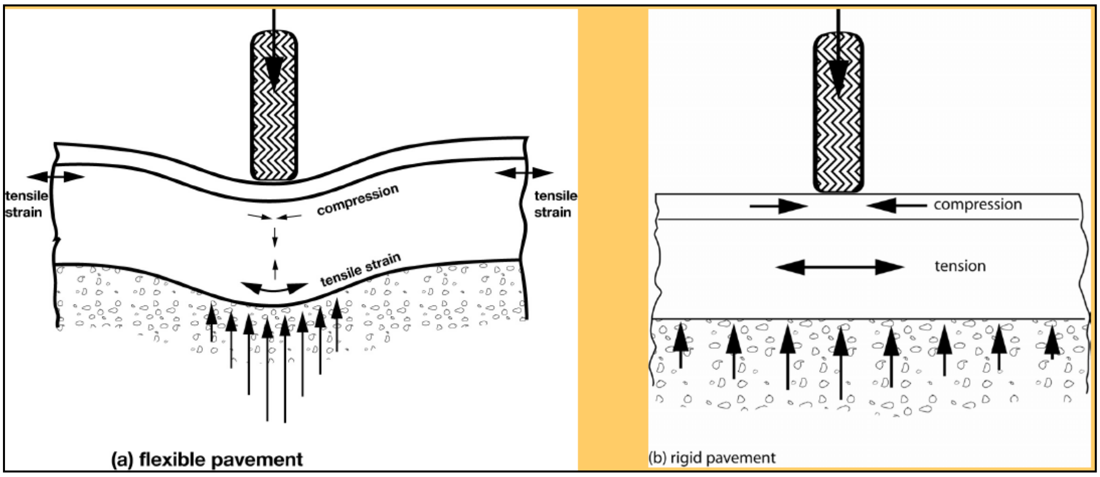

Figure 2-3: Loading behaviour of pavement (Austroads 2005)

2.3. Pavement Materials

Flexible pavement materials can be classified into four categories based on the behaviour of the materials under applied loading. These four categories include unbound granular materials, modified granular materials, bound material and stabilised materials. Table 2-1 gives a summary of the materials and their characteristics.

2.3.1. Unbound Granular Materials

Unbound granular materials consist of gravels or crushed rocks with a grading that makes them mechanically stable, workable and compactable (Austroads 2005). Their performance is governed by their shear strength, stiffness and resistance to material breakdown under construction traffic loading (Austroads 2005).

2.3.2. Modified Granular Materials

Modified granular materials are classified as granular materials which have a small amount of stabilising binder added to them in order to improve their stiffness or other deficiencies. An example is the inclusion of chemically modified materials and cement, lime, lime/fly ash or slag modified materials (Austroads 2005).

2.3.3. Bound Materials

Bound materials consist of particles which are strongly bound together by substances such as lime, cement or bitumen. When the materials are loaded, they behave as a continuous system which enables the system to develop tensile stresses without the materials separating (Austroads 2005).

2.3.4. Stabilised Materials

Stabilised materials consist of pavement material with a chemical binding agent mixed into it. The mixture is then compacted and cured in order to form a bound pavement layer (Austroads 2005). The most commonly used binders are lime, Portland cement, blended cement or bitumen. The chosen binder is added in order to produce a bound layer with high tensile strength.

Table 2-1: Pavement material categories and characteristics for flexible pavements (Austroads 2005)

Characteristics	Pavement Material Category		
	Unbound granular	**Modified granular**	**Bound (Stabilised)**
Material types	• Crushed rock • Gravel • Soil aggregate • Mechanically stabilised materials	• Chemically modified materials • Cement, lime, lime/fly ash or slag modified materials	• Lime stabilised materials • Cement stabilised materials • Bitumen stabilised materials • Lime/fly ash stabilised materials • Slag stabilised materials • Slag/lime stabilised materials
Behaviour characteristics	• Development of shear strength through particle interlock • No significant tensile strength	• Development of shear strength through particle interlock • No significant tensile strength	• Development of shear strength through particle interlock and chemical bonding • Significant tensile strength
Distress modes	• Deformation through shear and densification • Disintegration through breakdown	• Deformation through shear and densification • Disintegration through breakdown	• Cracking development through shrinkage, fatigue and over stressing • Erosion and pumping in the presence of moisture
Input parameters for design	• Modulus • Poison's ratio • Degree of anisotropy	• Modulus • Poison's ratio • Degree of anisotropy	• Modulus • Poison's ratio
Performance criteria	• Current materials specifications	• Current materials specifications	• Fatigue relationships

2.4. Flexible Pavement Failure Mechanism

Flexible pavement stress assessment is highly complex due to the layered nature of the pavement structure and the unpredictability of loads acting on it, as well as poor material selection or design. Figure 2-4 illustrates how the different layers behave during the failure process. The key to proper maintenance of flexible pavements is to understand the reasons behind these failures and carry out appropriate action to rectify the situation.

The following is a list of potential flexible pavement failures and their possible causes. The different failure types are shown in the photographs in Figure 2-5.

- Corrugation: instability of base layer, excessive moisture, poor mix design.
- Fatigue cracking: inadequate structural (base, sub-base or sub-grade) support and design, improper drainage, inadequate compaction.
- Depression: consolidation of sub-grade which causes settlement, poor sub-base and sub-grade construction (e.g. inadequate compaction).
- Rutting: inadequate compaction during construction, poor mix design, inadequate strength or thickness of surface or base layer which leads to settlement.

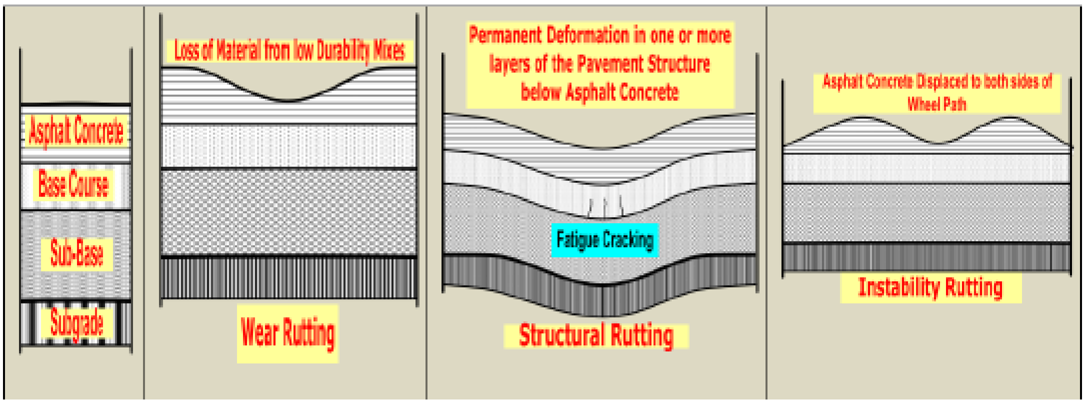

Figure 2-4: Cross section of pavement outlining failure types due to fatigue cracking and rutting (after White et al. 2002)

Figure 2-5: Common flexible pavement defects ([a]PTCA 2005, [b]Petts 1994)

2.5. Pavement Stabilisation Methods

In many industries there is currently a strong emphasis on sustainability. For the transport and pavement industry this means that completely replacing road pavements with new materials is no longer environmentally nor economically viable, and appropriate stabilisation methods have drawn greater consideration. Stabilisation is the treatment of a road pavement material to improve or correct a known deficiency in order to enable the material to better perform its function within the pavement structure (NAASRA 1970). The observation of failures in pavement sections leads to the exploration and implementation of different stabilisation methods rather than reconstruction, in order to increase the road service life. The typical types of stabilisation used in flexible pavement construction include mechanical, Portland cement, lime or bituminous stabilisation (NAASRA 1970) as well as bitumen in the form of foamed bitumen or bitumen emulsions.

2.5.1. Mechanical Stabilisation

Mechanical stabilisation is a process of improving the particle size distribution and/or plastic properties of a material by blending it with one or more selected materials. Common examples include the blending of granular and clay soils, or the addition of fine material for low plasticity to granular materials which lack fine material. By blending such materials, a marked improvement in strength, abrasion resistance, imperviousness and compactability can often be achieved (NAASRA 1970).

2.5.2. Portland Cement Stabilisation

Portland cement is one of the most widely used stabilisers. A proportion of cement is mixed with the material to be treated at the required moisture content. Compaction needs to occur as soon as possible due to the setting characteristics. The surface then needs to be protected to ensure that no loss of moisture occurs during the curing stage. This process is best suited to materials with low plasticity. Cement stabilisation can be an expensive method, which requires precise design and construction techniques in order to achieve optimum results (NAASRA 1970). The addition of cement to a pavement base material generally results in shrinkage cracking which reflects through to the wearing surface and affects the appearance of the pavement surface. As a result, the wearing surface is no longer waterproof and will eventually reduce its service life (NAASRA 1970).

2.5.3. Lime Stabilisation

Hydrated lime and quicklime are used as a stabilising treatment for more plastic materials such as clay soils, where the plasticity index exceeds 10. The addition of lime to clay soils causes flocculation of the clay, which increases the strength of the clay (NAASRA 1970). Lime stabilisation is more successful in warmer climates as the increase in strength is accelerated due to the higher temperatures. Lime stabilisation generally occurs in two stages, with the first stage including the mixing of clay and lime, which is where the flocculation occurs. The second stage includes further mixing with or without additional lime. This second stage can occur a few hours or a few days after the first stage (NAASRA 1970).

2.5.4. Bituminous Stabilisation

Bituminous materials such as cut back bitumen, foamed bitumen, road oil, and road tar and bitumen emulsion can be used to stabilise a wide range of soil types. Granular and less clayey soils are preferred for this method as the binding agent is easier to integrate. Bituminous materials are an effective stabilisation because of their visco-elastic and adhesive properties which improve the overall strength characteristics of the soil. Laboratory investigation is required for each proposal of stabilisation procedure to determine their effectiveness and the optimum conditions for their use in relation to mixing, curing and compacting (NAASRA 1970).

2.6. Foamed Bitumen Stabilisation

2.6.1. Description

Foamed bitumen (FB) is a mixture of air, water and bitumen. It is produced by injecting a small quantity of cold water and air into hot bitumen and spraying the resulting foam in a specially-designed expansion chamber. High pressure air is injected to ensure that water is fully vaporised during the foaming process as illustrated in Figure 2-6 (Ramanujam and Jones 2007). The steam generated produces an instantaneous expansion of the bitumen to about 15 times its original volume, resulting in a decrease in the bitumen's viscosity and an increase in its surface area. This makes it ideal for spraying and mixing with fine aggregates without adding bitumen emulsifiers to act as a viscosity reducer. The foam is mixed into the pavement material in the large mixing chamber of the stabiliser which includes a high speed rotating drum with specially designed teeth that break up the existing pavement and allow the bitumen to be dispersed into the existing pavement material. As the foam collapses, most of the water is lost in the form of steam and the foam is well dispersed through the aggregate in very small droplets without the original bitumen properties being altered. Since the foam collapses very quickly, vigorous mixing is required to effectively disperse the foamed bitumen throughout the material (AustStab 2002).

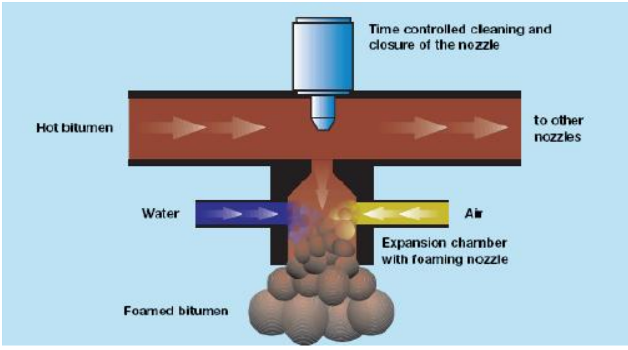

Figure 2-6: Foaming process (after Ramanujam and Jones 2007)

2.6.2. Characterisation of Foaming Quality

The parameters used to define the quality of the foamed bitumen are known as the expansion ratio (ER) and half-life. Figure 2-7 gives an example of how to determine these properties. As a general rule, a higher expansion ratio and half-life produces a higher quality foamed bitumen.

- Expansion ratio (ER) = Ratio of maximum expansive volume of foamed bitumen to initial volume of bitumen.
- Half-life ($\tau 1/2$) = Time measured in seconds for the foamed bitumen to subside from the maximum volume to half of the maximum volume.

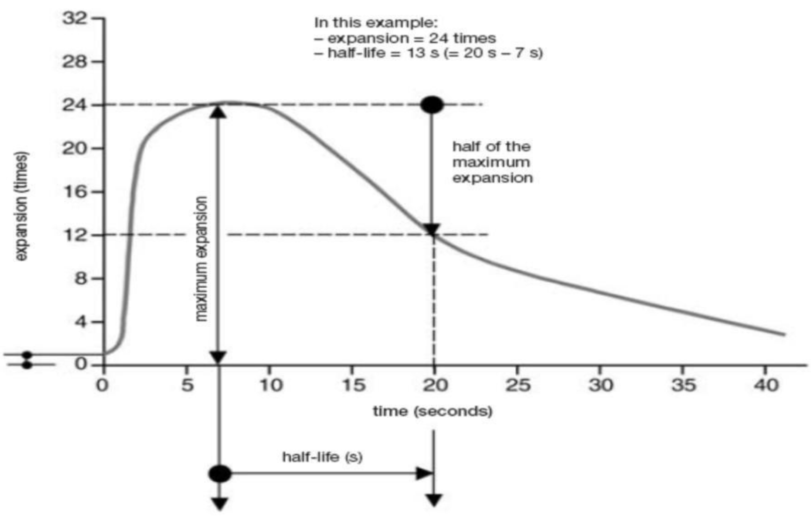

Figure 2-7: ER and $\tau 1/2$ relationship (after Austroads 2011)

2.6.3. Advantages and Disadvantages of Foamed Bitumen

The foamed bitumen process has both advantages and disadvantages. The following are some pros and cons gained from construction experiences (AustStab 2002, Ramanujam and Jones 2007).

- Advantages
  - Easy application.
  - Rapid construction time.
  - Reduced atmospheric pollution: little or no hydrocarbon emission from foamed bitumen.
  - Rapid strength gain, thus rehabilitated road can be trafficked immediately after construction without significant detrimental effects. This minimises traffic disruption and there are no temporary detours needed.
  - Increased suitability of aggregate types with which foamed bitumen binder is compatible, including a wide variety of virgin aggregate as well as sub-standard aggregates.
  - Reductions in secondary additive or binder requirements, resulting in reduced cost of binder and transport
- Disadvantages
  - Requires purpose-built equipment.
  - Requires high skill levels to produce quality and experienced operators to operate the equipment.
  - Hot bitumen is required for the foaming action to be successful.
  - Issues with sealing work have been encountered and extra attention is needed to prevent seal stripping problems.
  - Suitable grading of fines in the host material is required.
  - Relatively expensive compared to other forms of stabilisation.

2.7. Foamed Bitumen Mix Design Consideration

2.7.1. Aggregate Grading

Aggregate grading has been recognised as one of the most important elements in foamed bitumen mix design. A number of research papers have tested various grading limits to suit different conditions, establishing that a wide range of aggregates can be used, from crushed stone to silty sand.

2.7.1.1. Austroads Technical Report

The recommended particle size distribution obtained from the Austroads technical report, shown in Table 2-2, outlines the minimum and maximum percentages for different sieve sizes. It also mentions that the fines content is a critical property and that a minimum of 5% passing the 0.075mm sieve is recommended (Austroads 2011). In projects where the grading of coarse or fine materials falls outside of the recommended grading ranges, the materials can be adjusted by adding extra sized aggregates and/or fine material (Austroads 2011).

Table 2-2: Austroads specified grading range (Austroads 2011)

Sieve Size (mm)	Ideal Percentage Passing (%)	
	Minimum	Maximum
26.5	100	100
19.5	80	100
9.5	55	90
4.75	40	70
2.36	30	55
1.18	22	45
0.6	16	35
0.425	12	30
0.3	10	24
0.15	8	19
0.075	5	15

2.7.1.2. Asphalt Academy Guidelines

The Asphalt Academy proposed a new ideal grading requirement which established a less suitable zone alongside the ideal zone, as shown in Table 2-3. The Asphalt Academy has also noted that where necessary, additional aggregates can be blended with the mix to improve the grading if the aggregate grading falls in the less suitable zone. It was also emphasised that the dispersed bitumen droplets in the foamed bitumen only partially coat the larger aggregate, and that the mastic (combination of the filler, bitumen and water) produced “spot welds” on the coarser aggregate.

Table 2-3: Asphalt Academy specified grading range (Asphalt Academy 2009)

Sieve Size (mm)	Percentage Passing (%)	
	Ideal	Less Suitable
50	100	100
37.5	87–100	100
26.5	77–100	100
19.5	66–99	99–100
13.2	67–87	87–100
9.6	49–74	74–100
6.7	40–62	62–100
4.75	35–56	56–95
2.36	25–42	42–78
1.18	18–33	33–65
0.6	14–28	28–54
0.425	12–26	26–50
0.3	10–24	24–43
0.15	7–17	17–40
0.075	4–10	10–20

2.7.1.3. Californian Guidelines

The material grading specified in the Californian guidelines is for full depth reclamation which includes the underlying base, sub-base, and/or native material. The reclaimed material should conform to the range specified in Table 2-4 (Jones, Fu and Harvey 2009). The fines content mentioned of 5–12% does not include the active filler (Jones, Fu and Harvey 2009).

If the existing materials do not comply with the range noted in Table 2-4 then the following options can be used (Jones, Fu and Harvey 2009):

1) If the fines content is below 5%, extra non-plastic fines (in addition to the active filler) are required and can be obtained by increasing the recycling depth to incorporate more fines from the underlying layer, or by importing non-plastic fines from another source and spreading them onto the pavement prior to reclamation.
2) If fines content is from 12–15%, reclamation can proceed, however higher binder content may be required to ensure the additional fines are coated.
3) If fines content is from 15–20%, consideration should be given to decreasing the reclamation depth in order to reduce the quantity of fines. Alternatively,

additional tests can be carried out to determine the binder contents as they will need to be increased.

4) Projects where the fines content is greater than 20% should not be considered for foamed bitumen rehabilitation.

Table 2-4: Californian guidelines specified grading range (Jones, Fu and Harvey 2009)

Sieve Size (mm)	Ideal Percentage Passing (%)	
	Minimum	Maximum
50	100	100
37.5	90	100
19.0	50	85
4.75	25	45
0.6	10	25
0.075	5	12

2.7.1.4. Queensland Department of Transport Main Roads

The Queensland Department of Transport Main Roads (QDTMR) have recognised that the success of the stabilisation treatment is sensitive to the grading of the existing materials. The recommended grading limits, 5–20%, are outlined in Table 2-5 (Austroads 2011). The QDTMR have stated that for foamed bitumen stabilisation to be an effective rehabilitation technique, an adequate amount of fines are required to bind with the bitumen, as well as the material having some plasticity. The QDTMR have also specified that imported material can be used to achieve the desired grading where required.

Table 2-5: QDTMR specified grading range (after Austroads 2011)

Sieve Size (mm)	Ideal Percentage Passing (%)	
	Minimum	Maximum
26.5	100	100
19.5	80	100
9.5	55	90
4.75	40	70
2.36	30	55
0.425	12	30
0.075	5	20

2.7.1.5. Synthesis on Grading Requirements

Despite the different working conditions around the world, aggregate grading has been recognised as a universally important factor for consideration prior to foaming and needs a certain amount of attention. The fine content is the critical part in the grading and must fall within a specified range to ensure superior foaming quality. As previously noted, a range of 5–20% fine content is deemed necessary to meet basic requirements. Treatment is required when the fine content range falls outside of the boundaries normally achieved by importing some good quality aggregates to blend with existing materials.

2.7.2. Foamed Bitumen Characteristics

This section reviews the research conducted on optimum foamed bitumen content and foaming characteristics.

2.7.2.1. Austroads Technical Report

Austroads recommends testing three levels of bitumen content (2%, 3% and 4% bitumen by mass) in the laboratory to determine the optimum bitumen content. They also recommend a binder selection test to be undertaken to determine whether the foaming characteristics of the bitumen are applicable to the relevant site conditions (Austroads 2006).

The binder design is conducted to determine the half-life of the foam. Generally a foaming agent is added to the water to enhance the expansion ratio (Austroads 2006). An expansion ratio of 12–15 times is typical. The percentage of water content added to the bitumen can affect both the expansion ratio (as an increase in volume) and the half-life. Figure 2-8 shows that as the percentage of water content is increased, the expansion ratio increases and the half-life decreases (Kendall et al. 2001). Where the two intersect is known as the optimum foaming moisture content.

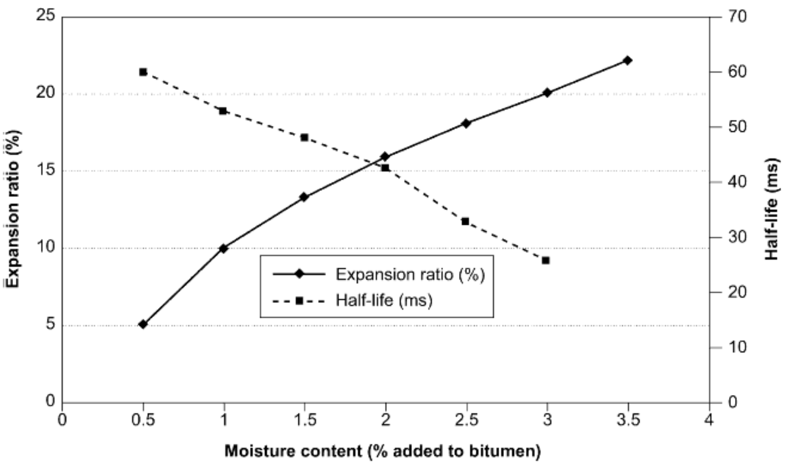

Figure 2-8: Effect of water content (%) on foaming characteristics

According to Austroads bitumen temperature also has an effect on the expansion ratio and the half-life foaming characteristics. Figure 2-9 illustrates that there is a decrease in half-life when the bitumen temperature is reduced to below 180°C (Austroads 2006).

