FOAMED BITUMEN STABILISED SANDSTONE AGGREGATES

By

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Thesis submitted to the University of Nottingham for the degree of Doctor of Philosophy

February 2015

ABSTRACT

Roads form a key element for the expansion of economy and development of a country. As with most countries, Brunei Darussalam has been facing a rapid development to meet the economic growth that requires an efficient road network. Therefore, the scarcity of conventional road aggregates in Brunei Darussalam means that the country has a strong dependence on imported aggregates from overseas to construct quality roads. Further restrictions on local road specifications make it almost impossible to include low quality granular materials. The study reported in this thesis was undertaken on the basis that the dependence on overseas resources is not a viable long-term solution. The research task has been, therefore, to ascertain the quality of local sandstones for road construction and then to propose means to upgrade their performance quality for optimum utilisation in cost effective applications. This study focused on the road base layer since that is where most aggregate is used.

The approach used for this study was to identify the common rock in Brunei Darussalam and review the candidate treatment methods. A weighted matrix for these candidate treatment methods was constructed to determine the overall ranking with selected key criteria on the basis of the local climatic condition, construction preferences and traditions. From the reviews, Foamed bitumen was selected as a feasible treatment method that can improve the sandstone characteristics under local conditions.

Three curing conditions were adopted in this study, simulating extreme field conditions in Brunei Darussalam, to characterise the mechanical properties of foamed bitumen stabilised sandstone mixtures, termed 'foam mix'. The following tests were conducted:

- The response of stiffness modulus behaviour in the foam mix produced at different levels of mixing moisture content and

cement content under dry and wet conditions was measured to study the mixing moisture content (MMC) in foam mix design.

- A humid curing study was performed to indicate the short term stiffness of foam mixes in order to aid in the prediction of the delay necessary before a road comprising these foam mixes could be opened to traffic, and to determine how curing time and moisture content affect the development of stiffness modulus with and without cement.
- A preliminary investigation was carried out into the potential of coir fibres as a reinforcement agent in the foam mix, measuring its effect on stiffness modulus, tensile strength and permanent deformation.
- Being sensitive to moisture, the climatic durability of foam mix was further assessed by studying the effect of dry/wet cycles on the stiffness modulus incorporating other additives such as hydrated lime and pre-blended bitumen with wet fix.
- Microscopic analysis has been undertaken as a guide to characterise the microstructure of the foam mix incorporating additives such as cement and coir fibres in order to support the laboratory findings.

The laboratory results confirmed that the stiffness behaviour of the foam mix could be influenced by the amount of MMC, cement content and humidity of the environment. It was found that the foam mix with 1% cement (by mass of dried aggregates) at MMC, 70% of OMC, produced a durable mixture with a high stiffness modulus value in both dry and wet conditions as well as when subjected to the effect of alternate dry and wet cycles. The investigation on the potential of coir fibre to reinforce the foam mix indicated that the fibre did help to prevent large cracks in the foam mixes but unfortunately the reinforced foam mixes were easily damaged under a wet environment.

ACKNOWLEDGEMENT

Foremost, I would to express my deepest appreciation to my supervisor, Professor Andrew Dawson for his generous contribution of time, immense knowledge and guidance to keep me going for the entire duration of my study.

My deepest thank you to Dr Alessandra Carrera whose advice and insights in the early year of the study has positively contributed to the research and she never hesitated to review my work from time to time when needed.

I would like to express my sincere gratitude to Dr Hussein Khalid and Dr Nick Thom for allowing me to exhibit and communicate the significance of my research findings in more effective way.

I extend my gratitude to the two former Directors of Roads, Ismail Jaafar, who believes so much in me and, Amer Hishamuddin Zakaria, for putting their trust in me to pursue my study.

Sponsorship by the Government of Brunei Darussalam is gratefully acknowledged and my appreciation goes to the Brunei High Commission (Recruitment and Training Section) who were responsible to provide the necessities during my stay in the United Kingdom.

Special thank you and deepest appreciation go to the current Director of Roads, Kok On Yee, to whom I am indebted for his continuous motivational words that fuel my determination and perseverance through the difficult times.

This thesis work was also made possible by the great assistance of the laboratory technicians, Richard Blakemore, Martyn Barrett, Richard

Meehan, Nancy Hodge, Lawrence Pont. Jon Watson and Michael Winfield for facilitating and supporting my laboratory works as needed.

I am very thankful for the cooperation and assistance rendered by Nigel Neate, for helping on the SEM and the Rock Mechanic Laboratory technicians for their kind help.

I gratefully acknowledged the Craig Yr Hesg Quarry for supplying the sandstone aggregates and Total Bitumen Company for its kind support to provide Bitumen throughout the research study period.

My eternal love goes to my parents for their devotions in providing good education since the early age; my father who has been the greatest role in shaping my educational path; my mother who has been unconditionally patient and my greatest soul through my years of hardship.

Special thank you to all of my family members, my dear sisters and brother for taking care of mum and dad whilst I was away; my dearest Uncle and family for always being there every now and then; my dear late brother-in-law who was a very attentive brother; and the others for their supports particularly through the difficult times.

My thank you goes to all my friends who have given the invaluable support and advice through the journey.

DECLARATION

The research reported in this thesis was conducted at the Nottingham Transportation Engineering Centre (NTEC), School of Civil Engineering, University of Nottingham. I declare that the research study is my own and has not been submitted for a degree in another university.

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February 2015

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CHAPTER 1 INTRODUCTION

1.1 BACKGROUND

The Brunei Government has been actively promoting the development of various target sectors including the roads sector through its five-year National Development Plans. The plans are formulated to enhance the provision of infrastructural facilities for the stimulation of economic growth and adequate public amenities to ensure the quality life of the people. The road network plays the main role in the overall growth and development of the country with a small population of 398,000 people.

As shown in Figure 1.1, there has been a vast development of road network in Brunei-Muara district, the smallest of four districts which boasts the capital and the most populated area. As with most capital cities, Brunei Muara district is the centre of the government activities, which are interspersed amongst the growing numbers of housing, commercial and industrial areas. The easternmost part of the country is the second largest district but the least populated. It is largely undeveloped as it includes the main national forest reserve. It can be observed from Figure 1.1 that the development of the road network is mainly at the northernmost part where its small town, Bangar is located.

Despite the small area of 5,765 square km, Brunei Darussalam also serves a major highway that is part of the Trans Borneo highway and numbered as AH150 in the Association of South East Asian Nation's (ASEAN) highway network. It forms part of the integral part of the Pan Borneo highway that provides a vital link between Brunei Darussalam and neighbouring countries such as Sarawak, Sabah and Kalimantan in Borneo Island. It runs along the coastal line about 1.712 kilometres and links Muara, the port entry point at one end of the country to the oil-producing district, Belait, at the other end.

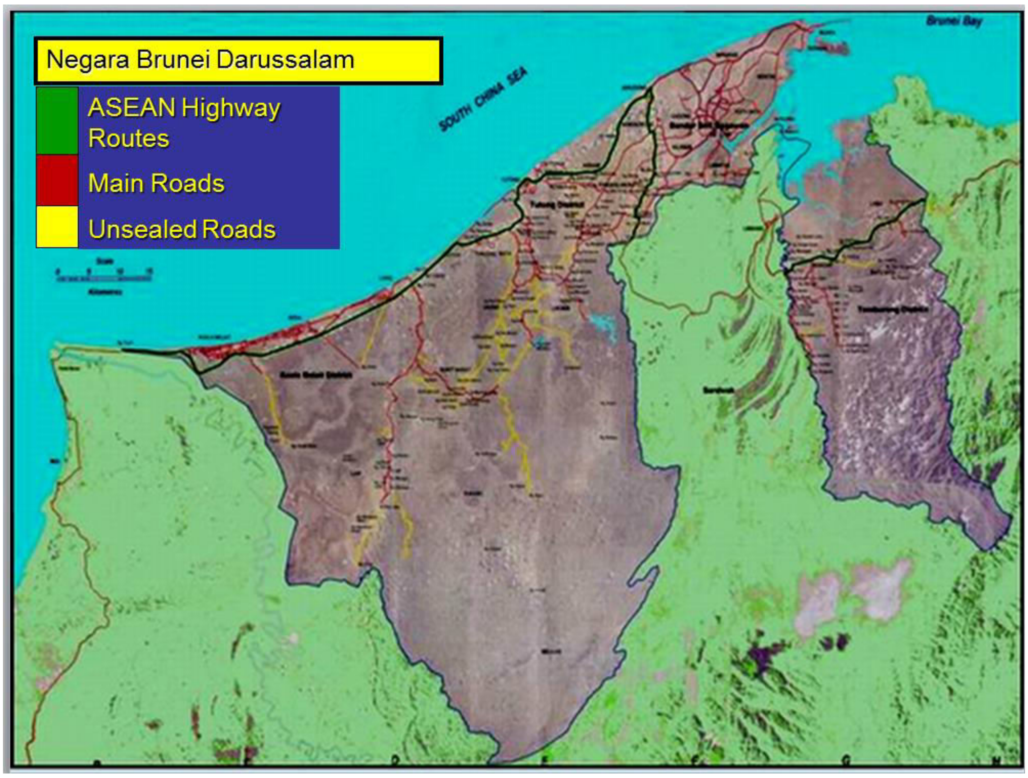


Figure 1.1 Map of Brunei Darussalam showing the road network across the country (Brunei Shell Petroleum, 1997)