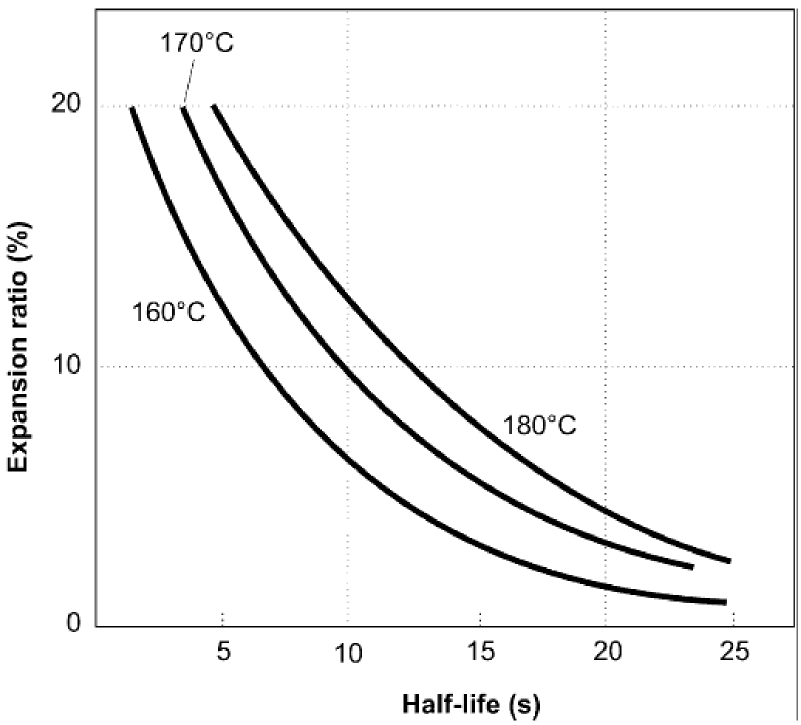

Figure 2-9: Foaming characteristics at different bitumen temperatures (after Austroads 2006)

2.7.2.2. Asphalt Academy Guidelines

Foamed bitumen only requires a low percentage of bitumen. The Asphalt Academy generally recommends a bitumen content of 1.7–2.5% and can utilise a softer grade of bitumen without compromising the stability of the mix (Asphalt Academy 2009). For practical reasons, harder bitumen is not used due to poor foaming characteristics leading to poor dispersion of the bitumen throughout the mix.

The Asphalt Academy has recognised important factors which can influence foam properties, including water content and quality and bitumen temperature. In order to ensure that the foam characteristics measured in the laboratory and in the field are comparable, it is recommended that at least three tests are conducted for each set of conditions. This will help to attain an acceptable level of reliability (Asphalt Academy 2009).

The Asphalt Academy has determined a value for the expansion ratio and the half-life of the bitumen based on the aggregate temperature. Table 2-6 outlines the minimum foam characteristics that are required based on aggregate temperatures of 10–25°C or higher than 25°C (Asphalt Academy 2009).

Table 2-6: Minimum foam characteristic limits (after Asphalt Academy 2009)

Aggregate Temperature	Expansion Ratio	Half-life (Seconds)
10°C–25°C	10	6
25°C	8	6

2.7.2.3. Californian Guidelines

The selection of a bitumen binder suitable for foamed bitumen applications should be based on the foaming characteristics (half-life and expansion ratio). It is recommended that the designer should check the foaming characteristics throughout the project and on each day of construction to monitor the conformance with the mix design (Jones, Fu and Harvey 2009). The Californian guidelines use an expansion ratio of 10 and a half-life of 12 seconds.

Previous research has confirmed that for a given water content, an increase in bitumen temperature results in higher expansion ratios and a longer half-life. By contrast, for a given bitumen temperature, an increase in water content results in higher expansion ratios but a shorter half-life (Jones, Fu and Harvey 2009). Based on these findings, the binder dispersion in the aggregate improves with increasing expansion ratio and half-life and a better dispersion leads to better material properties.

2.7.2.4. Queensland Department of Transport Main Roads

The Queensland Department of Transport Main Roads (QDTMR) has investigated the bitumen content from another angle, based on the percentage of fines passing the 0.075mm sieve. The respective bitumen content is shown in Table 2-7 (Ramanujam and Jones 2007).

Table 2-7: Bitumen content based on percentage of fines

Passing 4.75mm sieve (%)	Passing 0.075mm sieve (%)	Bitumen content (% of dry aggregate)
< 50	5.0–7.5	3.0
	7.5–15	3.5
	15–20	4.0
> 50	5.0–7.5	3.5
	7.5–15	4.0
	15–20	4.0

2.7.2.5. Synthesis

There is no evidence of a common, world-wide agreement on optimum bitumen content. Instead, it has been suggested that laboratory testing be undertaken for every individual project in order to determine the optimum bitumen content, as many factors may vary. It is therefore not recommended to quantify the optimum bitumen content, but rather to standardise the testing procedure.

Foaming characteristics are essential factors for the stabilisation qualities of foamed bitumen. Foamed bitumen that can extend the expansion rate without compromising half-life is highly valued. A 2.5% foaming water content has generally been applied in many situations and has been widely used in Western Australia. However, this recommended value of foaming water content could be changed if different bitumen sources have been adopted.

2.7.3. Secondary Binder Selection

The secondary binder which is also known as the active filler has several functions within the foamed bitumen stabilisation mix design. The secondary binder can increase the stiffness of the pavement for early strength gain and can allow the pavement to be driven on earlier, reducing traffic disruption. The secondary binder can also be used as a modifier to reduce the plasticity of a material and aid in the dispersion of the bitumen throughout the mix. The types of secondary binder used include cement, lime or fly ash.

2.7.3.1. Austroads Technical Report

Austroads recommend the use of 1–2% supplementary binder, generally either lime or cement (Austroads 2006). The 2% upper limit is based on practical field experience. The guide also notes that lime may not be required for lightly trafficked roads if the plasticity index (PI) of the material is very low.

2.7.3.2. Asphalt Academy Guidelines

The Asphalt Academy suggests that the PI of the material being used should not exceed 10 unless lime is added to reduce the plasticity. The guidelines also suggest that if cement is being used as the secondary binder, it should not exceed 1% by mass. The Asphalt Academy primarily uses either hydrated lime or cement as the secondary binder. When lime is used, the laboratory testing should take into account the time required for the lime to react with the plastic fines before the addition of bitumen.

2.7.3.3. Californian Guidelines

The Californian guidelines recommend use of at least 1% active filler of either Portland cement or hydrated lime to provide initial strength for early opening to traffic.

2.7.3.4. Queensland Department of Transport Main Roads

The secondary binder content is dependent on the grading and the plasticity of the material used. For in-situ construction works, the hydrated lime content is based on:

- 2% lime for a PI $\geq$ 6 up to a maximum PI of 10.

- 1.5% lime for PI < 6.

Other than the recommendation of the hydrated lime content, the QLD also states the benefit of use of lime:

- To flocculate and agglomerate the clay fines in the material.
- To stiffen the bitumen binder.
- To act as an anti-stripping agent to help disperse the foamed-bitumen throughout the material, and
- To improve the initial stiffness of the material and increases the early rut resistance of the stabilised material.

2.7.3.5. Synthesis

Cement and lime are two common secondary binders used world-wide. The amount of these binders is limited to 2% regardless of the plastic index, as increasing the amount will result in a stiffer mixture rather than a flexible structure. Due to the cost and construction application referred in QLD reports, lime is preferable for practical work.

2.7.4. Mixing Moisture Content

Sufficient moisture content can assist the dispersal of foamed bitumen and later the compaction. Therefore, it is important to spray water onto the raw aggregate to provide enough moisture content.

2.7.4.1. Austroads Technical Report

There was no specified quantity for mixing moisture content mentioned in the Austroads reports. However, it was suggested that insufficient water reduces the workability, bitumen dispersion and compaction, whereas too much water reduces the strength, increases curing time and affects aggregate coating (Austroads 2011).

2.7.4.2. Asphalt Academy Guidelines

The sample moisture contents vary through the mix preparation process. The mixing moisture content is 65–85% of optimum moisture content (OMC) as determined by the modified American Association of State Highway and Transportation Officials (AASHTO) compaction of the untreated material (Asphalt Academy 2009). The minimum mixing moisture content is the aggregate “fluff point”, the point at which the maximum bulk volume of the loose aggregate is obtained (Asphalt Academy 2009).

A vibratory hammer is used for laboratory mix characterisation. The OMC for the material is determined using this form of compaction. Due to compaction with heavy vibratory rollers, the appropriate moisture for field compaction may be 1.5% lower than the OMC by laboratory vibratory compaction.

2.7.4.3. Californian Guidelines

The Californian Guidelines refer to the moisture content as the mixing moisture content (MMC), which is defined as the moisture content in the material when the foamed bitumen is injected (Jones, Fu and Harvey 2009). There is currently no standard test method for determining the MMC for foamed bitumen mixes. The method suggested in the California guidelines is as follows (Jones, Fu, and Harvey 2009):

- Prepare four samples of the material.
- Place material and active filler in the mixer.
- Add sufficient water to meet the following mixing moisture contents (some moisture will already be present): 1) OMC; 2) OMC minus 1%; 3) OMC minus 2%; 4) OMC minus 3%.
- Inject the foamed bitumen.
- Compact the samples and measure the moisture content.
- Select the MMC which yields the maximum dry density of the foamed bitumen mix.

From experience, the MMC yielding the maximum dry density is typically 75–90% of the OMC of the material (Jones, Fu and Harvey 2009).

2.7.4.4. Queensland Department of Transport Main Roads

The QDTMR method requires that the moisture content is determined from the PI of the material. The following criteria apply (Austroads 2011):

- PI < 6% – prepare test samples at 70% OMC of the untreated material using standard compaction.
- PI 6-10% – prepare test samples at 70% OMC of the untreated material using standard compaction or higher.
- Where field moisture content is greater than 70% OMC by standard compaction, prepare test samples at field moisture content.

2.7.4.5. Synthesis

At present, there is no specific requirement regarding the mixing moisture content in foamed bitumen mix design. However, as this factor can affect bitumen dispersion and further compaction, more research into this parameter is required. It is suggested that 80% of OMC moisture content has to be achieved during field construction but no more than 100% of OMC.

2.7.5. Laboratory Compaction

Compaction has a great influence on the performance and quality of foamed bitumen during construction or the production of samples. If compaction is insufficient, premature failure is likely to occur and specimens will not withstand traffic loadings. Over-compaction can result in breakage of coarse aggregates.

2.7.5.1. Austroads Technical Report

Austroads (2011) recommends that compaction of test cylinders be undertaken using gyratory compaction (80 cycles) or Marshall (50 blows) compaction. Austroads have stated in their review, that in the interim, and pending further research, they have recommended that no change be made to the previous version.

2.7.5.2. Asphalt Academy Guidelines

Asphalt Academy recommends vibratory hammer compaction, as they believe it emulates field density and particle orientation achieved in the field. Where a vibratory hammer is not available, Marshall Compaction may be used, but this is not the preferred method. The OMC is relative to the compaction method used, and is based on the untreated material.

The vibratory method was used on specimens 150mm in diameter and 300mm high. The recommended moisture content is the OMC of the untreated material determined using modified AASHTO compaction. Compaction is undertaken at 25 ± 2°C.

2.7.5.3. Californian Guidelines

Samples of 100mm in diameter and 63.5mm in height are produced using Marshall compaction in which 75 compaction blows are applied on each face of each sample. The compaction is to occur within 8 hours of the mix being prepared.

2.7.5.4. Queensland Department of Transport Main Roads

Cylindrical test specimens are compacted using the Marshall hammer (50 blows/layer), as follows:

- Riffle stabilised material into four samples of 2.5kg.
- Compact in 150mm diameter Marshall moulds (not heated) using 50 blows/face.

2.7.5.5. Synthesis

So far, the argument over whether the Marshall compactor or the gyrator compactor is the preferred method for foamed bitumen mixture has not been resolved. Although the Marshall compactor is widely utilised in most laboratory tests, more attention should be given to the gyrator compactor as its kneading and shear behaviour is more likely to replicate field compaction practice. No comments will be made upon the number of Marshall blows or gyrations due to the fact that the existing standards are based on asphalt mix design.

2.7.6. Laboratory Curing Regime

In the field, curing may take many months and it is an ongoing process. The laboratory process needs to simulate the field conditions for curing, but as it is impractical to cure for months, an accelerated method is required to emulate field conditions.

2.7.6.1. Austroads Technical Report

Austroads recommends the following approach to sample curing (Austroads 2011):

- Immediately after compaction, the cylinders are tested for modulus without curing. The uncured modulus needs to exceed 700 MPa to ensure that the pavement can be opened to traffic after trimming.
- The cylinders are then oven cured at 60°C for three days and then tested dry for indirect tensile modulus (Mdry).
- The cylinders are then soaked in water prior to testing for their soaked modulus (Mwet). Two methods may be used for soaking the cylinders: submerged under water for 24 hours or in a vacuum chamber for 10 minutes.
- The wet and dry modulus results are then plotted versus bitumen content to define the optimum modulus.
- Note that samples should be prepared with moisture contents such that Mwet/Mdry is 0.5 or more, because bituminous binders will not cure at excessive moisture contents.

Austroads also recommends that on larger projects additional testing may be undertaken including unconfined compressive strength, flexural fatigue and creep. The Austroads review proposes as an interim measure to amend previous version to make it consistent with the current Queensland method. Further research is recommended to assess whether the samples should be cured to their long-term equilibrium moisture contents, as well as the extremes of dry and soaked.

2.7.6.2. Asphalt Academy Guidelines

The Asphalt Academy recommends the following process be used for the curing of samples (Asphalt Academy 2009):

1) Curing at 60°C in an oven results in lower moisture content than field equilibrium.
2) Characterising materials with secondary binders where curing for seven days or 28 days resulting in time delays.
3) Sealing specimens in plastic bags and curing for 72 hours in an oven at 40°C retains excessive moisture and gives conservative results.
4) Curing for 24 hours at 25°C (unsealed) followed by 48 hours sealed in a bag and cured in an oven at 40°C more closely reflects equilibrium moisture

content but does not provide evidence that the laboratory stiffness truly reflects field stiffness.

5) Field testing in South Africa to re-evaluate prediction models for equilibrium moisture content has provided more robust predictions based on optimum moisture content, bitumen content and climate. This showed that the 24 hours at 25°C unsealed and then 48 hours sealed at 40°C provided the most accurate prediction.

Whilst research is currently underway, the current interim recommended method is to cure for 24 hours at 25°C unsealed and then seal and cure for a further 48 hours at a temperature of 40°C (Asphalt Academy 2009).

2.7.6.3. Californian Guidelines

The following steps are followed for the curing of samples (Jones, Fu and Harvey 2009):

- The samples are removed from their molds immediately after compaction and cured in a forced draft oven at 40°C for 72 hours.
- After curing, the samples are soaked in water for 24 hours at a water temperature of between 20°C to 25°C. The water depth is to be 100mm above the sample surface. Samples must not be stacked on top of one another.
- After soaking, the samples are removed from the water and drained for 60 minutes at room temperature (25°C ± 2°C) and covered with a damp cloth to prevent additional evaporation.

2.7.6.4. Queensland Department of Transport Main Roads

The QDTMR recommends the following tests (Austroads 2011):

- Initial modulus after three hours curing at 25 ± 5°C is determined to provide an indication of susceptibility to permanent deformation early in pavement life.
- Cured modulus where the sample is oven cured at 40°C for three days to provide an indication of medium term stiffness 3–6 months after construction.

- Soaked modulus where the sample is submerged under water for 10 minutes under a 95 kPa vacuum to provide an indication of the moisture sensitivity of the material.

QDTMR results showed that laboratory cured samples compacted to 50 Marshall blows achieved similar resilient modulus values to the upper half of field cores after 12–14 months field curing (Austroads 2011).

2.7.6.5. Synthesis

The curing regime is considered to be the most controversial factor among the mix design considerations. Its importance and uncertainty have made such research much more urgent and difficult. Based on the reviews, it is very hard to determine which curing method is the most representative, due to ambient conditions varying widely from area to area. With a consideration of Western Australian local condition, the California curing method is more favourable.

2.8. Literature Summary

This chapter provides a detailed insight into the current applications of foamed bitumen stabilisation nationally and internationally. Although the importance and popularity of foamed bitumen had been well accepted, this rejuvenated technology still requires more research and proofing in many areas. Western Australian road department agencies are keen to see the full potential utilisation of foamed bitumen stabilisation in Western Australian pavement. This has resulted in an imperative understandings of the characteristics of the foamed bitumen stabilised mixture and their effects.

This chapter reviewed a considerable amount of previous research in foamed bitumen mix design considerations where aggregate grading, foaming characteristics, secondary binder selection, mixing moisture content, laboratory compaction as well as laboratory curing regime are included. It has been accepted that all the above factors are essential in the mix design and must all meet specific standards in order for foamed bitumen to be the most effective stabilisation method applicable. However, the following research will focus on the varied Western Australian aggregate properties due to the limitation of the nature of this thesis.

3. METHODOLOGY AND EXPERIMENTAL PROGRAM

3.1. Overview

This chapter presents the research methodology and the laboratory experimental program carried out throughout this project. The main purpose of the research is to evaluate the mechanical performance of foamed bitumen mixtures under Western Australian conditions, based on laboratory test results obtained from variations on aggregate selections. Figure 3-1 displays a working diagram of the laboratory experiments, broken down into sub-sections.

The project started with a preliminary study of aggregate composition to determine the percentage at which the laboratory blend was able to replicate a field sample. A subsequent step was the gradation of this representative laboratory sample. Five designated gradation curves are examined from coarse section to fine section. Moreover, this project aimed to investigate the fine content in the aggregate gradation by means of adjusting the fine content from nil to excessive. Once the experimental programs were completed, the laboratory test results were analysed and discussed in order to extrapolate recommendations and conclusions.

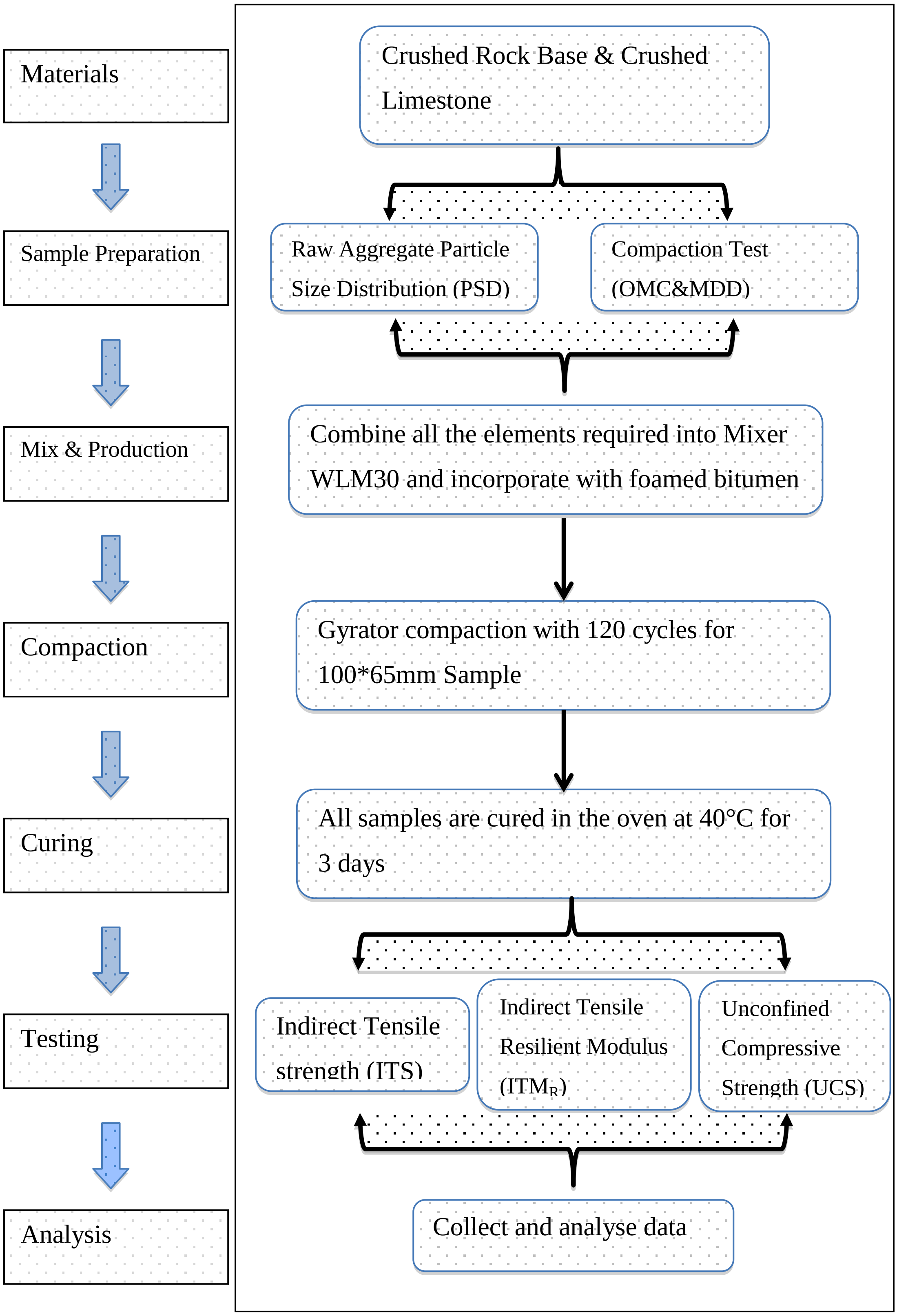

Figure 3-1: Diagram showing laboratory experimental approach

3.2. Materials

3.2.1. Aggregate

In order to replicate real field conditions, it is highly preferable to use in-situ recycled base course as the parent material throughout the laboratory experiments. However, when in-situ recycled material is not available, crushed rock base (CRB) and crushed limestone (CLS) that comply with Main Roads Western Australia (MRWA) Specification 501 Pavements, are used as a representative of the real in-situ materials (Main Roads Western Australia 2010). After randomly sampling the materials from a local Gosnells quarry, they were directly transported to the laboratory at the Department of Civil Engineering, Curtin University. In accordance with MRWA Test Method WA 115.1, the particle size distributions (PSD) of CRB and CLS were obtained, as listed in Table 3-1, along with the specifications. Figure 3-2 shows the PSD of CRB and CLS and compares CRB with MRWA base course specifications.