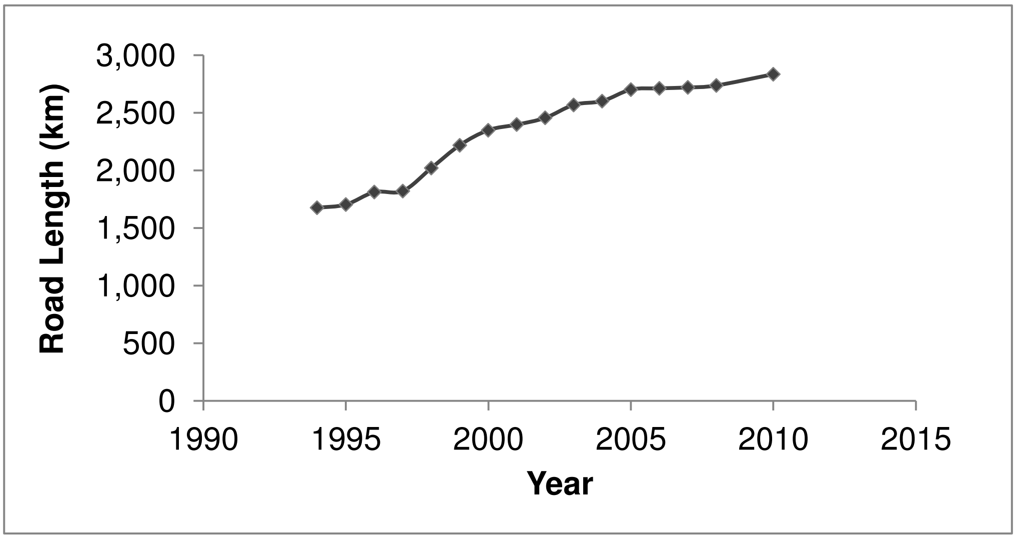


Figure 1.2 Road network in Brunei Darussalam over 20 years

Therefore the result is a large volume of traffic and Figure 1.2 further illustrates the rapid growing of the road network over 20 years to the total of 2836km.

1.2 PROBLEM STATEMENT

The growing demand for quality aggregates for the road construction industry has been noticed for many years with the rapid development of infrastructure all over the world. Brunei Darussalam is not left out and the demand imposes problems of availability and the requirements as to specification in order to achieve better road performance.

Aggregates are the main materials in a road pavement structure particularly the conventional flexible pavement in its unbound road base. They originate geologically from natural finite resources therefore, they are eventually becoming scarce. The extraction activities would be becoming less easy and more environmental issues would be raised.

In some developed countries, the attention has been given to the utilisation of secondary aggregates to overcome the shortage problems. Secondary aggregate includes waste product materials such as fly ash, slag and recycled pavement materials. The utilisation of these secondary aggregates can be due to the large production of waste product materials.

Unfortunately Brunei Darussalam has little or no significant amounts of waste products that can be utilised as secondary aggregates. However, the country still has its own natural aggregate resources, derived from tertiary sedimentary rocks and quaternary consolidated sediments (TCP, 1986), but they are somehow limited and usually rejected for use in road constructions. In certain cases, they are allowed for use in the sub-base layer, in compliance to the specification, for low traffic volume roads and subgrade protection during construction activities. Past failures were often induced due to their friable nature and inability to form a strong interlocked skeleton.

However, the information on failures has not been properly documented and the failures are rather confined at the country levels. Such failures however are of high research value in terms of carrying out scientific investigation to identify cause of failures as a guide to future use of the local aggregates. Instead a strict adherence to technical specifications, testing standards and quality control procedures are imposed which narrow the choice to only one quality type of aggregate for each unbound road base and sub base layer. Furthermore the aggregates must comply with the physical properties stated in the specifications and be graded within one designated gradation envelope. More details on the road construction practice can be found in Chapter 2. This specific quality of aggregates could not be found locally thus, the country has since developed a strong reliance on the imported quality aggregates to build roads.

It has been unfortunate that little or no research has been done on the origin, nature, distribution as well as the engineering properties of local aggregates since Tate (1968) therefore, the quality is somewhat uncertain. The poor selection of the local aggregates and lack of engineering classification system can be the cause of failures in their field performances.

Therefore, it should be realised that construction quality aggregates are scarce resources but dependence on overseas resources is not the long term solution as it is believed that there are hard rock sources that may be suitable which are not yet exploited. The permanent situation therefore should be for these unexploited sources to be tapped into or for engineers to use innovative methods to better utilise the indigenous resources.

Dawson (2003) stated that 'to ensure a successfully performing base, the real solution is not to adopt ever-tightening restrictions but to permit a wider range of materials and then to carefully characterise the available material so that it can be used optimally in the pavement'.