Table 3-1: PSD of CRB and CLS compared with Specification 501

Sieve Analysis	Passing by Mass (%)		
	CRB	Specification 501-Base course	CLS
19 mm	100.0	95–100	100.0
13.2 mm	85.8	70–90	98.4
9.5 mm	71.4	60–80	96.4
4.75 mm	55.5	40–60	89.2
2.36 mm	45.2	30–45	83.7
1.18 mm	32.3	20–35	74.6
0.6 mm	22.7	13–27	65.0
0.425 mm	19.4	11–23	56.7
0.3 mm	16.5	8–20	45.1
0.15 mm	12.1	5–14	21.4
0.075 mm	9.2	5–11	11.1

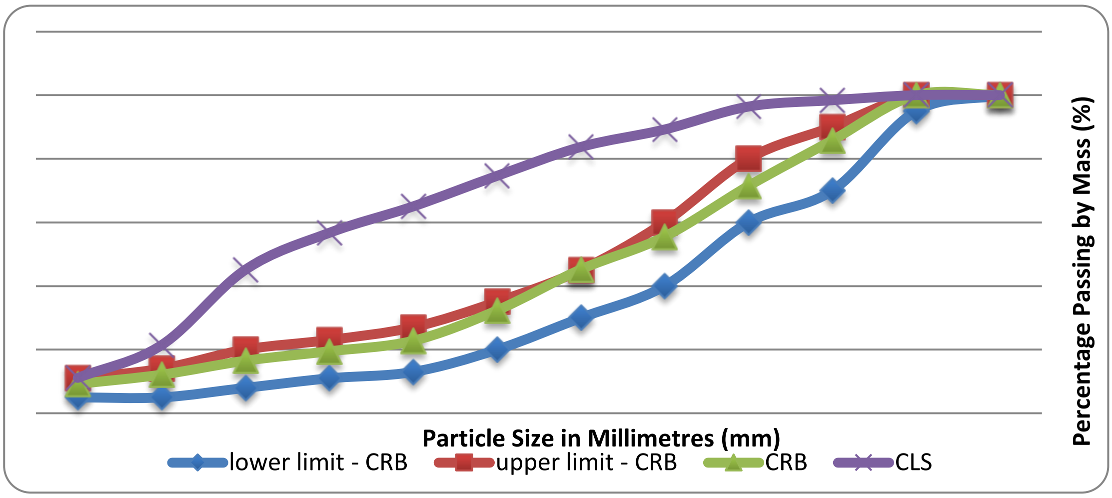

Figure 3-2: Particle size distributions of CRB and CLS compared with MRWA Specification 501

3.2.2. Foamed Bitumen Sample Preparation

Foamed bitumen samples were produced in a Wirtgen WLB 10S, with guidance taken from the procedures outlined in the operating manual for this unit (Wirtgen GmbH 2008). This laboratory plant is capable of producing foamed bitumen under varying parameters, including bitumen temperature, water content, and air pressure during the foaming process in the laboratory. The Wirtgen WLB 10S is also capable of delivering the required mass of foamed bitumen to the nearest gram, into an operating twin-shaft pugmill (WLM 30). Figure 3-3 illustrates the state-of-the-art machine used in this study.

Figure 3-3: Wirtgen WLB 10S and Wirtgen WLM 30

The grade of bitumen used in this project was a standard Class 170 binder sourced from a local BP distributor as shown in Figure 3-4. Bitumen C170 is widely used in light asphalt application and spray seals to provide durability and fatigue resistance. BP bitumen C170 at 135°C has a viscosity of 0.40Pa.s and a density of 1.04kg/m3 at 15°C (BP Australia 2012). In this project, 2.5% of cold foaming water was chosen as an optimum foaming water content, together with a constant air pressure of 5 bars and water pressure of 4 bars. As a result, a foamed bitumen product with an expansion ratio of 15–20 and a half-time of 20 seconds could be produced. A summary of foaming parameters is presented in Table 3-2. According to Ramanujum, Jones and Janosevic (2009), this product is considered a good foam quality with no foaming agent.

Figure 3-4: Bitumen C170 used in this study

Table 3-2: Foaming parameters used in the mixing design process (Huan et al. 2010).

Parameters	Values
Bitumen Temperature (℃)	170–175
Foaming Agent (%)	0
Added Water Content (%)	2.5
Bitumen Flow Quantity (g/s)	100
Water Flow Quantity (g/s)	9.0
Air Pressure (bars)	5
Water Pressure (bars)	4
Expansion Rate	15–20
Half-time (s)	20

3.2.3. Active Filler

Hydrated lime was selected as the active filler throughout the experiments by virtue of the fact that it can provide some additional desirable performance properties to the mixture by aiding in the distribution of foamed bitumen through the mix (Asphalt Academy 2009). Table 3-3 lists the main composition and general properties of hydrated lime.

Table 3-3: General properties of hydrated lime in WA (Swan Cement 2005).

Properties	Description	Range
Appearance	White amorphous powder	
Specific gravity (kg/m^3)	2300	
pH	12	
Chemical composition (%):		
Calcium hydroxide ($Ca[OH]^2$)		80–90
Magnesium hydroxide ($Mg[OH]^2$)		0–6
Silicon dioxide (SiO^2)		2–6
Aluminium oxide (Al^2O^3)		0.2–0.6
Iron Oxide (Fe^2O^3)		0.1–0.3

In this project, it was not desirable to measure the resulting mechanical strength caused by the additional hydrated lime. Therefore, its content was kept to a minimum of 1% by mass of dry aggregate in order to prevent it becoming the dominant additive through the significant effects of hydrated lime binding, while still allowing it to act as an aiding agent.

3.3. Sample Preparation

Prior to blending with foamed bitumen, some basic properties of raw aggregate, also defined as non-mechanical characterisations, have to be investigated in the sample preparation phase. These include particle size distribution (PSD) and maximum dry density as determined by the modified Proctor compaction test.

3.3.1. Particle Size Distribution

Particle size distribution (PSD) testing was used to determine the gradation characteristics of the blended aggregates used in this study. In order to construct the most useable research outcomes, the aggregate's suitability for foamed bitumen stabilisation was assessed by determining the passing percentage by mass of aggregates in each respective particle size. PSD was tested by machine sieving, illustrate in Figure 3-5 and 3-6. The test was carried out in accordance with Australian Standards 1141.11.1-2009: Particle size distribution –Sieving method (AS1141.11.1). A summary of this procedure is presented in Appendix A.

Figure 3-5: Coarse sieve apparatus.

Figure 3-6: Fine sieve apparatus

3.3.2. Compaction Test

There are two general compaction methods used for engineering purposes, which are the standard Proctor method and the modified Proctor method. In this study, the modified method of compaction was used to determine the relationship between moisture content and dry density as the nominal size of parent aggregate is over 19mm. The compaction test was conducted in accordance with WA133.1 (Main Roads WA 2007). The equipment used is presented in Figure 3-7 and the compaction test apparatus dimensions are outlined in Table 3.4. A summary of the procedures is shown in Appendix B. When obtain all the raw data, graph the dry density and moisture content in a parabola curve and spot the peak point as the maximum dry density and corresponding X-axial point is optimum moisture content, illustrated in Figure 3-8).

Figure 3-7: Compacting the mixture with modified compaction method

Table 3-4: Modified compaction test apparatus dimensions

Parameter	Value
Mould	
Diameter (mm)	105
Height (mm)	115.4
Calculated Volume (cm^3)	999
Weight (g)	4548
Rammer	
Diameter (mm)	50
Drop (mm)	450
Mass (kg)	4.9
Energy Delivered per Blow (J)	22
Number of Layers	5
Number of Blows per Layer	25

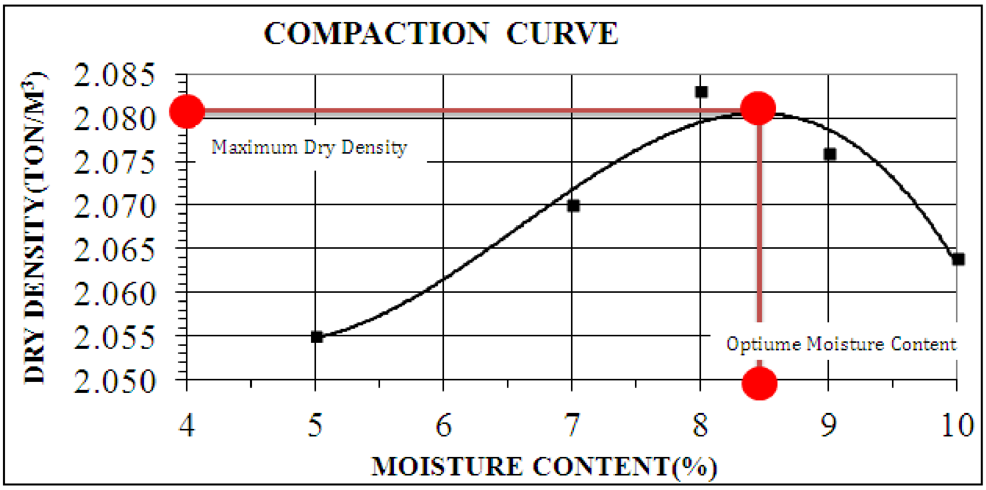

Figure 3-8: A typical compaction curve indicating MDD and OMC

3.3.3. Mixing Process

The apparatus used to mix and combine the raw materials outlined in Section 3.2.2 was the WLM 30 mixing chamber which functioned in conjunction with the Wirtgen WLB 10S.

Both the raw materials and the apparatus required preparatory work to be carried out prior to mixing. It was necessary for the CLS and CRB to both be dried until moisture contents were below 0.5% which was typically achieved by heating the materials overnight in an oven set at 105°C, causing all moisture to be evaporated.

The oven-dried aggregates were then placed into the Wirtgen WLM30 while cooling off to room temperature, with a nominated percentage of hydrated lime for pre-mixing until the active filler was homogeneously blended with the aggregates. This step, defined as a “dry mix” only ideally operated in laboratory conditions, was carried out to eliminate the concern that hydrated lime contacts with water to form lumps and thereupon loses its designated purpose (Huan, Jitsangiam and Nikraz 2011). Subsequently, a certain amount of water was added to achieve the target moisture content raised to 100% of OMC of raw aggregates in this study.

It was also advantageous to pre-heat the Class 170 bitumen to a temperature range of 150°C to 180°C. This allowed the viscosity of the bitumen to be reduced enough to enable an easy pour into the Wirtgen WLB 10S.

The Wirtgen WLB 10S required certain preparations prior to pouring the foamed bitumen into the mixing chamber. In order to achieve a reasonable foamed bitumen expansion rate, the bitumen temperature should fall within a range of 170°C to 180°C in the light of the correlations between bitumen temperature and expansion rate and half-life described by the Wirtgen Group, illustrated in Figure 3-9 (Wirtgen GmbH 2008).

This in turn requires all operating components which come into contact with the bitumen to be as close as possible to 170°C, also to ensure that the bitumen does not cool upon contact with these components. Pre-heating of the Wirtgen WLB 10S components typically took around 30 minutes.

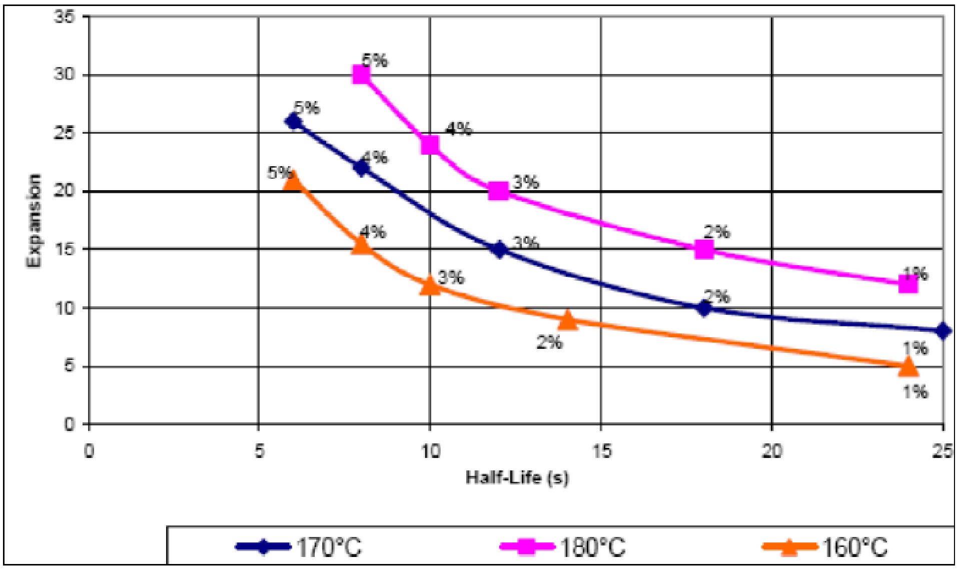

Figure 3-9: Example of expansion and half-time at three different temperatures and water contents from 1% to 5% (Wirtgen GmbH 2008)

The mixes were then prepared by spraying a determined mass of foamed bitumen as a percentage of dry aggregate mass into the aggregates, producing approximately 15kg batches of foamed bitumen mixtures. Figure 3-10 shows a typical foamed bitumen mixture after cold mix. A technique that was used to roughly investigate the binding quality of the treated material after mixing was carried out, as demonstrated in Figure 3-11. When a small amount of loose mixed material was firmly squeezed in the hand, a few black dots of bitumen sticking to the palm were seen as an indicator of good quality. Mixtures with no black dots of bitumen or visible nodules of bitumen were considered to be deficient.

Figure 3-10: A typical foamed bitumen mixture   Figure 3-11: Quality indicating test

3.4. Compaction

Upon successful completion of the mixing process, the combined foamed bitumen mixture was then to be compacted into two different size moulds dependent on the testing requirements.

3.4.1. Indirect Tensile Sample

The Servopac gyratory compactor, as pictured in Figure 3-12a, was used to compact all the samples in this study which would later be tested for indirect tensile strength and indirect tensile resilient modulus testing. The sample preparation and compaction of foamed bitumen test specimens using a gyratory compactor were carried out in accordance with AS 2891.2.2-1995 (Standards Australia 1995a).

The gyratory compactor was set to apply a vertical loading stress of 240kPa to a test specimen in the mould for 120 cycles, or to a nominal height of 55mm, at a total fixed angle of 2 degrees measured at the centre of the height of the mould. Specimens were not expected to reach this height of 55mm – it was set as a lower limit to ensure that all samples underwent an equal compactive effort. A nominal sample height of 60mm and diameter of 100mm was expected. After compaction, height and diameter of all specimens were measured with a Vernier calliper in three directions and the average values recorded. An example of a compacted specimen is shown in Figure 3-12b. Later, the bulk density was calculated from the weight and volume obtained from the samples to ensure all the samples were similar.

(a) (b)

Figure 3-12: a) Gyratory compactor, b) Compacted specimen

3.4.2. Unconfined Compressive Strength Sample

Compaction was undertaken according to AS1289.5.2.1-2003 using a steel rammer and mould of 200mm height and 100mm diameter (Standards Australia 2003). Details of the steel rammer and mould used are provided in Table 3-5 below. The modified compaction method applies eight layers of compaction in the sample, with each layer required to be approximately 25mm in depth. Each layer is required to be compacted with 25 equally distributed blows from a 4.9kg rammer (shown in Figure 3-13).

As noted before, there are two compaction methods – the standard compaction method and modified compaction method. In this study, modified compaction was employed to deliver a relatively large amount of compactive effort to the sample and to better simulate field conditions where high levels of compaction are used through machines such as pad foot rollers, smooth drum rollers and multi tyre rollers. A compactive effort of 2753 kJ/m^3 is applied through this method compared to the Standard Method of Compaction that delivers 596 kJ/m^3 of compactive effort.

Table 3-5: Dimensions for compaction rammer and moulds

Apparatus	Dimension
RAMMER	
Diameter (round foot), mm	50
Radius (sector foot), mm	74
Arc of segment (sector foot),degrees	41
Area of rammer, mm^2	1964
Drop, mm	450
Mass, kg	4.9
Energy delivered per blow, J	21.62
MOULD	
Internal diameter, mm	100
Height, mm	200
Nominal volume, cm^3	1571

Figure 3-13: Modified compaction equipment – rammer and mould

3.5. Curing

Curing is the process whereby a foamed bitumen mix gradually gains strength over time accompanied by a reduction in the moisture content (Bowering 1970). Two separate methods of curing were undertaken in this study, namely soaked and unsoaked curing.

Unsoaked curing was used for the majority of samples prepared in this study, and entailed placing the sample in an oven at 40°C for 72 hours. The purpose of unsoaked curing is to obtain similar moisture contents to that obtained in optimum field conditions. Realistically this level of reduced moisture content would not be achieved in the field for a short period under optimum conditions, so an accelerated drying method was adopted. The samples remained unsealed, simulating the environment of typical stabilised materials. The choice of 40°C is based on findings which suggest that higher temperatures result in aging of the bitumen within samples, which will impact the accuracy of results obtained (Muthen 1998). This method was utilised for all specimens, including UCS test specimens and the indirect tensile test specimens.

Soaked curing was also applied to half of the tensile test samples, and required immersion of the sample in room-temperature water for 24 hours preceding the unsoaked curing. The purpose of soaked curing is to assess the properties of the stabilised mix under worst-case conditions of moisture content.

Samples were weighed upon completion of specimen compaction, and again after testing had been undertaken. Following testing, specimens were placed in a drying oven for 24 hours, and then weighed once more. The purpose of this was to determine moisture content at the various stages, with the goal of assessing the efficiency of the curing process.

Any variations occurring during sample preparation and testing in curing methods from those outlined above were noted. Relevant results obtained were analysed separately to ensure any resulting impact was documented and accounted for.

3.6. Mechanical Testing

The mechanical characteristics of base course material are the essential limiting factors that affect the quality and long-term performance of the pavement. Therefore, a better understanding of pavement material is necessary to maintain and improve its long-term performance during service life. Three tests were performed at room temperature, each with the purpose of measuring a different mechanical property of the specimens. Specifically, indirect tensile strength (ITS) measures the tensile strength and flexibility, while indirect tensile resilient modulus (ITM_R) evaluates the maximum tensile stiffness, and unconfined compressive strength (UCS) measures the maximum compressive strength without confining pressure. These testing procedures were performed according to the guidelines set out in the Australian Standards.

3.6.1. Indirect Tensile Strength (ITS)

Indirect tensile strength (ITS) was determined using the Marshall Stability Machine (CL40580) as pictured in Figure 3-14, in accordance with the Australian Standard AS 1012.10-2000 (Standards Australia 2000). The strain rate was fixed at 50.8mm/min. The purpose of ITS testing is to study the foamed bitumen mix performance under load and its ability to resist tensile loading, as this is commonly the critical failure loading of base course or sub-base course materials. The simple operation of this testing method has made it well-accepted in literature and laboratory based studies. The interpretation of this test is very straightforward, making it readily comparable and giving a good indication of the performance of foamed bitumen mix when working along with ITM_R test results.

Figure 3-14: Marshall stability machine

The load is applied on the samples diametrically to split the sample from the centre. Meanwhile the maximum force applied indicated by the testing machine is recorded. The ITS values of the specimens can be obtained using the following equation that relates the maximum applied force with indirect tensile strength:

$$ITS = T = \frac{2000P}{\pi LD} \qquad \text{Equation 3-4}$$

Where:

T = indirect tensile strength (MPa)

P = maximum applied force indicated by the testing machine (kN)

L = length (mm)

D = diameter (mm)

3.6.2. Indirect Tensile Resilient Modulus (ITM_R)

The indirect tensile resilient modulus (ITM_R) test was performed under the guidance and procedures outlined in Australian Standard AS2891.13.1-1995 (Standards Australia 1995b) to determine the resilient modulus of foamed bitumen mixture. To measure the resilient modulus, the IPC UTM 25 machine was used as shown in Figure 3-15. During the testing process, the cabinet temperature is controlled as 25°C.

Resilient modulus is the ratio of applied deviator stress to the recoverable strain. The purpose of the ITM_R test is to obtain an estimated modulus value for stabilised materials. When a cylindrical specimen is loaded diametrically, a uniform distribution of tensile stress will occur in perpendicular with the applied load. This simulates the tensile stress experienced by the underside of pavement layers when the pavement is subjected to repeated axial loads.

Figure 3-15: IPC UTM 25

Figure 3-16: Loading during ITM_R test

The resilient modulus test measures the recoverable axial deformation response of the sample to different cycles of loading to generate the resilient modulus parameter for that sample. The specimen was diametrically loaded, as shown in Figure 3-16, and a pulsated load and horizontal tensile stress was induced which led to horizontal deformation of the sample. This horizontal resilient strain was recorded with the external LVDTs. The testing began with inputting sample information such as sample diameter, height, an initial estimation of loading pulse width, estimated modulus and target strain into the program. The program later initiated a pre-conditioning phase in which the sample was subjected to five pulses to calculate the required seating force, followed by a test phase which consisted of a further five pulses during which the program calculated the resilient modulus by referring to the peak load and resulting strain obtained, as shown in Figure 3-17. With the applied load (deviator stress) and recoverable strain data obtained, resilient modulus could be calculated with the following equation:

$$M_R = \frac{\sigma d}{\varepsilon r} \qquad \text{Equation 3-5}$$

Where:

M_R = resilient modulus (MPa)

σd = deviator stress (MPa)

εr = recoverable axial strain

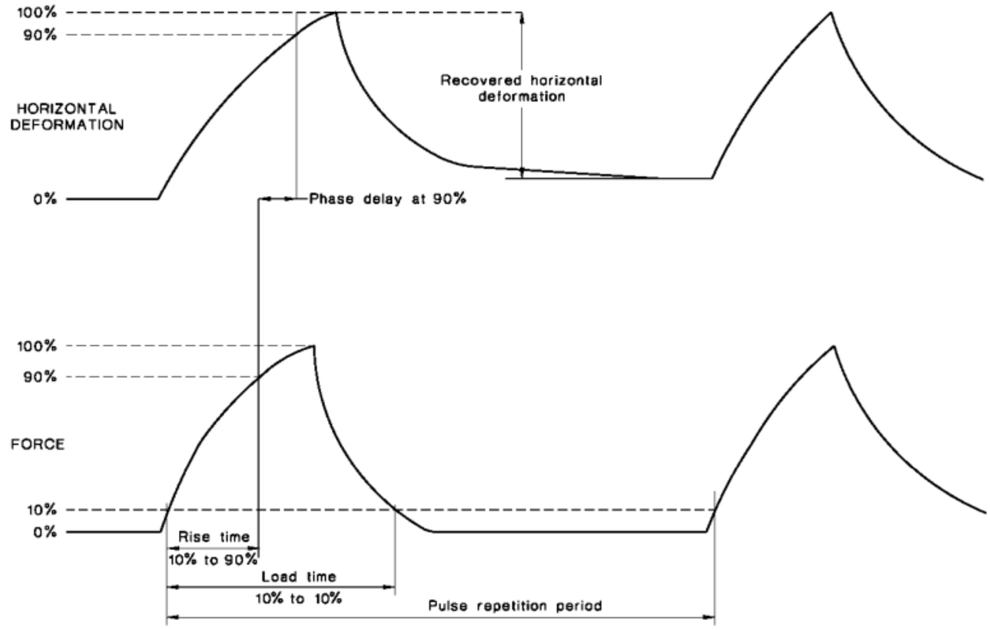

Figure 3-17: Graphical identification of resilient modulus calculation nomenclature

In order to generate comparable results, the loading pulse width was adjusted to provide a rise time of 40±5ms in accordance with the requirements of AS 2891.13.1-1995 (Standards Australia 1995b). Figure 3-18 shows the typical output of five pulse loading.

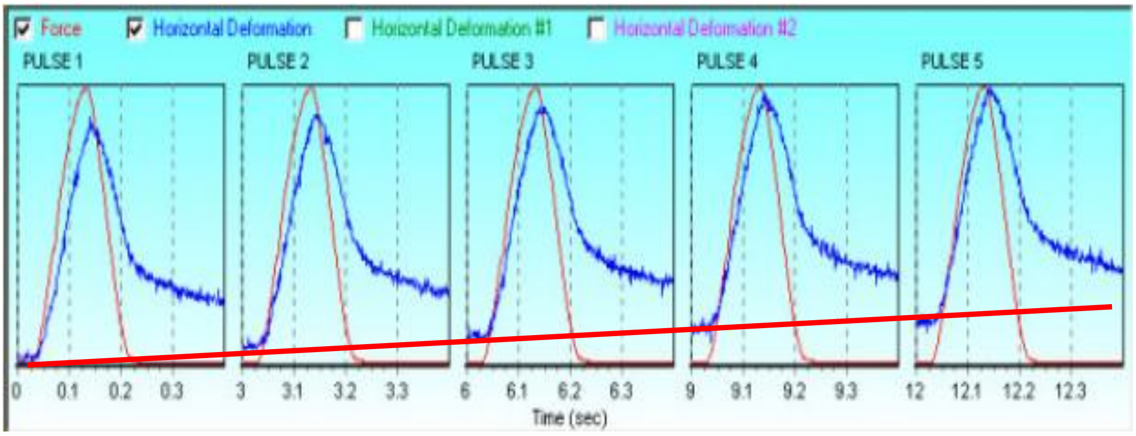

Figure 3-18: Typical five-pulse output

3.6.3. Unconfined Compressive Strength (UCS)

Unconfined compressive strength (UCS) testing provides information on the shearing resistance of a sample, and is useful to this research in providing an indication of the material properties when encountering similar loading patterns. The testing was undertaken in accordance with AS 5101.4-2008 (Standards Australia 2008). A hydraulic testing machine, pictured in Figure 3-19, was involved in the testing, in conjunction with an associated software package named “CATS” which controlled the loading actuator at a constant rate of 1mm/min whilst measuring and recording the load applied. This load was converted into a stress measurement in kilopascals, by relating the diameter of the sample with the stress, then plotting it against time.

A rubber sleeve was placed around the specimen during testing, as can be seen in Figure 3-19. This sleeve was of equal diameter to the specimen, and as such provided no confining pressure to the specimen; the primary role of this sleeve was to reduce loss of material upon specimen failure.

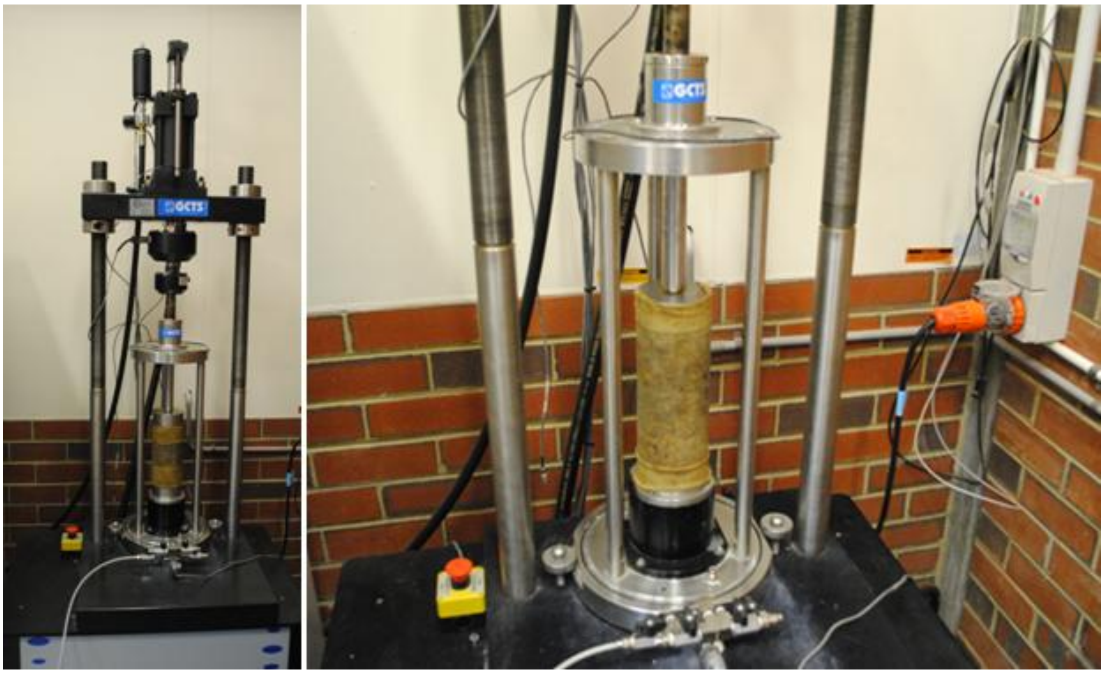

Figure 3-19: UCS testing apparatus

4. ENGINEERING PROPERTIES OF AGGREGATE COMPOSITION ON FOAMED BITUMEN MIXTURE

4.1. Overview

This chapter preliminarily examines the engineering properties of aggregate composition on foamed bitumen mixture by means of altering the percentage of crushed rock base and crushed limestone to fabricate a laboratory blend that adequately replicates field conditions. Different foamed bitumen contents were applied to each aggregate mixture in order to investigate the effect on the various mechanical tests. These included gradation, maximum dry density, ITS and UCS.

The majority of this work was completed in the first phase of the project and published in a journal paper in 2010, named: "A preliminary study on foamed bitumen stabilisation for Western Australian pavements" (Huan et al. 2010). This chapter is based on the results of this journal paper, which is also referred to as the main resource.

4.2. Background

As commonly used in a standard Western Australian pavement structure, asphalt is usually adopted in surface course with a depth of 30–60mm. Crushed rock base (CRB) is normally found in base course with a depth of 50–150mm on top of a 200–300mm sub-base course often consisting of crushed limestone (CLS), under which in-situ sand is commonly found as the subgrade material, although clay is common in river flats in Perth. As an in-situ rehabilitation method, foamed bitumen stabilisation requires a recycling depth to provide the required cover over the subgrade, but in some cases, additional material has been spread over the surface to increase the thickness or change the grading of existing pavement materials. Figure 4-1 illustrates the layout of a typical Western Australian pavement along with the different recycling depths.

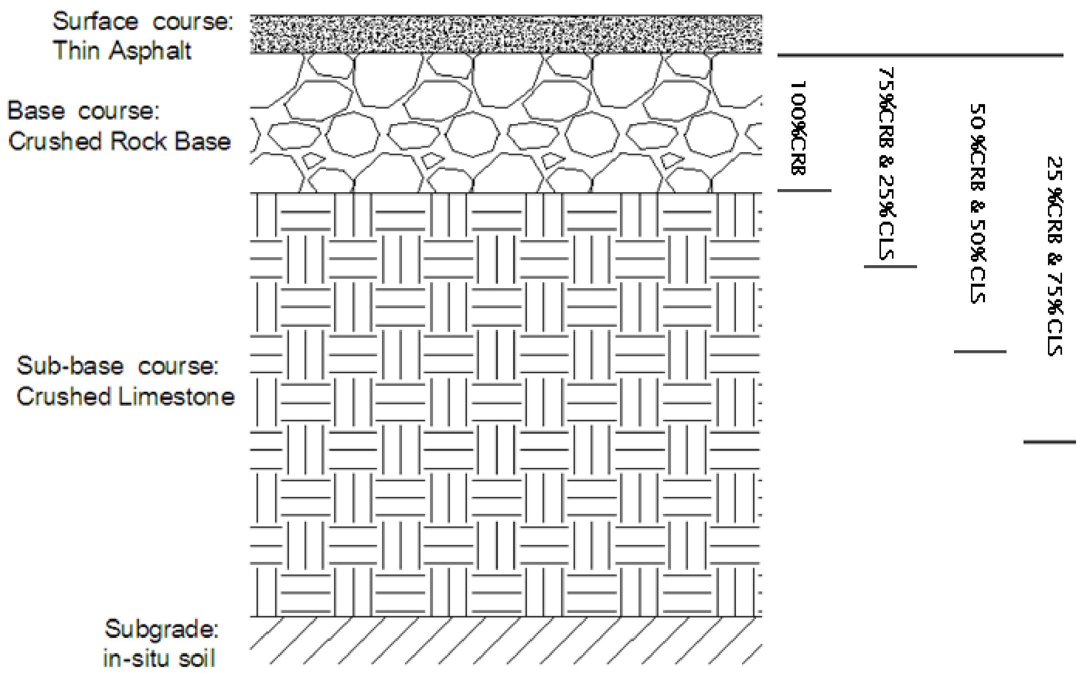

Figure 4-1: Layout of a typical Western Australian pavement

Experience from the construction site indicates that a recycling depth of 260–320mm is currently adopted in Western Australian conditions, which means that a blend of crushed rock base and limestone is evident. However, the optimum percentage is still unknown and needs to be established.

4.3. Methodology

Four different representative proportions of aggregate mixtures were nominated on the basis of different reclaimed depths in the real trial project, i.e. 100%CRB, 75%CRB and 25%CLS, 50%CRB and 50%CLS, and 25%CRB and 75%CLS. Non-mechanical tests such as the PSD, optimum moisture content and maximum dry density tests were undertaken on each blend. For each blend, during the foaming process, 3%, 4% and 5% foamed bitumen by mass of dry aggregate were injected into the four different batches at the optimum moisture content which would produce 12 samples respectively. On the completion of laboratory compaction and curing, some mechanical tests were undertaken including gradation, density, indirect tensile strength and unconfined compressive strength.

4.4. Analysis of Results

4.4.1. Gradation

Gradation analysis explored three different angles including fine content, conformation with recommended gradation limits as well as the comparison with field reclaimed materials. Table 4-1 lists the results of PSD tests which were undertaken in conformance with MRWA Test Method 115.1.

Table 4-1: Particle size distribution of aggregate mixtures (after Huan et al. 2010)

Sieve Analysis	100%CRB	75%CRB & 25%CLS	50%CRB & 50%CLS	25%CRB & 75%CLS
26.5 mm	100.0	100.0	100.0	100.0
19 mm	100.0	99.3	99.7	100.0
13.2 mm	85.8	91.7	94.2	94.1
9.5 mm	71.4	81.1	85.3	88.9
4.75 mm	55.5	65.4	73.7	80.5
2.36 mm	45.2	54.5	65.3	73.8
1.18 mm	32.3	43.3	54.8	65.1
0.6 mm	22.7	34.1	45.0	54.8
0.425 mm	19.4	29.7	39.4	48.2
0.3 mm	16.5	24.5	32.4	39.2
0.15 mm	12.1	14.3	17.0	19.5
0.075 mm	9.2	9.0	9.7	10.5

Examination of the fine contents of these four mixtures revealed no apparent differences as they all fell within a range of 9.0–10.5%. With an increasing CLS content, the fine content also increased. All of these values were acceptable in terms of the Austroads-specified fine content range in which a requirement of 5–15% is achievable (Austroads 2011).

Figure 4-2 presents the PSD of different mixtures and the recommended grading zones for foamed bitumen introduced by the Asphalt Academy (2009). Three of the four grading curves fall in the less suitable zone with the exception of 100% CRB, which falls in the ideal zone. This indicates that special treatment would be required to bring the grading to the ideal zone.

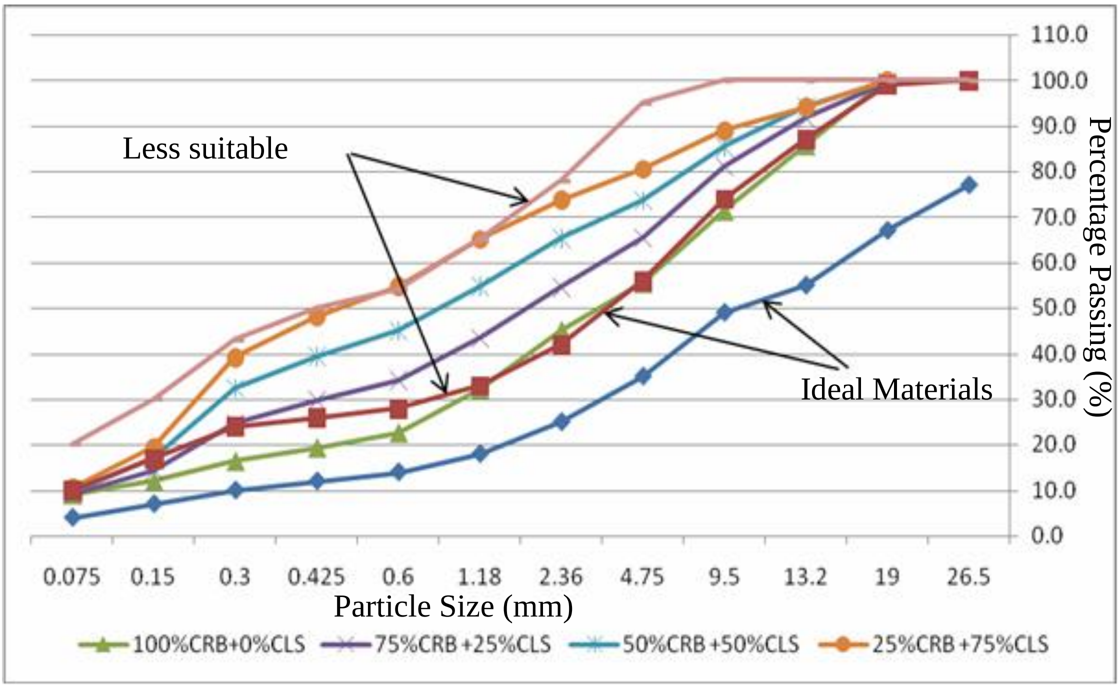

Figure 4-2: Particle size distribution of different mixtures compared with the grading zones for foamed bitumen introduced by the Asphalt Academy (after Huan et al. 2010)

In order to observe the similarity of laboratory blend mixture and real field recycled materials, two local roads, Sheffield Road and Felspar Road located in Kewdale WA, were selected as candidates where the raw materials were milled out and collected for comparison. Figure 4-3 indicates that the grading of Sheffield Road and Felspar Road lies between the 75%CRB+25%CLS and 50%CRB+50%CLS curves in terms of the intermediate course.

Examination of the fine content showed that 50%CRB+50%CLS was representative of the field recycled material which possesses similar fine content.

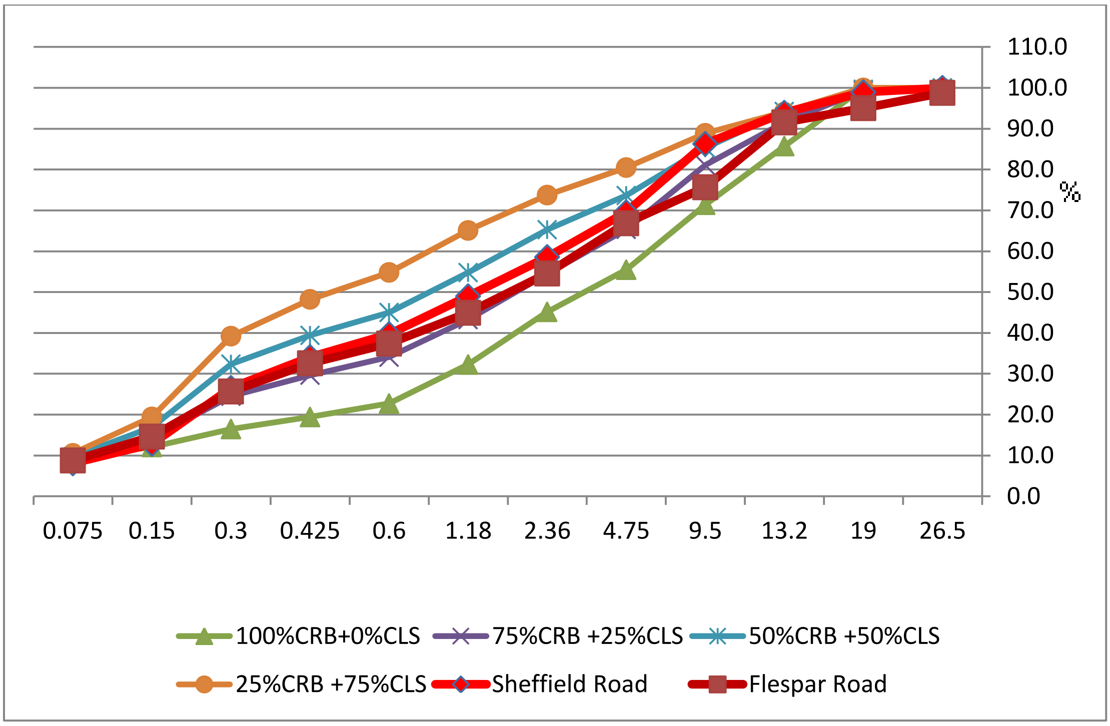

Figure 4-3: Aggregate mixture gradations in comparison with field recycled materials

4.4.2. Maximum Dry Density

The results for maximum dry density were derived from the compaction testing after adding foamed bitumen into the mixture, as shown in Table 4-2. Figure 4-4 is a graphical representation of Table 4-2. A general trend that can be observed is that maximum dry density gradually decreases with an increase in foamed bitumen and crushed limestone content. It was noted that the 50%CRB+50%CLS at 3% foamed bitumen and 25%CRB+75%CLS at 4% foamed bitumen as additional bitumen did not decrease the density, instead a small peak point was observed. It was inferred that at this specified bitumen content, the mixture was at its optimum condition and the bitumen content should be recorded as the optimum bitumen content.

Table 4-2: Maximum dry density with each different proportion aggregate and foamed bitumen content (after Huan et al. 2010)

Maximum Dry Density (g/cm^3)		Foamed Bitumen Content (%)			
		0	3	4	5
Aggregate Mixtures	100%CRB+0%CLS	2.385	2.221	2.168	2.163
	75%CRB+25%CLS	2.252	2.178	2.062	2.034
	50%CRB+50%CLS	2.081	2.092	2.021	1.984
	25%CRB+75%CLS	1.989	1.902	1.922	1.899

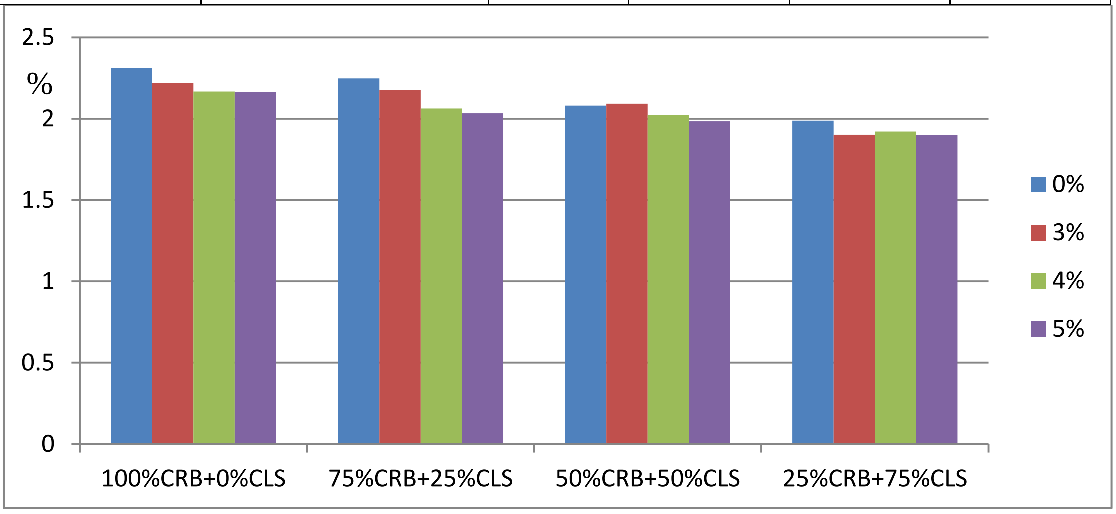

Figure 4-4: Maximum dry density of aggregate mixtures

4.4.3. Indirect Tensile Strength (ITS)

Although ITS is a simple test, it is a good means of measuring the tension resistance of the foamed bitumen mixture. Figure 4-5 presents four graphs showing the ITS results for different foamed bitumen contents.

As expected, the unsoaked ITS values exceed the soaked ITS in most cases except for 100%CRB at 3% bitumen content. Among all the testing results, 100%CRB mixed with 3% foamed bitumen content produced the highest ITS values regardless of the curing conditions. A slight difference was found in the sample of 50%CRB+50%CLS, where 4% foamed bitumen content showed the highest value whilst 3% foamed bitumen content generally exhibited the highest ITS values. It should be noted that a similarly unclear trend was observed in the case of most soaked samples. Moreover, it was found that, only in a soaked sample of 25%CRB & 75%CLS was the ITS value slightly increased with increased foamed bitumen content, compared with most cases where ITS values were reduced with increased foamed bitumen content (Huan et al. 2010).

ITS (kPa)

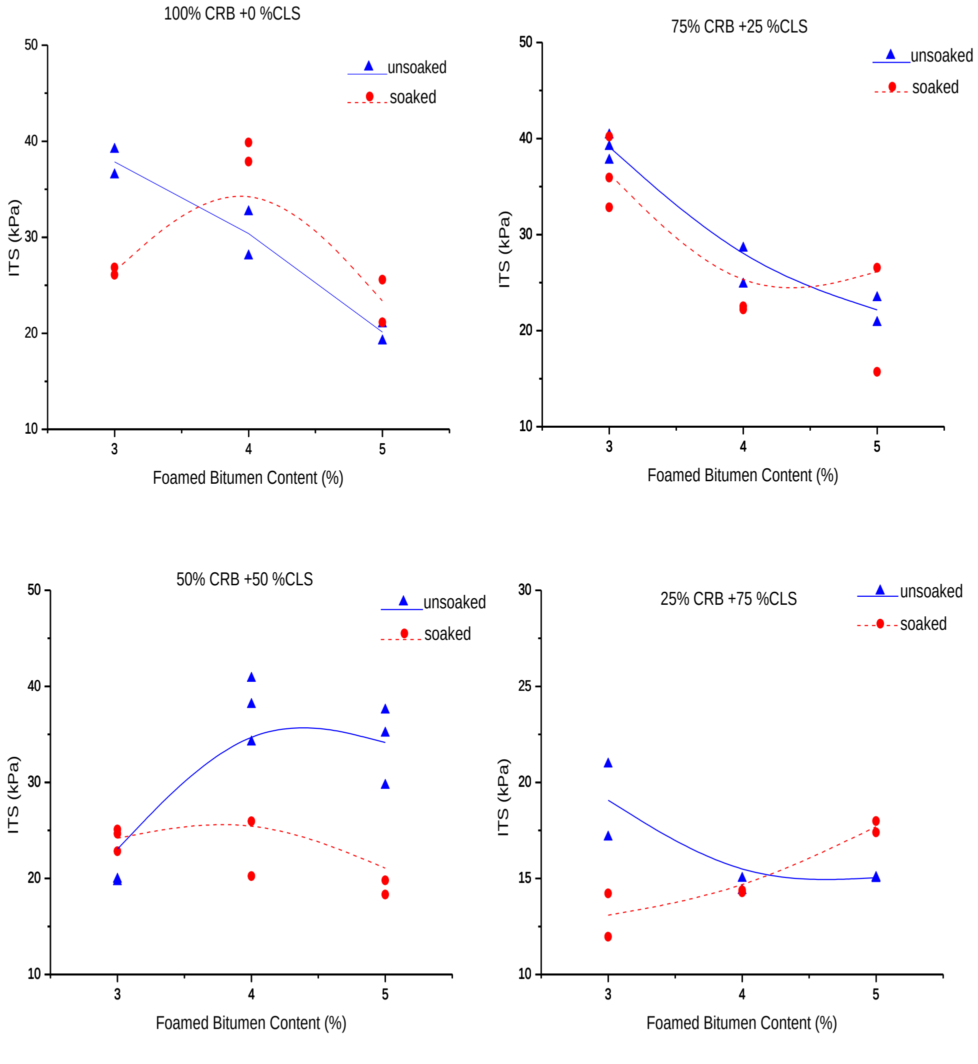

Figure 4-5: Plots of ITS versus foamed bitumen content for four different mixtures (after Huan et al. 2010)

4.4.4. Unconfined Compressive Strength (UCS)

Generally, at 3% foamed bitumen content, all the aggregate compositions returned the highest UCS values. Moreover, a blend of 75%CRB and 25%CLS provided the most superior UCS performance of all mixtures. It should be noted that the 4% foamed bitumen-treated materials demonstrated the lowest UCS values. A possible explanation is an inappropriate mixing process whereby large size particles could become coated when additional bitumen is injected, rather than the fines becoming coated, as predicted by the theory. Based on this explanation, it is not hard to understand that large particles coated with foamed bitumen would interfere with the interlocking in resisting compressive strength. However, more research is needed in the future to further examine this phenomenon (Huan et al. 2010).

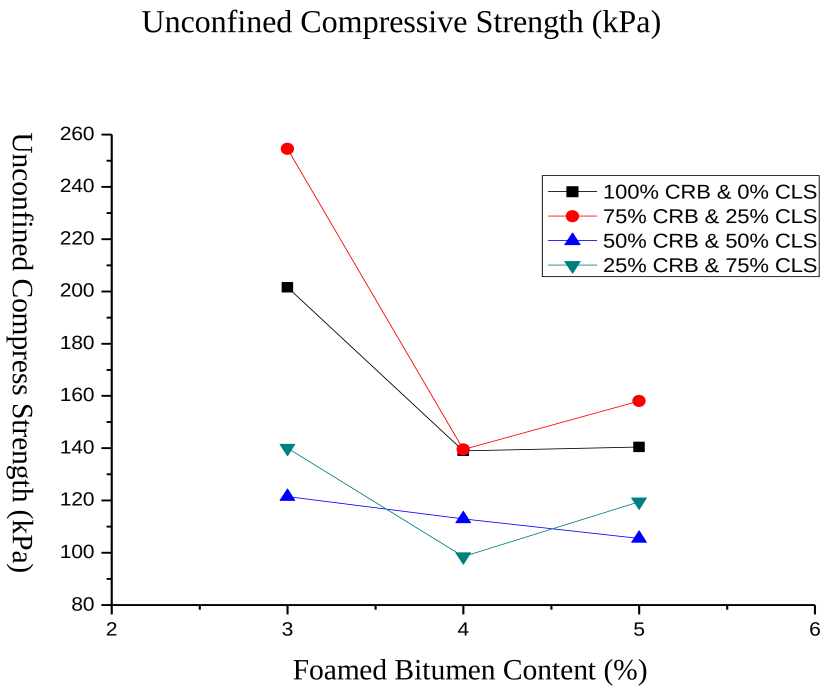

Figure 4-6: Plot of UCS versus foamed bitumen content for four different mixtures (after Huan et al. 2010)

4.5. Summary

In this preliminary study, mix designs with four different aggregate mixtures treated with three different foamed bitumen contents were observed under laboratory conditions. Based on the mechanical testing results, 75%CRB+50%CLS and 50%CRB+50%CLS were the most suitable for replicating field construction situations in terms of the gradation and ITS as well as UCS, signifying that the current recycling depth used in Western Australia is reasonable and practicable. Furthermore, density testing demonstrated that the additional limestone may decrease the maximum dry density. The blend of 50%CRB+50% LS was chosen as the representative laboratory mixture for further research purposes, as detailed in Chapter 5.

5. Strength Characteristics of Foamed Bitumen Mixture Due to Aggregate Gradation

5.1. Overview

This chapter aims to assess the effect of aggregate gradation on the foamed bitumen stabilised mixture with particular attention to the aggregate fine contents and aggregate grading envelopes. This information will be used to expand industry knowledge of the raw aggregate properties of foamed bitumen, and of its practical application within Western Australia. Further to this generalised goal, the chapter aims to contribute to the development of a standard design procedure for aggregate selection in foamed bitumen stabilisation in Western Australia, through structured laboratory testing and identification of comparable previous research in the field.

5.2. Background

Aggregate gradation is recognised as one of the most important factors when assessing the suitability of foamed bitumen as a stabilising agent for road rehabilitation processes. The suitability of specific gradations is highlighted by the recommendations of numerous papers, which provide grading curves intended to guide the evaluation of foamed bitumen use. Figure 5-1 presents a number of typically recommended grading curves, recreated from those proposed by Akeroyd and Hicks (1988) on behalf of Mobil, AustStab (2002) and Foley (2002) on behalf of Austroads.

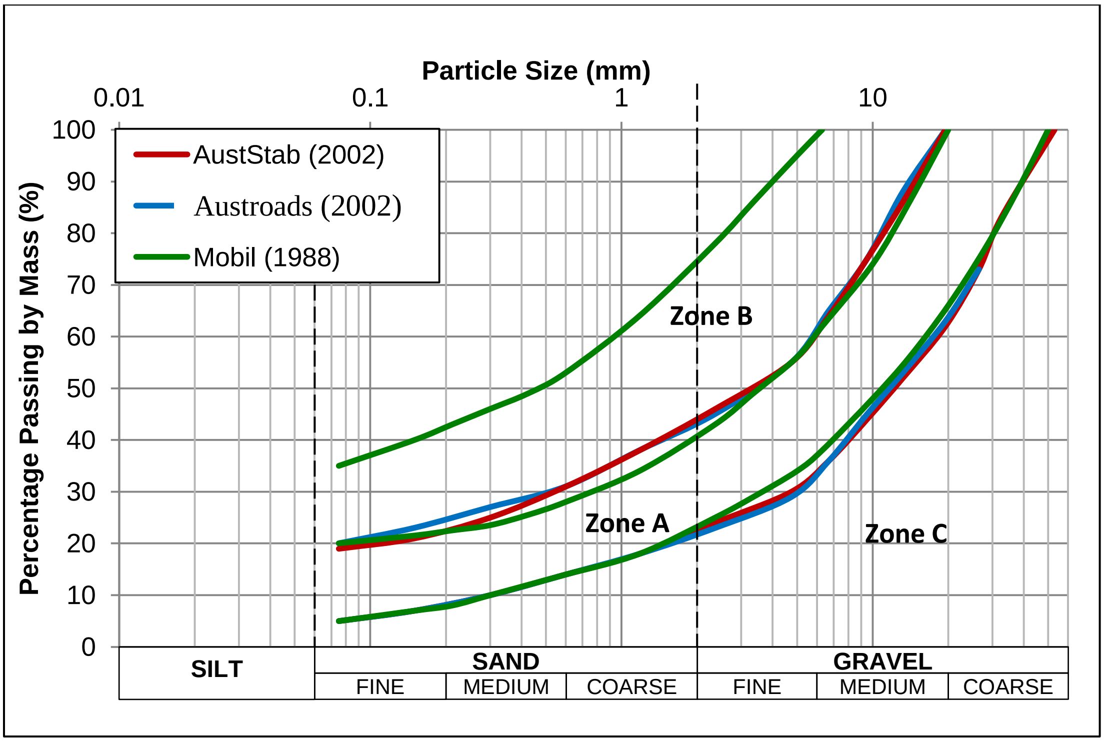

Figure 5-1: Recommended grading envelope

All three sources from which the grading envelopes have been adapted outline the importance of "zones" on the curve. Three important zones are labelled in Figure 5-1. Zone A is deemed to include materials which are ideal for stabilisation, and very little assessment and design is required prior to the application of foamed bitumen stabilisation. Zone B indicates materials which are suitable, but need to be assessed carefully and have experimental mix preparations tested in order to determine the optimum binder content and compaction required. While both AustStab and Austroads fail to address this Zone B, the Mobil envelope provides a boundary curve for Zone B, presumably illustrating that materials outside this limit are unsuitable. Finally, Zone C indicates too much coarse material which is deemed unsuitable for foamed bitumen treatment – aggregates should be modified by the addition of fines until the final gradation lies within Zone A, or be assessed for alternative stabilisation methods.

These curves from the different sources indicate the similarities behind the theory, or the development of one specification from a pre-existing specification. As mentioned in Chapter 4, the gradation of a typical Western Australian recycled raw aggregate normally lies across Zone A and Zone B and in most scenarios the grading curves are located in Zone B. When the grading curves are located far inside Zone A, for example a material with a gradation that wholly lies in Zone C, it is interesting to examine how the strength characteristics are affected and to what extent this effect will manifest.

In a North American study, Marquis et al. (2003) observed, on the basis of results taken from a series of case studies using foamed bitumen in pavement rehabilitation, that the single most important factor affecting the performance of a properly designed foamed bitumen rehabilitated mix is the percentage of large particles, particularly those exceeding 50mm in the existing material. Marquis et al. (2003) noted that the presence of these large particles results in very low resistance to moisture damage and low modulus values. However, these tests were carried out in Maine, and the significant presence of large particles is a common occurrence in the aggregates used. Alternatively, the Western Australian aggregates commonly used in pavement construction rarely have a significant proportion of particles greater than 25mm, thus the inherent problems associated with this portion of the mix are removed entirely. Thus, as is the case in numerous other papers regarding the aggregate, the percentage of fines content is the most critical factor in designing the stabilised mix.

Significant recognition is given to the importance of fines content within a foamed bitumen mixture in many research papers dealing with the topic. The reason for this lies in the method by which foamed bitumen imparts strength through stabilisation of an aggregate material. It is understood that upon foaming, the bitumen will attract and coat the fines particles, forming a uniform matrix which effectively binds the mixture of particles together (AustStab 2002). Particle coating within an aggregate mix has a significant effect on the performance of the mix. Jenkins et al. (1999) note that particle coating is especially important in foamed preparations, as the inherent strength is gained by “spot-welds” of bitumen droplets providing tensile strength in the mix. This is an important consideration, as foamed bitumen will bind readily and coat fines particles as dispersed throughout the mix. A careful assessment of the fines within a mix will therefore allow for optimisation of the abovementioned mechanism.

5.3. Experimental Design

The test procedure can be divided into two dependent parts including aggregate gradation envelopes research and aggregate fine contents research, as shown in Figure 5-2. As recommended in Chapter 4, a blend of 50%CRB+50%CLS was chosen as the parent material with an alternation on either grading or fine content.

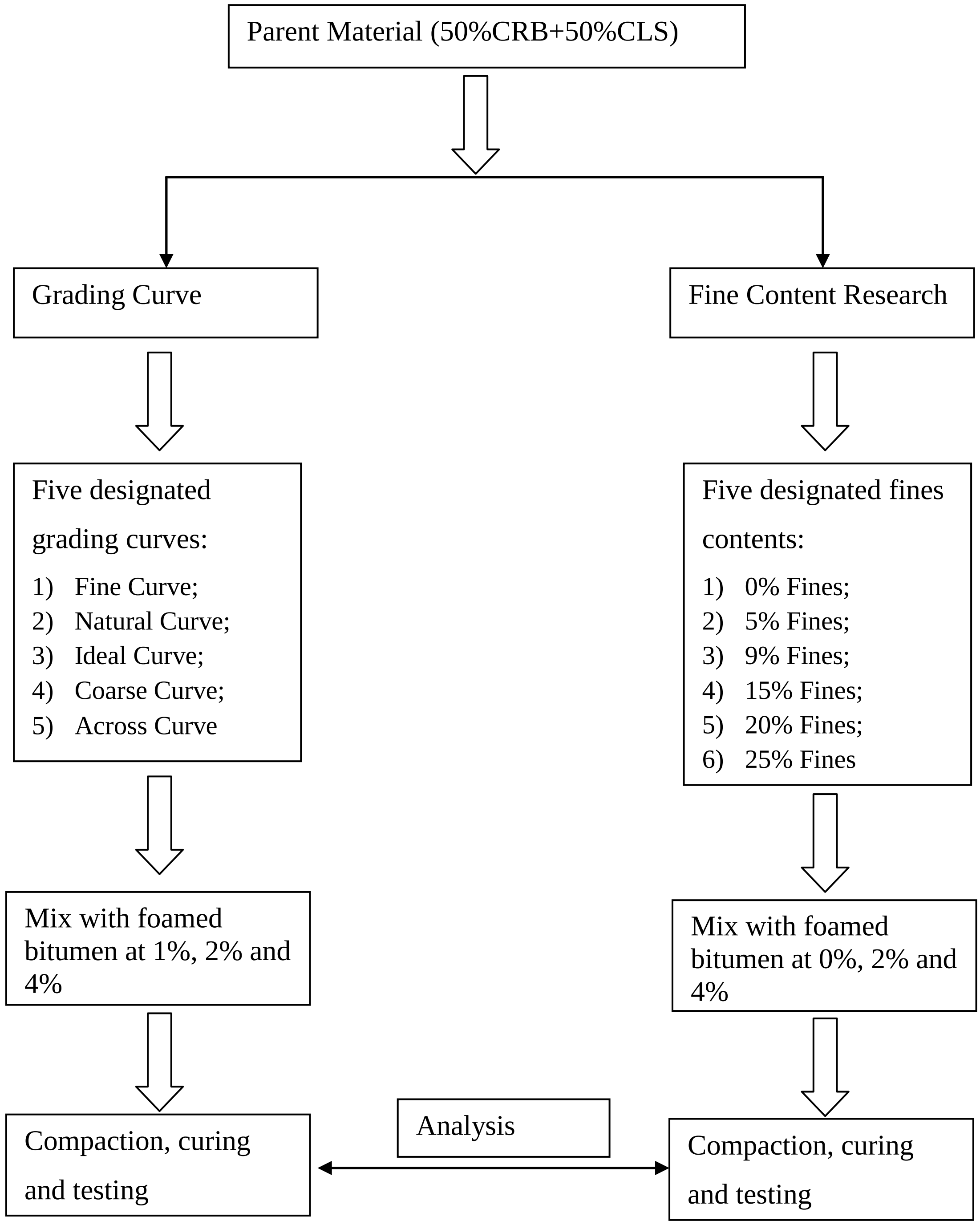

Figure 5-2: Flow chart of test methodology

The sample preparation phase was recognised as the hardest part during this project. In the grading alternation stage, parent material had to be sieved into each individual size first and then recombined to achieve the required designated curves. The following five curves were prepared:

1) Fine Curve: the grading wholly lies in Zone B which is the fine section;
2) Natural Curve: the parent material without any adjustment whose curve lies between Zone A and Zone B;
3) Ideal Curve: the grading wholly lies in Zone A which is deemed as the most suitable curve;
4) Coarse Curve: the grading wholly lies in Zone C which is the coarse section;
5) Across Curve: the grading is across all the three Zones i.e. fine section locates in Zone C, intermediate section locates in Zone B and coarse section locates in Zone B.

For Part 1, removal of fines was found to be necessary in the first step. Fines were removed by bulk washing of aggregates, which was undertaken by adding water, agitating the aggregate, then decanting over a 75µm wash sieve. This process involved washing aggregates a number of times to ensure that fines were removed. The following step was to re-introduce an inert filler, baghouse dust, to provide the increased content of fines where appropriate.

The variations in this project are listed as: 0%, 5%, 9%, 15%, 20% and 25%. The intention was to consider 0% and 25% fines content which are outside of the normal range of recognition, in order to examine the strength development in these two extreme cases.

Subsequently, different percentages of foamed bitumen were injected into the mixture to fabricate foamed bitumen mixtures with different properties. With laboratory compaction and curing conditions kept constant, a series of mechanical tests was undertaken to compare the development of strength characteristics. These tests include PSD, ITS, ITM_R and UCS.

5.4. Result and Discussion

This section discerns the testing results in terms of PSD, Grading Characteristics, ITS, ITM_R as well as UCS.

5.4.1. Particle Size Distribution

5.4.1.1. Performance Effects of Particle Size Distribution

Figure 5-3 presents the PSD test results for all sample batches. Specifically, the gradation curves of Groups 1 and 4 lay outside of the ideal envelope, in the 'unsuitable' ranges named as fine curve and coarse curve, respectively. The natural aggregate (Group 2) is gap graded in the mid-sized and coarse aggregate, being deficient in the 0.3–14mm size. Group 5 is dominant in the sand sized aggregate. Group 3 is designed to fit within the ideal zone. It would be expected that Curves 1, 2, 4 and 5 would demonstrate inferior performance compared with Curve 3, which is perfectly located in the ideal zone and thus would demonstrate optimum performance.

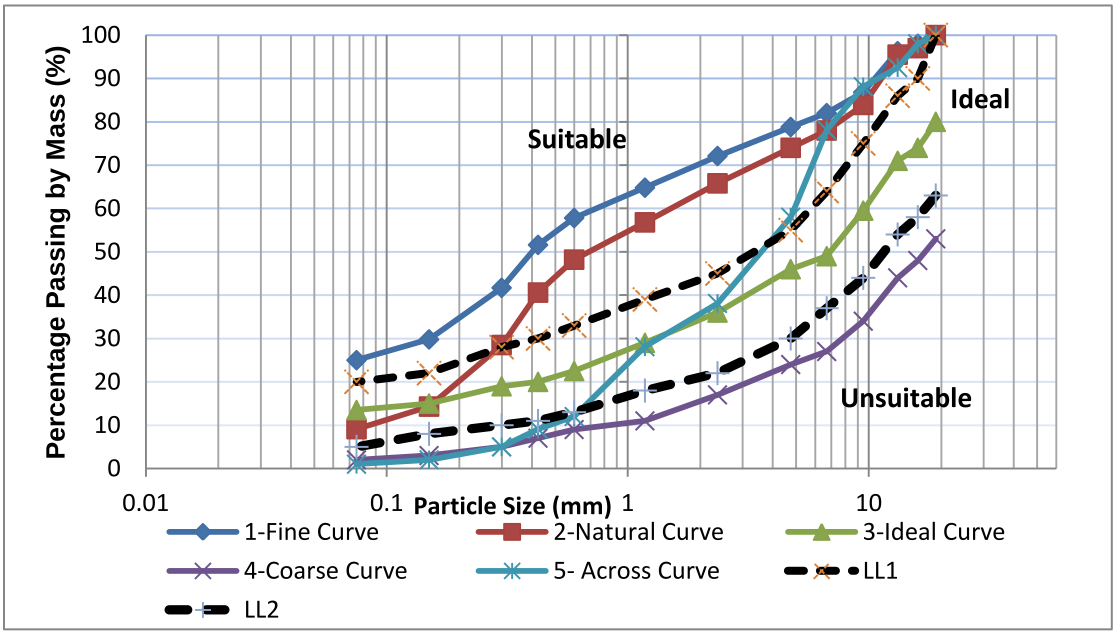

Figure 5-3: Particle size distribution of aggregate grading

5.4.1.2. Performance Effects of Fines Variation

A summary of the typical particle size distribution for each batch is presented in Table 5-1. Variations will occur over the entire grading due to the shift in percentage composition arising as a result of fines addition, yet these are minimal.

Table 5-1: Particle size distribution for fines-content samples

	Fines Content of Test Batches (% by mass)					
Particle Size (% by mass passing sieve)	0	5	9.1 (Natural)	15	20	25
19.0 mm	100.0	100.0	100.0	100.0	100.0	100.0
13.2 mm	95.5	95.5	95.5	95.8	96.1	96.3
9.50 mm	83.9	83.9	83.9	85.0	85.9	86.8
4.75 mm	74.0	74.0	74.0	75.8	77.3	78.8
2.36 mm	65.8	65.8	65.8	68.1	70.2	72.1
1.18 mm	56.8	56.8	56.8	59.8	62.3	64.8
0.60 mm	48.2	48.2	48.2	51.7	54.8	57.8
0.425 mm	40.6	40.6	40.6	44.6	48.1	51.6
0.30 mm	28.5	28.5	28.5	33.4	37.6	41.7
0.150 mm	14.3	14.3	14.3	20.0	25.0	29.8
0.075 mm	0.0	5.0	9.1	15.0	20.0	25.0

Figure 5-4 presents the grading curves for all sample batches, identifying the gradation range in which aggregate testing will occur. As can be noted, the majority of the gradation curves for sample aggregate lie somewhat on the finer upper limit of recommended grading envelopes suggested by other literature.

Only the fine portion of gradation curves is shifted outside of this envelope, with batches consisting of 0% and 5% fines lying in or close to the “unsuitable” range. The 20% and 25% fines batches remain entirely within the “suitable” envelope. The 15%, 9.1% and 5% fall within the suitable range at >0.15mm aggregate size and the ideal range in the <0.15mm aggregate size. The 0% fines fall into the unsuitable range only in the 0.075mm sieve. Following guidance presented by previous literature, these gradation curves should indicate that the 0% and 5% batches will perform significantly worse than other batches, and that the natural (9.1%) and 15% fines batches should display superior performance.

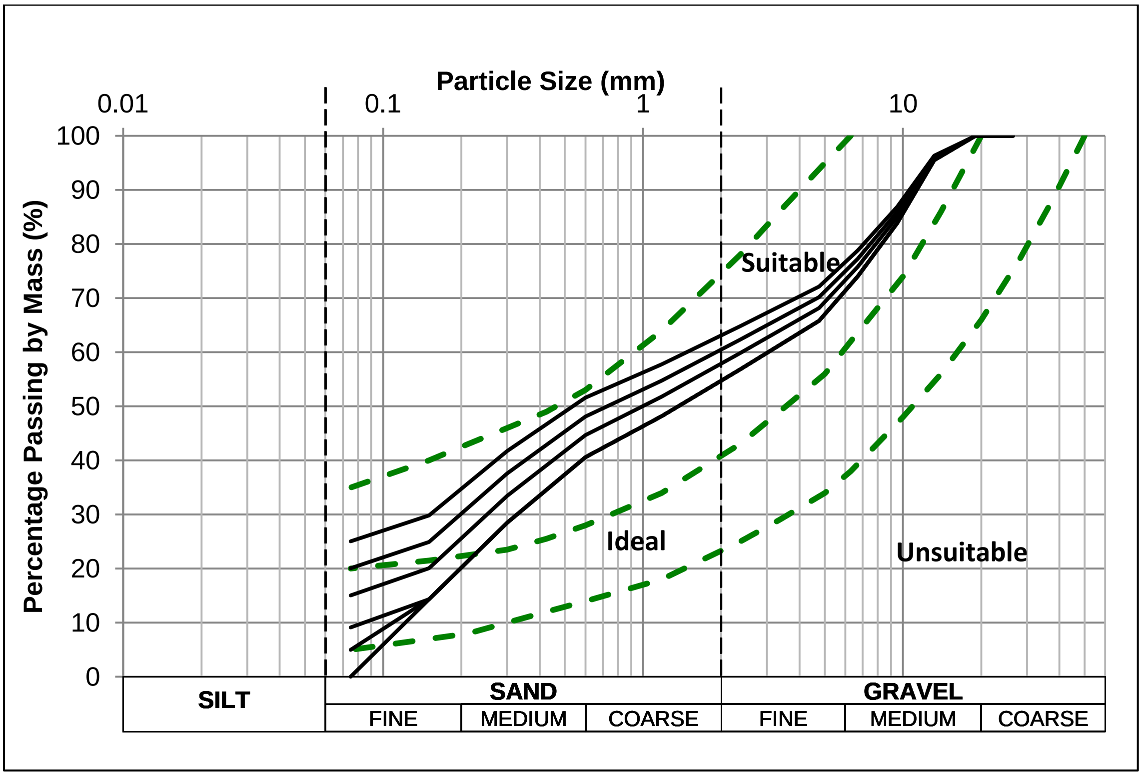

Figure 5-4: Particle size distribution of specimen grading

5.4.2. Grading Characteristics

5.4.2.1. Performance Effects of Particle Size Distribution

Table 5-2 indicates that Curve 1 and 3 are well graded as their C_z values are in the range of 1–3, while all other mixes are classed as poorly graded. Observation of this classification provides an important basis on which to assess the suitability of aggregates by means other than simply fitting the gradation envelopes.

Table 5-2: Grading characteristics of different aggregate mixtures

	Batch No.				
	1	2	3	4	5
D_{10}	0.01	0.08	0.05	0.9	0.51
D_{30}	0.15	0.36	1.2	8.1	1.2
D_{60}	1.13	1.7	9.7	20	4.9
C_Z	1.99	0.95	2.97	3.65	0.58

5.4.2.2. Performance Effects of Fines Variation

The values shown in Table 5-3 indicate that mixes of 15%, 20% and 25% fines are well graded, while all other mixes are classed as poorly graded.

Table 5-3: Grading characteristics of different fines content in aggregate mixtures

	Fines Content of Aggregate Mix					
	0%	5%	9.1%	15%	20%	25%
D_{10}	0.12	0.11	0.08	0.02	0.01	0.01
D_{30}	0.31	0.31	0.32	0.23	0.2	0.4
D_{60}	3	3	3	2.3	2	1.3
C_Z	0.27	0.29	0.43	1.15	2.00	1.51

5.4.3. Indirect Tensile Resilient Modulus

5.4.3.1. Performance Effects of Particle Size Distribution

ITM_R testing was undertaken on six specimens from each batch, three of each undertaken on unsoaked and soaked samples.

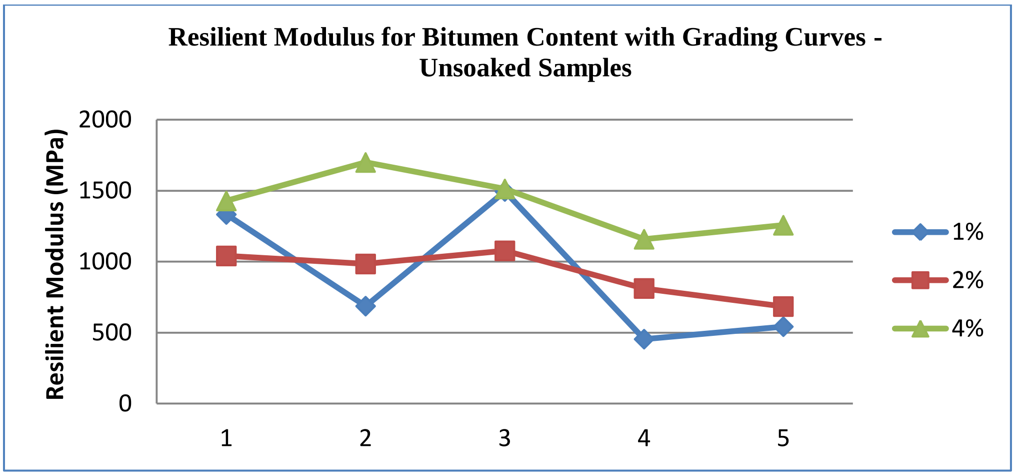

Figure 5-5: Resilient modulus for unsoaked samples in different grading curves

The plot in Figure 5-5 shows a slightly increased modulus with grading characteristics for both 2% and 4% bitumen content. Conversely, the 1% foamed bitumen specimens showed a fluctuating trend in resilient modulus.

A relatively constant modulus variation was seen for the 2% and 4% foamed bitumen content groups. This phenomenon may occur because the additional bitumen can somehow stabilise the aggregate particles.

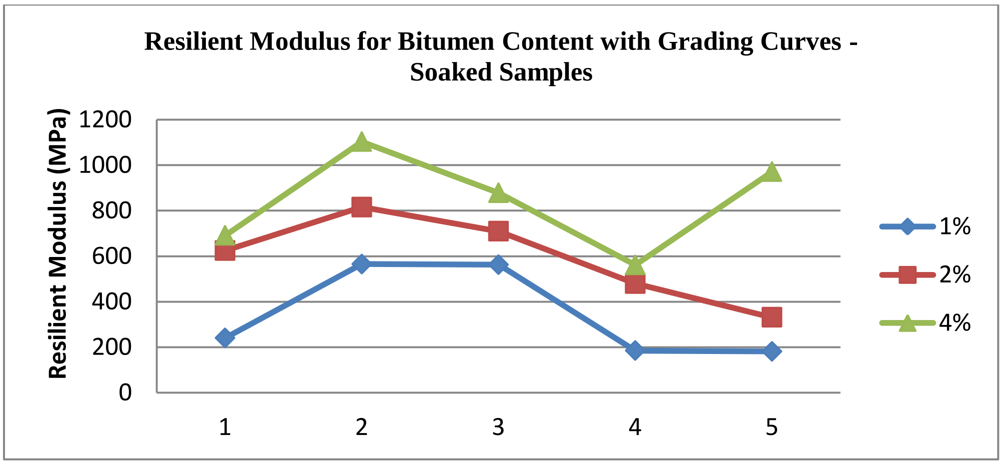

Figure 5-6: Resilient modulus for soaked samples in different grading curves

From the soaked test results shown in Figure 5-6, an increased percentage of foamed bitumen content provides a more consistent resistance to moisture sensitivity. At each bitumen content level, Curve 2 always represents the highest modulus value as is expected through the gradation analysis. Unfortunately, no flamboyant performance was observed in Curve 3 as it was slightly lower than Curve 2. A strange upward trend was found for 4% foamed bitumen content across Curve 5. There is no reasonable explanation for this phenomenon once laboratory operation error is excluded.

5.4.3.2. Performance Effects of Fines Variation

Unsoaked Samples

The ITM_R for unsoaked samples showed a significant trend within samples of varied fines content, between the separate batches of varied bitumen content. Figure 5-7 presents the results for unsoaked indirect tensile resilient modulus for all samples, taken as an average between specimens.

The plot presents a clear trend of an increased modulus with fines content for both 2% and 4% bitumen content, reaching a maximum at 20% fines. Conversely, specimens prepared without foamed bitumen show no appreciable increase in modulus until fines content is increased to 25%. Specimens prepared with 4% foamed bitumen display a strong, near-linear trend of increasing modulus with fines content, however this reduces significantly after increasing fines content to 25%. This reduction in modulus is displayed almost identically in the 2% foamed bitumen specimens, which present very similar results for both 20% and 25% fines content. However specimens prepared with 2% foamed bitumen only show an appreciable increase in modulus over the 4% foamed bitumen mixes at extremely low fines contents (0–5%).

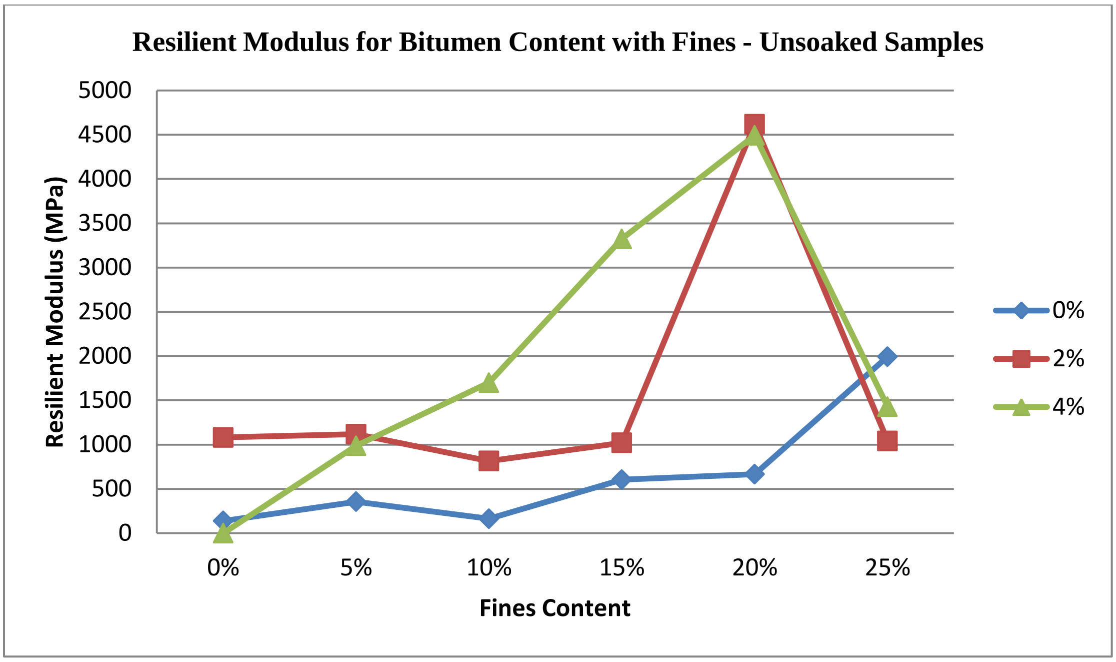

Figure 5-7: Resilient modulus for unsoaked samples in different fines content

An important observation is the difference between the trends displayed by stabilised sample groups. Both 0% and 2% foamed bitumen groups display a relatively constant modulus value for varied fines contents, except where the maximum value is observed. Conversely, the 4% FBS mix presents a gradual increase up to a maximum addition of fines, after which it decreases significantly following the addition of further fines.

<u>Soaked Samples</u>

Soaked sample curves present a similar trend to that of unsoaked samples, indicating (for batches prepared with bitumen) a general increase in resilient modulus with fines content up to 20% fines, and a significant reduction in modulus at 25% fines, as illustrated in Figure 5-8. While the trend remains relatively constant for foamed bitumen treated specimens, it is evident that for those prepared without bitumen, moisture sensitivity increases significantly at fines contents above 15%.

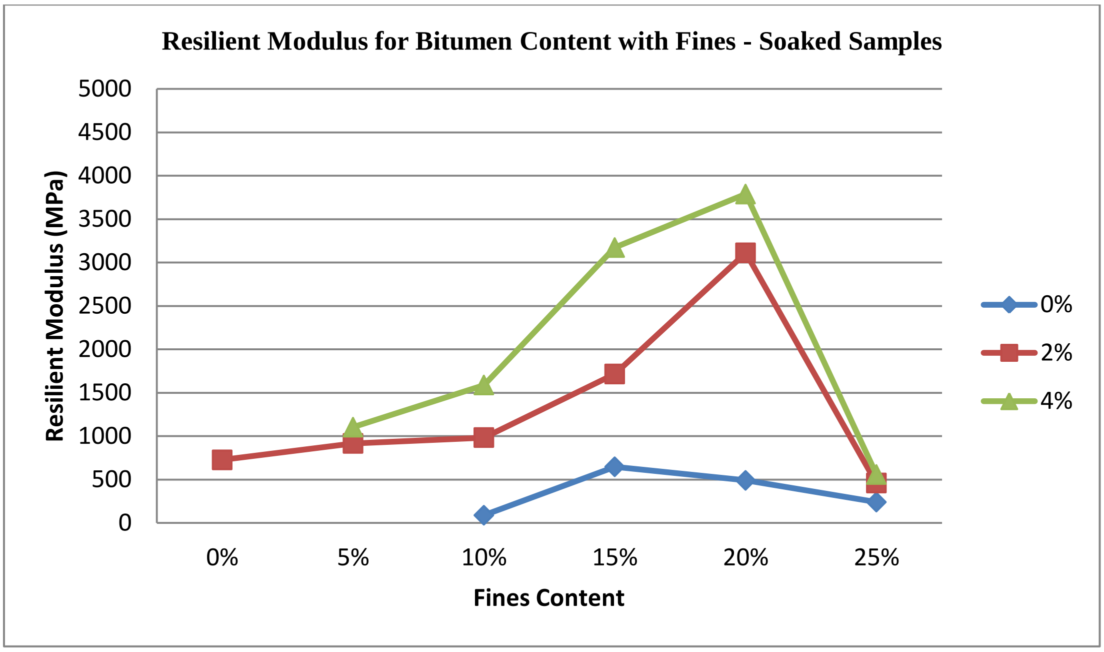

Figure 5-8: Resilient modulus for soaked samples in different fines content

<u>Fragile Samples</u>

Of the 108 sample specimens prepared, four samples were crushed during ITM$_R$ testing as they were not strong enough to withstand the seating force applied. Figure 5-9 displays one such sample – as can be seen the actuator arm has descended, and the disturbed specimen has fallen apart. These specimens included:

Figure 5-9: ITM$_R$ crushing failure

1. one unsoaked sample of 0% fines and 4% foamed bitumen;
2. two soaked samples each of 0% fines content, with 0% and 4% foamed bitumen (pictured); and
3. one soaked sample of 5% fines content and 0% foamed bitumen.

As a result of this damage, a decision was made not to undergo further ITM$_R$ testing on the remaining samples from these sample groups, and as such ITM$_R$ data is not available for these specimens.

Undertaking ITM_R testing on samples with zero fines proved somewhat difficult, not only due to the crushing of some weaker samples, but also due to the occurrence of ongoing deformation during testing. Of the samples tested, a trend was noticed in which samples underwent significant deformation during the pre-conditioning phase, and then in subsequent testing pulses. This ongoing deformation indicates instability within the sample, possibly due to less rigid foamed bitumen mastic and a reduced compactibility. The result of this is that the sample strength is primarily gained from granular friction, however a lack of fines means that the particles must realign somewhat, resulting in some granular "settling".

Similarly, soaked samples of fines content less than 10% proved difficult to test, as loading responses were highly variable. This is expected to be due to the effects of pore pressures within the specimen, as trapped water will attempt to flow as a result of the induced strain. As such the response becomes somewhat more "dynamic" and results are highly variable. This is important to note as it provides an insight into the behaviour of the material under loading in an inundated environment, and identifies possible causes of failure.

5.4.4. Indirect Tensile Strength

5.4.4.1. Performance Effects of Particle Size Distribution

ITS testing also was undertaken on six specimens from each batch, three of each undertaken on unsoaked and soaked samples.

Unsoaked Samples

Overall, a trend towards increasing tensile strength is found regardless of the grading curves, and 4% foamed bitumen exhibits two times greater tensile strength than the other two counterparts, as shown in Figure 5-10. Another apparent trend is that Curve 3 presents the greatest performance, as expected, no matter what the bitumen content variations. It is unclear why two concave points are found at Curve 2 and Curve 4. A possible explanation is that particle orientation in these two serials is disordered due to the lack of fines content.

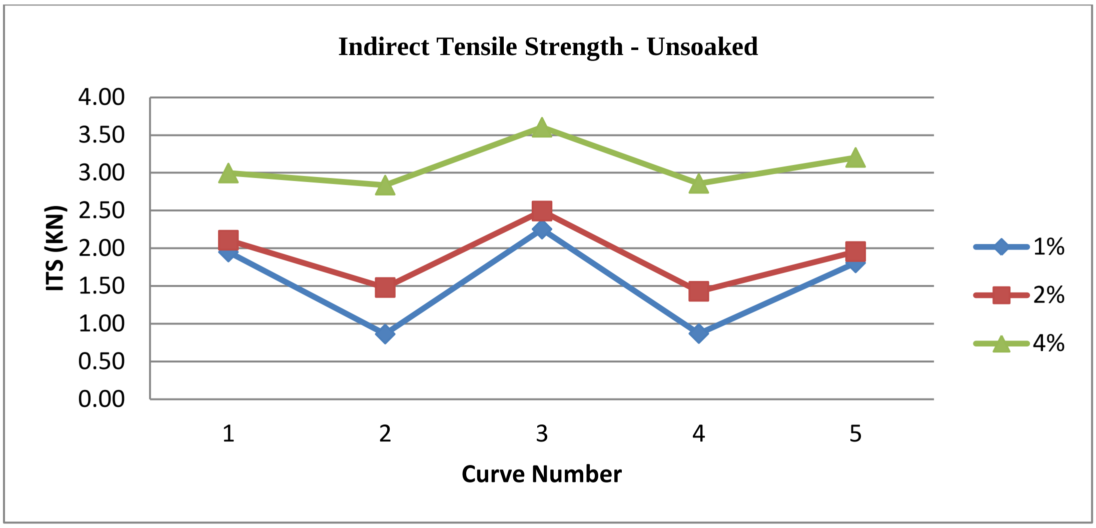

Figure 5-10: Tensile strength for unsoaked samples in different grading curves

Soaked Samples

No obvious difference can be determined between the soaked samples and unsoaked samples as these two graphs are very similar. Figure 5-11 illustrates the tensile strength for soaked samples in different grading curves. As a general trend, the samples gain tensile strength with increasing bitumen content. Curve 3 still presents the greatest performance of all of the curves.

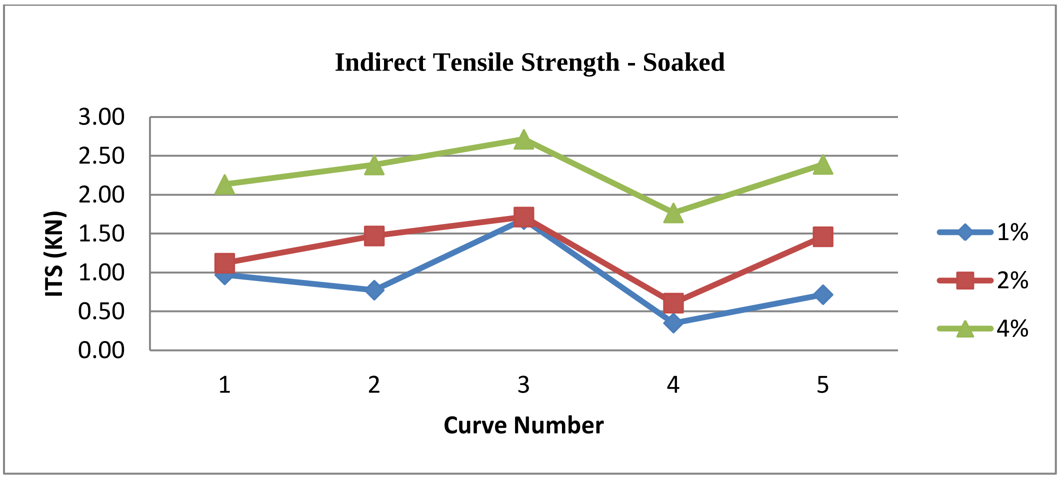

Figure 5-11: Tensile strength for soaked samples in different grading curves

5.4.4.2. Performance Effects of Fines Variation

ITS testing was undertaken on six specimens from each batch, three of each undertaken on unsoaked and soaked samples.

Unsoaked Samples

Of the unsoaked samples tested for ITS, those prepared with 15% fines content and 4% foamed bitumen displayed the greatest performance, with an average strength exceeding 400kPa. This peak value is the result of a trend within the 4% bitumen samples of increasing strength with fines contents less than 15%, and a similar incremental decrease in strength as fines content was increased above 15%. This trend can be observed in Figure 5-12, where the results for ITS testing are presented as an average value for all specimen groups.

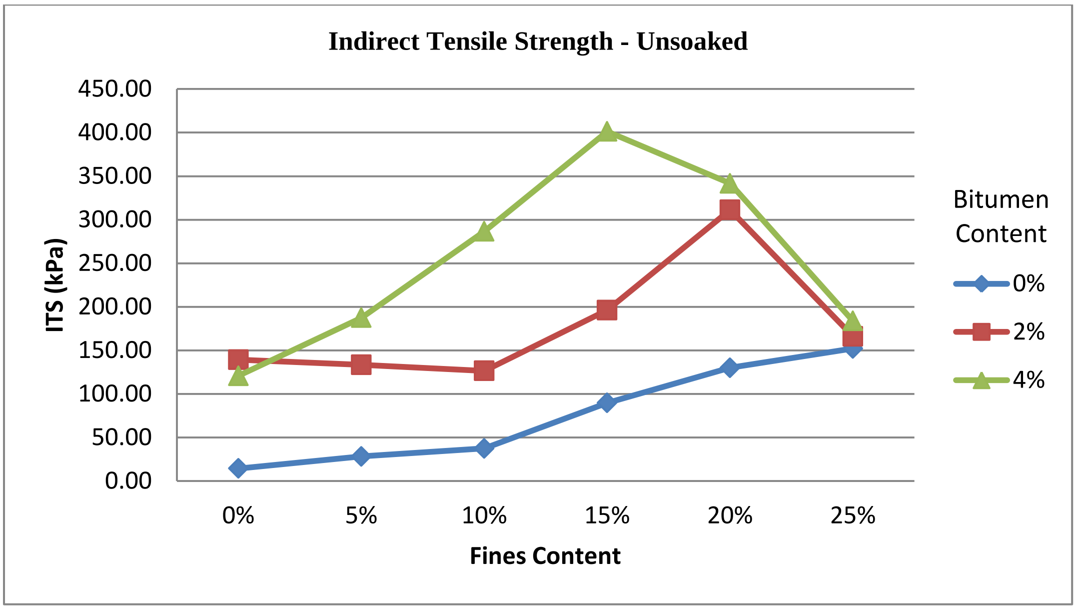

Figure 5-12: Tensile strength for unsoaked samples in different fines content

For the majority of fines contents tested, the 4% foamed bitumen samples displayed the greatest tensile strength, with the only exception being at 0% fines, where the 2% fines samples displayed a slight increase in tensile strength performance.

Unlike the 4% bitumen samples, those prepared with 2% foamed bitumen displayed a relatively unchanged tensile strength for samples of fines content ranging from 0% to 9.1%. An increase in strength is then notable at 15% and again at 20% fines, followed by a reduction once more at 25% in a similar fashion to the 4% foamed bitumen samples at the higher fines contents. This trend among the 2% foamed samples may be attributable to the existence of voids within the soil structure, which are replaced by the foamed bitumen coated fine particles – below 15% fines, no appreciable change is observed with the addition of fines. However when the voids are filled, an increase both in granular friction and in bonds formed by the foamed bitumen mastic results in an increase in sample strength. Similarly, as fines are increased above 20% and voids are no longer present, the bitumen and fines particles begin to replace larger particles within the mix, while an increase in fine particles may result in poorer coating of these particles, and thus diminished mastic strength.

Soaked Samples

A pattern similar to that displayed in the unsoaked samples was observed after undertaking testing on the soaked specimens. Figure 5-13 presents the results of ITS for soaked samples, where comparison with Figure 5-12 identifies nearly identical trends for strength gain with fines content for each separate mix.

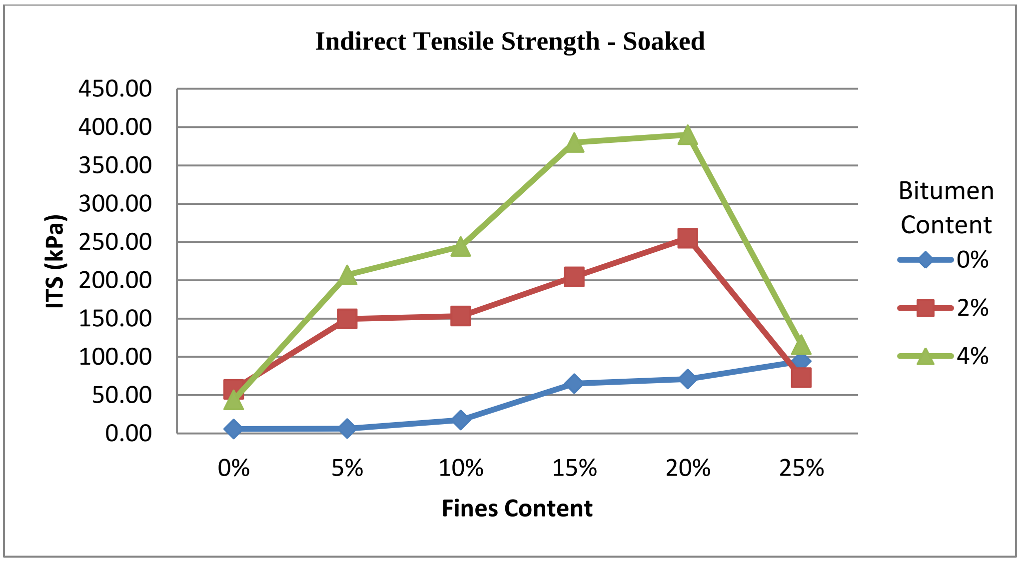

Figure 5-13: Tensile strength for soaked samples in different fines content

As in the soaked samples, the 0% bitumen mix gained strength with increasing fines content with a near-linear relationship. Similarly, the 2% foamed bitumen mix presented a peak value at 20% fines as with unsoaked samples, and the 4% bitumen mix displayed a similar trend which implies a peak value at approximately 17% fines.

5.4.5. Unconfined Compressive Strength

5.4.5.1. Performance Effects of Particle Size Distribution

Figure 5-14 displays the results of UCS testing as an average of specimen performance for each sample group. Curve 3 is observed to have the highest compressive strength at each bitumen content percentage as expected. This may be due to the effect of increasing density within the soil structure, resulting in fewer voids and an increased effect of granular friction, whilst also increasing the cemented structure of the specimen. Unlike the results for the tensile samples, 2% foamed bitumen offers higher compressive strength than 4% foamed bitumen. This can be explained by the theory that the main strength is from the aggregate interlock rather than the foamed bitumen mastic adhesion in the mixture compared with the indirect tensile strength. Besides which, more bitumen is not beneficial to the compressive strength because more bitumen acts as a sort of lubricant, bringing the strength down.

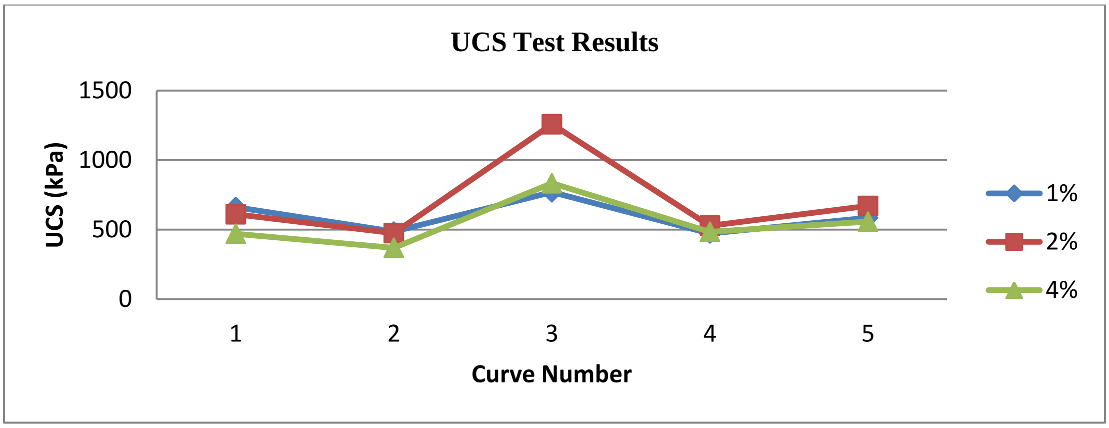

Figure 5-14: UCS results for different grading curves

5.4.5.2. Performance Effects of Fines Variation

Results of UCS testing displayed a similar trend to previous tests of increasing strength with fines content. As presented in Figure 5-15 below, samples prepared without foamed bitumen are observed to receive a near-linear increase in compressive strength from 179kPa at zero-fines to 662kPa at 25% fines.

Further analysis of the 0% foamed bitumen curve indicates a sharp increase beyond the natural fines content, with 15%, 20% and 25% fines content samples appearing to display a more significant strength gain with fines addition.

Samples prepared with 2% foamed bitumen present a trend similar to those of the 0% foamed mix at lower fines contents. While there is a strong linear correlation between compressive strength and increasing fines content for the 2% foamed mixes from 0% to 15% fines, no appreciable strength gain is recognised in comparison to the 0% foamed bitumen samples. However as fines content is increased to 20%, compressive strength is observed to increase significantly, yet reduces just as significantly with a further increase in fines to 25%. While the significant increase in compressive strength at 20% fines content indicates optimum fines content for 2% foamed mixes, there is no appreciable increase in compressive strength when compared to the 0% foamed mix at the same fines content.

Similarly, samples prepared with 4% foamed bitumen displayed a weak trend of a comparatively minor increase in strength with increasing fines. However it should be noted that for 4% foamed bitumen, as with 2%, a fines content of 20% presents a peak compressive strength.

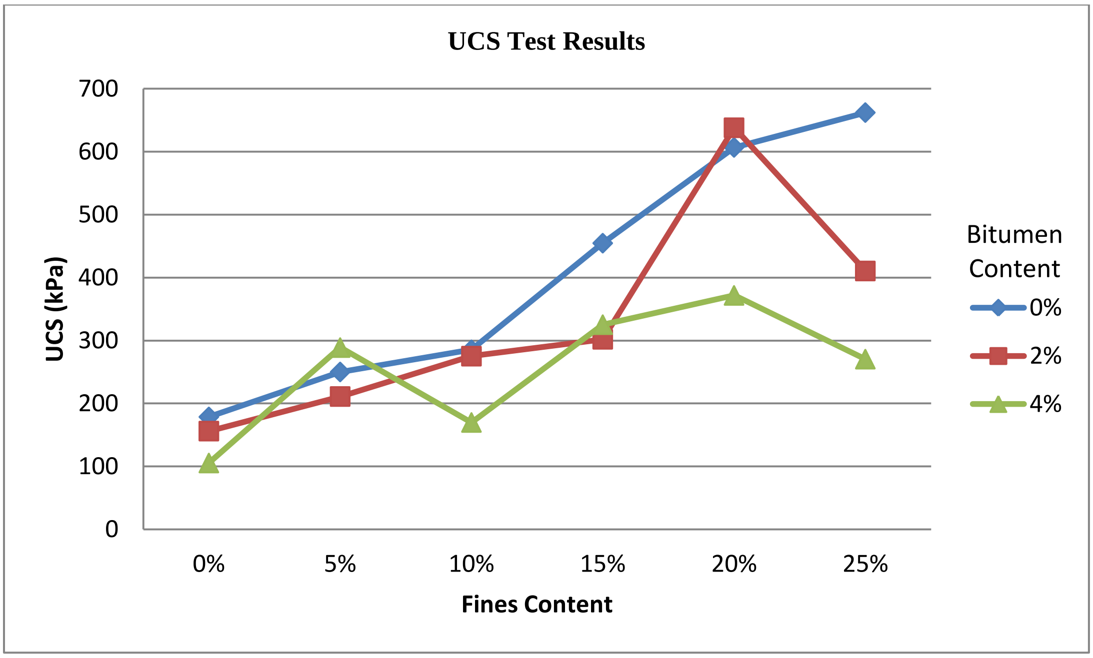

Figure 5-15: UCS results for different fines content

5.5. Summary

This chapter investigates the variations on the aggregate gradation and aggregate fine contents with an attempt to determine a better aggregate selection for the future work based on three mechanical testing methods. The results indicate that aggregate with the gradation located in the ideal zone performs better when stabilising with foamed bitumen. More importantly, the importance of fine content has also been highlighted in this chapter that the lack of fine content may reduce the aggregate performance when incorporating with foamed bitumen. More detailed conclusion has been summarised in Chapter 6.

6. CONCLUSION AND RECOMMENDATIONS

6.1. Conclusion

The overall objective of this research study was to examine the development of mechanical strength when the aggregate composition and gradation properties are changed. The study started with a literature review on the relevant information and the development of a laboratory experimental procedure suited to Western Australian conditions. A series of mechanical tests were carried out to investigate the strength development. The conclusions drawn from this paper are presented below with recommendations made on the basis of these conclusions and on trends recognised between testing results acquired under Western Australian laboratory experimental procedures and those from similar national and international literature.

6.1.1. Aggregate Composition

The outcomes of aggregate composition are:

1) Two laboratory blends of 75%CRB+25%CLS and 50%CRB+50%CLS were found to be suitable for replicating the field construction situations in terms of gradation and ITS as well as UCS test results.
2) Accordingly, the current recycling depth, 250–300mm from existing top wearing course downwards, is signified to be reasonable and practical for the foamed bitumen in-situ stabilisation process.
3) Different aggregate compositions would require varied foamed bitumen contents as optimum foamed bitumen is not a simple constant value and should be determined beforehand by means of laboratory testing methods.
4) Additional crushed limestone is found to be detrimental to the foamed bitumen mixture density as it will shift the gradation curve to the finer zone.

6.1.2. Aggregate Gradation

The findings for aggregate gradation are:

1) The grading curve of raw aggregate should be given more attention prior to the foamed bitumen stabilisation trial as varied aggregate grading can, to some extent, affect the foamed bitumen mixture strength.
2) The aggregates whose grading curves approach the ideal zone exhibit superior strength performance compared with those located outside of the boundaries. However, the mechanical test results indicate that not only those samples whose grading curves lie in the ideal zone recommended by previous literature display outstanding strength characteristics.
3) Aggregate fine content is another important factor that should be accounted for in the mix design procedure. The ITS and ITM_R test results confirm the expectation that increasing fines content will increase the strength, after which point further addition of fines reduces the tensile strength. Moreover, a general trend of similarities in UCS test results is also evident.
4) As common recycling aggregate with a fines content at 9–10% provides an acceptable strength, it is still beneficial to increase the fines content up to 15% to optimise the mechanical strength.
5) As the theory indicates that foamed bitumen mainly coats the fine particles to form the bitumen mastic, it is therefore noticeable that optimum aggregate gradation is somewhat related to the foamed bitumen content.

6.2. Recommendations

Recommendations intended to benefit the technological field that have arisen from assessment of the outcomes of this study include:

1) Crushed limestone content should be limited to no more than 50%. Any percentage over this value should be reassessed and adjusted to reach the necessary requirements.
2) The establishment of a laboratory mix design for foamed bitumen stabilisation is imperative as this study only aims to address the aggregate properties. All the other factors should also be considered: secondary binder selection, laboratory compaction, curing regimes, and mixing moisture content. Laboratory procedures need to be reviewed and adjusted to meet Western Australian conditions.

3) The current gradation curve proposed by other researchers around the world is useful as a guide but there is still a lack of knowledge and understanding of optimum gradation analysis. This therefore needs to be reviewed and rectified in order to meet local requirements.
4) Research is needed into the effects of altering various particle size ranges within the natural aggregate mix, with the intention of assessing the efficacy of adding fine sands in order to improve grading characteristics.
5) Simultaneous analysis of laboratory and field samples needs to be carried out in order to derive a relationship which correlates the significantly reduced laboratory strength to the expected field strength.

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APPENDIX

APPENDIX A: Aggregate Sieve Procedure

1. Obtain aggregate test sample and record its initial mass.
2. Nest the sieves needed for the analysis of the aggregate tightly in an ascending order, from bottom to top. The largest sieve used should be slightly larger than the nominated size (19mm) used in this study, which is a 26.5mm sieve.
3. Place a base tray at the bottom of the smallest sieve, which is 2.36mm, to collect aggregates that pass through.
4. Place the test portion (normally 2kg in this study) in the top sieve which later is covered tightly with a lid.
5. Agitate sieves using mechanical vibrate sieve shaker (shown in Figure 3-5) after all the sieves are placed in position.
6. Remove the sieves from the sieve machine with care after five minutes, weigh and record the mass retained on each sieve and in the base tray.
7. Divide portion of aggregates passing the 2.36mm sieve into adequate amounts for the fine sieve.
8. Wash the fine portion (divided portion) over a 75µm wash sieve, dry in oven and record the weight.
9. Repeat steps 1 to 6 by replacing the largest sieve size to 1.18mm and smallest sieve size to 0.075mm in another fine sieve apparatus (shown in Figure 3-6).
10. Calculate the passing percentage in each sieve to obtain the gradation curve for the aggregate sample.
11. Determine grading characteristics by means of the coefficient of curvature, C_Z, as per the equation below:

$$C_Z = \frac{D_{30}^2}{D_{60}D_{10}} \quad \text{Equation 3-1}$$

Where D_X denotes the size (in mm) of particles at which X% of all particles are less than this size.

APPENDIX B: Compaction Testing Procedure

1. Obtain 12kg of raw aggregates and oven-dry them for 24 hours.
2. Divide the dry aggregates into five plastic bags, with approximately 2kg in each plastic bag.
3. Five designated moisture contents dependent on the aggregate property, are added into each plastic bag.
4. Seal the plastic bags for five minutes and shake the bags well before the commencement of the compaction test.
5. Weigh and record the mass of the mould, W_0.
6. Measure and record the dimension of the mould, V.
7. Place the mould in a clear working area with a flat and stable bottom surface.
8. Compact the materials layer by layer with 25 equal blows each with a designated 4.9kg hammer, with a total of five equal layers.
9. Scrabble the top surface of each layer to ensure good bonding between layers after compaction.
10. Remove the top adjustable collar upon the completion of the 5th layer compaction.
11. Use a steel straightedge to level the top surface and a rubber mallet to compact the top surface to ensure the mould is at maximum capacity with minimum air voids.
12. Weigh and record an empty tray prior to the placement of the compacted sample, W_1.
13. Weigh and record the compacted sample together with the tray, W_2.
14. Oven-dry the sample over a period of 24 hours.
15. Weigh and record the oven-dried sample, W_3.
16. Repeat steps 6 to 14 with the remaining aggregate sample in each plastic bag.
17. Moisture content percentage and dry density value can be calculated with following equation:

$$\text{Moisture Content } (\%) = \text{MC} = \frac{W_2 - W_3}{W_3 - W_1} \times 100\% \qquad \text{Equation 3-2}$$

Where:

W_1 = Weight of Empty Tray (g)

W_2 = Weight of Wet Sample and Tray (g)

W_3 = Weight of Dry Sample and Tray (g)

$$\text{Dry Density} = \rho\text{dry} = \frac{\rho\text{wet}}{1+\text{MC}} = \frac{\frac{W_2 - W_0}{V}}{1+\text{MC}} \qquad \text{Equation 3-3}$$

Where:

ρwet = Wet Density (g/cm3)

MC = Moisture Content (%)

W_0 = Mass of Mould (g)

W_2 = Mass of Wet Sample (g)

V = Volume of Mould （cm3）

APPENDIX C: DETERMINATION OF OMC AND MDD ON DIFFERENT AGGREGATE COMPOSTIONS

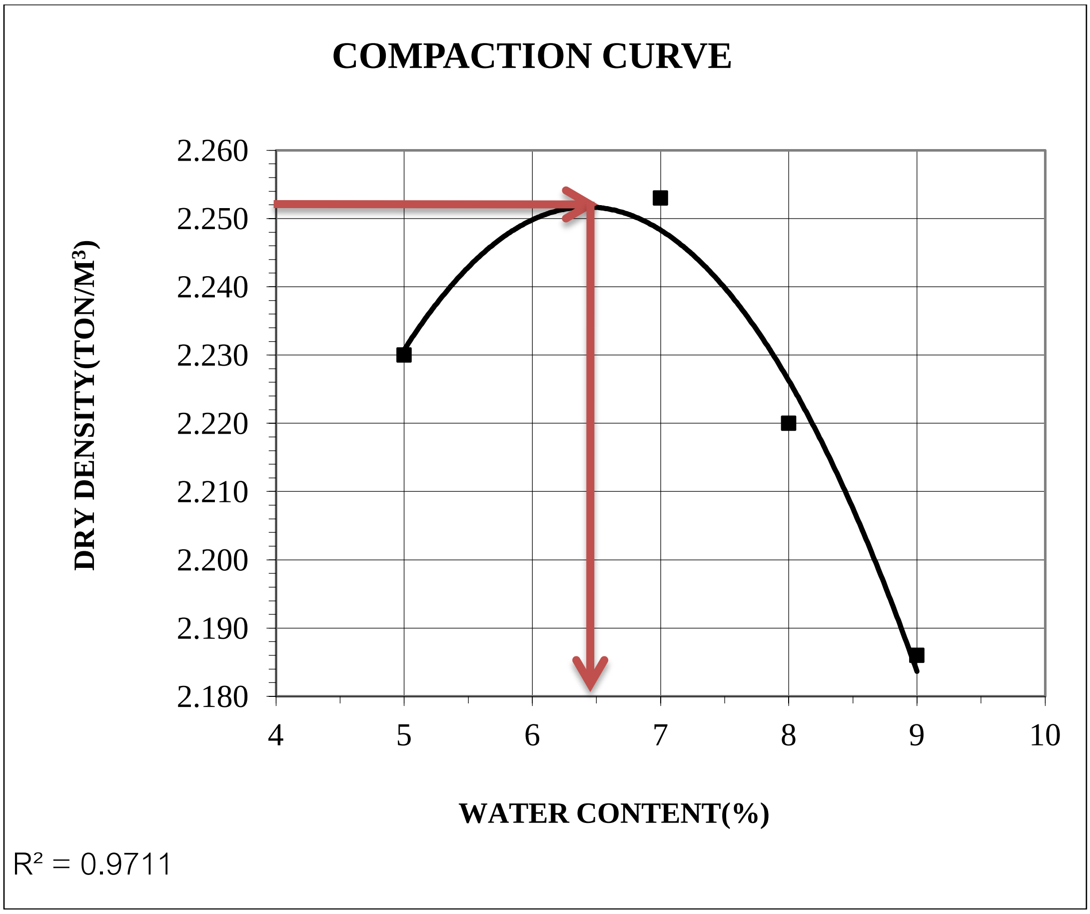
SOIL MECHANIC LABORATORY CURTIN UNIVERSITY

Project:	Foamed Bitumen Stabilisation	Source of Soil:	Perth Local Quarry
Location:	Curtin Mechanics Lab	tested Date:	31/3/2010
Soil Description:	75% CRB , 25% CLS	Test By:	Ryan H/ Richard C
Compaction Method:	Standard	Mold Dimension:	Dia105.0xH115.5mm
Hammer weight	4.9 Kg	Droped Height	450 mm
No. of Layer	5	No. of Blow	25 blows/layer

RESULT OF SOIL COMPACTION TEST

Test Number			1	2	3	4	5
Dry Density,	$\rho_{(dry)}$	ton/m^3	2.230	2.253	2.220	2.186	
Water Content.	W	%	5.00	7.00	8.00	9.00	

OMC = 6.45 %
MDD = 2.252 Ton/M^3



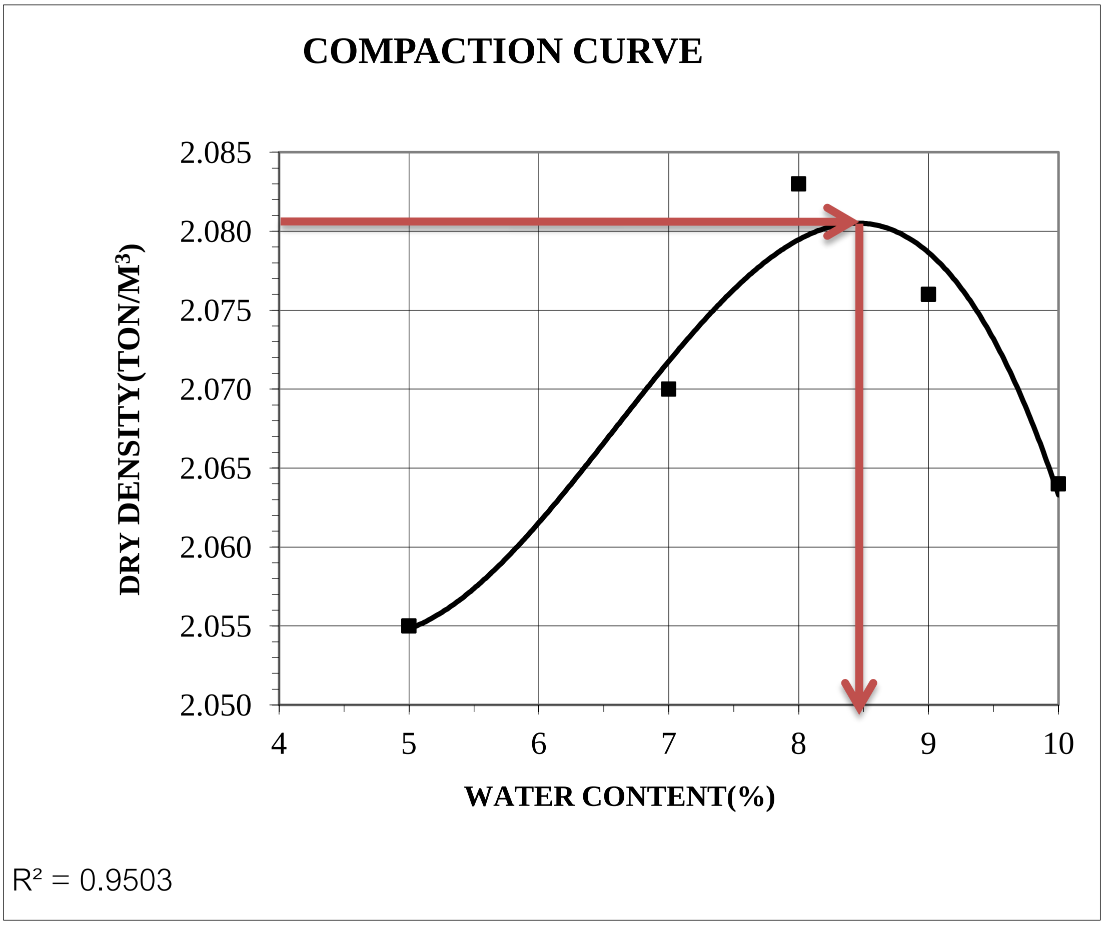
SOIL MECHANIC LABORATORY CURTIN UNIVERSITY

Project:	Foamed Bitumen Stabilisation	Source of Soil:	Perth Local Quarry
Location:	Curtin Mechanics Lab	tested Date:	31/3/2010
Soil Description:	50% CRB , 50% CLS	Test By:	Ryan H/ Richard C
Compaction Method:	Standard	Mold Dimension:	Dia105.0xH115.5mm
Hammer weight	4.9 Kg	Droped Height	450 mm
No. of Layer	5	No. of Blow	25 blows/layer

RESULT OF SOIL COMPACTION TEST

Test Number			1	2	3	4	5
Dry Density,	$\rho_{(dry)}$	ton/m^3	2.055	2.070	2.083	2.076	2.064
Water Content,	W	%	5.00	7.00	8.00	9.00	10.00

OMC = 8.48 %
MDD = 2.081 Ton/M^3



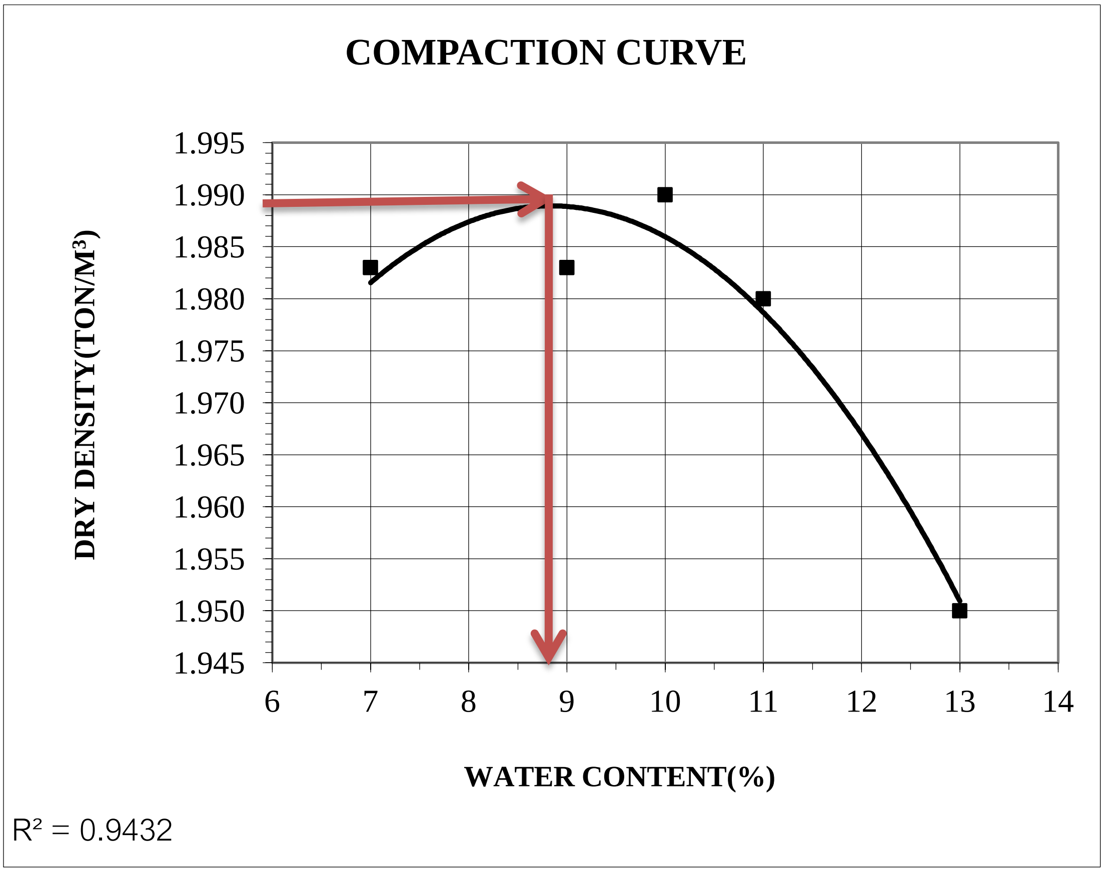
SOIL MECHANIC LABORATORY CURTIN UNIVERSITY

Project:	Foamed Bitumen Stabilisation	Source of Soil:	Perth Local Quarry
Location:	Curtin Mechanics Lab	tested Date:	31/3/2010
Soil Description:	25% CRB , 75% CLS	Test By:	Ryan H/ Richard C
Compaction Method:	Standard	Mold Dimension:	Dia105.0xH115.5mm
Hammer weight	4.9 Kg	Droped Height	450 mm
No. of Layer	5	No. of Blow	25 blows/layer

RESULT OF SOIL COMPACTION TEST

Test Number	1	2	3	4	5
Dry Density, $\rho_{(dry)}$ ton/m^3	1.983	1.983	1.990	1.980	1.950
Water Content . w %	7.00	9.00	10.00	11.00	13.00

OMC = 8.85 %
MDD = 1.989 Ton/M^3



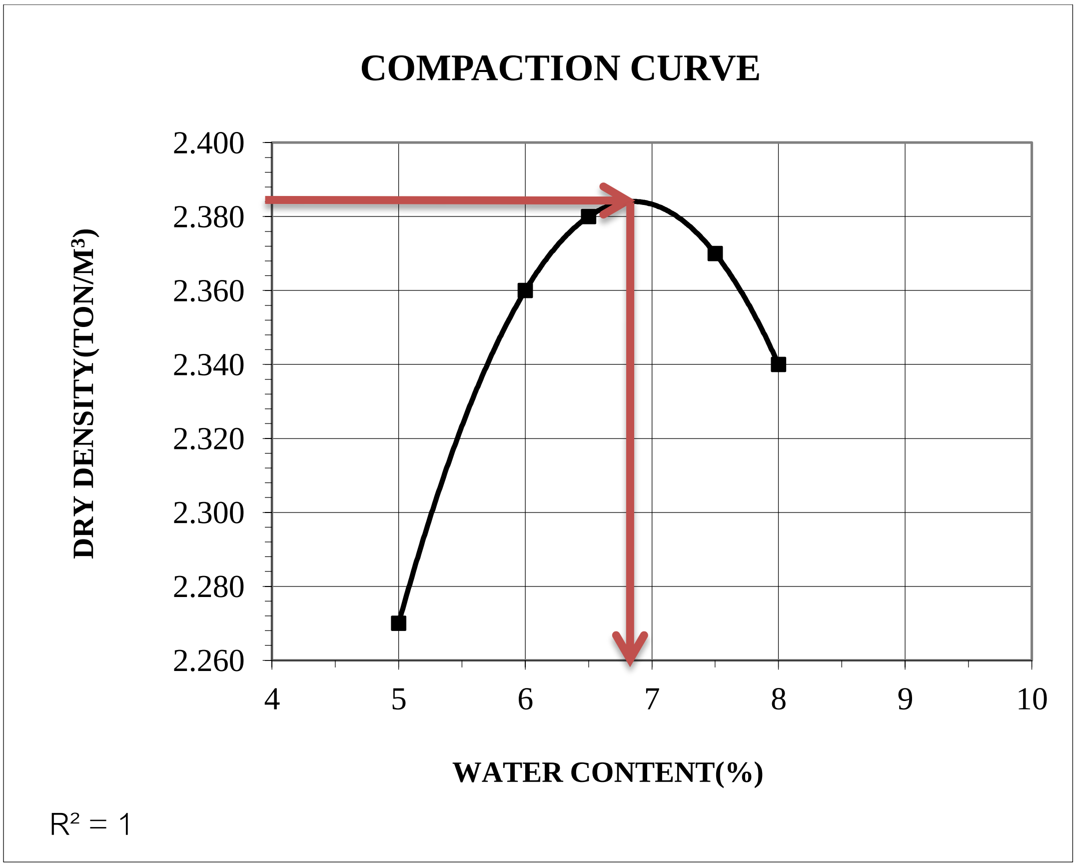
SOIL MECHANIC LABORATORY CURTIN UNIVERSITY

Project:	Foamed Bitumen Stabilisation	Source of Soil:	Perth Local Quarry
Location:	Curtin Mechanics Lab	tested Date:	31/3/2010
Soil Description:	100% CRB , 0% CLS	Test By:	Ryan H/ Richard C
Compaction Method:	Standard	Mold Dimension:	Dia105.0xH115.5mm
Hammer weight	4.9 Kg	Droped Height	450 mm
No. of Layer	5	No. of Blow	25 blows/layer

RESULT OF SOIL COMPACTION TEST

Test Number	1	2	3	4	5
Dry Density, $\rho_{(dry)}$ ton/m^3	2.270	2.360	2.380	2.370	2.340
Water Content . w %	5.00	6.00	6.50	7.50	8.00

OMC = 6.90 %
MDD = 2.385 Ton/M^3



APPENDIX D: ITS TEST RESULTS OF UNSOAKED SAMPLES

ITS Testing Results of Unsoaked samples for different aggregate compostions at various foamed bitumen contents

			OMC (%)	MDD (g/cm3)	Dry Density			Max-load(KN)				Tensile Strength (KPa)			
FB (%)	CRB (%)	CLS (%)			S 1	S 2	S 3	S 1	S2	S 3	Mean	S 1	S 2	S 3	Mean
3	100	0	6.79	2.221	2.140	2.135	2.164	0.44	0.50	0.57	0.50	36.51	39.16	45.90	40.52
3	75	25	6.41	2.178	2.122	2.090	2.101	0.48	0.50	0.47	0.48	40.38	39.18	37.75	39.10
3	50	50	7.8	2.092	1.969	1.968	1.974	0.36	0.25	0.24	0.28	29.47	19.93	19.68	23.03
3	25	75	9.1	1.902	1.815	1.816	1.781	0.25	0.27	0.22	0.25	20.96	21.07	17.16	19.73
4	100	0	6.82	2.168	2.094	2.036	2.050	0.35	0.43	0.55	0.44	28.06	32.66	43.20	34.64
4	75	25	7.17	2.062	2.021	2.025	2.058	0.31	0.36	0.45	0.37	24.87	28.60	37.32	30.26
4	50	50	8.54	2.021	1.900	1.930	1.938	0.50	0.49	0.43	0.47	40.85	38.13	34.22	37.73
4	25	75	9.41	1.922	1.824	1.857	1.839	0.18	0.20	0.18	0.19	15.01	16.22	14.38	15.20
5	100	0	6.81	2.163	2.080	2.112	2.084	0.23	0.40	0.26	0.30	19.21	32.07	20.99	24.09
5	75	25	5.92	2.034	1.978	1.992	1.975	0.26	0.29	0.40	0.32	20.85	23.45	31.96	25.42
5	50	50	7.91	1.984	1.943	1.968	1.962	0.37	0.46	0.43	0.42	29.72	37.55	35.14	34.14
5	25	75	8.21	1.899	1.798	1.816	1.812	0.28	0.18	0.18	0.21	23.36	15.06	15.00	17.81

Note:
FB=Foamed Bitumen
CRB=Crushed Rockbase
CLS=Crushed Limestone
S1=Sample 1

APPENDIX E: ITS TEST RESULTS OF SOAKED SAMPLES

ITS Testing Results of Soaked samples for different aggregate compostions at various foamed bitumen contents

			OMC (%)	MDD (g/cm3)	Dry Density			Max-load(KN)				Tensile Strength (KPa)			
FB (%)	CRB (%)	CLS (%)			S 1	S 2	S 3	S 1	S2	S 3	Mean	S 1	S 2	S 3	Mean
3	100	0	6.79	2.221	2.16	2.159	2.15	0.32	0.47	0.33	0.37	26.07	37.91	26.83	30.27
3	75	25	6.41	2.178	2.143	2.138	2.155	0.49	0.4	0.43	0.44	40.2	32.81	35.93	36.32
3	50	50	7.8	2.092	1.971	1.981	1.95	0.31	0.3	0.29	0.3	25.07	24.64	22.81	24.18
3	25	75	9.1	1.902	-	1.800	1.816	-	0.18	0.15	0.11	-	14.21	11.95	13.08
4	100	0	6.82	2.168	2.093	2.086	2.071	0.5	0.48	0.35	0.44	39.86	37.87	27.32	35.02
4	75	25	7.17	2.062	1.998	1.998	2.04	0.28	0.21	0.28	0.26	22.19	17.06	22.5	20.58
4	50	50	8.54	2.021	1.961	1.931	1.909	0.43	0.25	0.33	0.34	34.33	20.22	25.93	26.83
4	25	75	9.41	1.922	1.84	1.827	1.812	0.18	0.23	0.18	0.2	14.37	18.08	14.27	15.57
5	100	0	6.81	2.163	2.062	2.079	2.037	0.32	0.26	0.21	0.26	25.57	21.13	16.83	21.18
5	75	25	5.92	2.034	2.01	1.989	-	0.33	0.2	-	0.18	26.53	15.68	-	21.11
5	50	50	7.91	1.984	1.946	1.928	1.947	0.31	0.23	0.25	0.26	25.09	18.29	19.77	21.05
5	25	75	8.21	1.899	1.86	1.837	1.802	0.22	0.16	0.22	0.2	17.98	12.86	17.39	16.08

Note:
FB=Foamed Bitumen
CRB=Crushed Rockbase
CLS=Crushed Limestone
S1=Sample 1

APPENDIX F: UCS TEST RESULTS ON DIFFERENT AGGREGATE COMPOSTIONS

UCS Testing Results for different aggregate compostions at various foamed bitumen contents

			OMC (%)	MDD (g/cm3)	dry density (g/cm3)		deviation (%)		Deviator Stress(KPa)		
FB (%)	CRB (%)	CLS (%)			Sample 1	Sample 2	Sample 1	Sample 2	Sample 1	Sample 2	Mean
3	100	0	6.79	2.221	2.186	2.172	98.42	97.81	204	199	201.5
3	75	25	6.41	2.178	2.086	2.082	95.78	95.61	267	242	254.5
3	50	50	7.8	2.092	2.028	2.051	96.93	98.03	124	119	121.5
3	25	75	9.1	1.902					147	133	140
4	100	0	6.82	2.168	1.936	2.124	89.28	97.97	114	156	135
4	75	25	7.17	2.062	2.075	2.087	100.62	101.20	136	143	139.5
4	50	50	8.54	2.021	1.982	1.979	98.06	97.91	71	113	92
4	25	75	9.41	1.922	1.891	1.903	98.38	99.02	99	98	98.5
5	100	0	6.81	2.163	2.106	2.122	97.38	98.12	135	146	140.5
5	75	25	5.92	2.034	2.019	2.046	99.28	100.61	169	147	158
5	50	50	7.91	1.984	1.996	1.986	100.59	100.11	108	103	105.5
5	25	75	8.21	1.899	1.881	1.918	99.03	100.99	116	123	119.5

Note:
FB=Foamed Bitumen
CRB=Crushed Rockbase
CLS=Crushed Limestone