Application of successful low quality aggregates in the roads based on research studies and practical experiences elsewhere has come with mixed results of successes and failures. The success and failure results depend on the applications and conditions in the different geographical regions in the world. Therefore, it is important to properly evaluate them with suitable additives to make them as usable pavement materials. This technique is commonly called as stabilisation.

However, the introduction of additives for pavement stabilisation in Brunei Darussalam is often accompanied by overseas technical guidelines in which the successful track records are based on the materials and performances in the supplier's country of origin. This has led to application in Brunei Darussalam without properly understanding its behaviour resulting difficulty in the interpretation of its performance as road pavement material. Engineers have no choice but to rely on these overseas specifications that may not be able to deliver the required pavement performance. In effect, their use particularly the new ones have had been merely on a trial basis. Yet the information on neither the successes nor failures of their performance is properly compiled and documented for further evaluation. And unfortunately the stabilised materials are confined to the aggregates of the same specified quality for the unbound aggregate road base. Therefore, regardless of stabilised or non-stabilised pavement, the road aggregates are primarily from imported sources.

1.3 NEED FOR RESEARCH

There has been no appropriate modification to current standards and technical specification to include the local material resources that suit the local environmental conditions. The selection of locally available materials to build strong and durable pavements has many potential advantages. The primary advantage is the lower cost, because hauling costs for imported materials are often the highest cost component of the material. However, the trade-off in cost savings can be poor structural performance. Therefore, it is important to study and understand the engineering behaviour of these stabilised materials by proper evaluation and taking into account the specific local environment and climatic conditions. The outcomes of the study will ensure the development of effective engineering specifications and standards for utilisation that can classify low quality aggregates as road construction material.

1.4 AIM AND OBJECTIVES

The main aim of this research is, therefore, to optimise the utilisation of local aggregate (i.e sandstone) by improving its properties with a selected additive, foamed bitumen, such that the pavement performance is maintained or improved. Deliverables from this study will allow opportunity to revise material specification and develop guidelines for pavement stabilisation. In order to achieve the aim, the following objectives have to be met:

- Review of geological features and identification of available aggregate resources
- Evaluation and assessment of potential candidate stabilisation methods or agents to select the most suitable as means of improving the characteristics of sandstone aggregates

- Carry out the study the engineering properties of the stabilised mixtures, foamed bitumen mixtures, to indicate any weaknesses
- Investigate ways to improve the behaviour of foamed bitumen stabilised mixtures by incorporating additives.
- Investigate the effect of the curing and moisture susceptibility on the modified improved mixtures.
- Pavement analytical study to make appropriate recommendation on design thickness of pavement for foam mix materials.

1.5 RESEARCH METHODOLOGY

The research methodology employed is shown in a flow diagram as illustrated in Figure 1.3 to achieve the objectives and consisted of two study phases:

Study Phase 1

The evaluation study in phase 1 was carried out to clearly define the scale and scope of the problem being investigated as well as establish the objectives of the study. The two main tasks are as described below:

The first task was to obtain a basic understanding of the attributes of the pavement engineering practice in Brunei Darussalam. This includes the reviews of the climatic conditions and the geological features to understand the origin of bedrock to aid in the location of aggregate resources, either current or unexploited reserves. Another relevant review covers the common practices in road pavement design and construction as well as the limitations.

The second task was to identify the candidate treatment methods that can potentially improve the sandstone aggregates for their application in Brunei Darussalam by conducting literature surveys. These candidate treatment methods were then evaluated and assessed by constructing a weighted matrix with selected key criteria. This was important to identify the best suitable treatment method that fulfils the needs to improve sandstone aggregates. Each criterion was weighted based on the level of significance of its contribution to the optimisation of using sandstone as road base material and a scoring factor was also assigned to measure the positive and negative impact levels of each criterion on each method against the others. The impact could be influenced by the regional condition, local availability of the materials, local preferences in the structural types, climatic factors, construction standards and traditions. The scores for each candidate treatment were summed up and then ranked accordingly to identify the best suitable method, which in this study was the foamed bitumen treatment. Thus this ranking evaluation task was mainly to determine the laboratory approach of the succeeding phases.

Study Phase 2

The second phase of the study began with an extended literature review on foamed bitumen including the design and application of foamed bitumen modified mixtures in different geographical regions in the world. A set of experimental works on the foamed bitumen stabilised sandstone aggregates mixture, which is now refered to as 'foam mix', were undertaken. The mix was designed by modifying the available UK sandstone aggregates of similar or poor quality to closely fit the typical gradation of Brunei Darussalam's local sandstone aggregates to study the mechanical properties such as stiffness modulus and resistance to permanent deformation or rutting. The durability test consisted of moisture susceptibility assessment. The test and curing conditions of the experiments had to be closely matched to the climatic conditions that can be experienced in Brunei Darussalam in terms of temperature, humidity and moisture factors. Two additional additives were introduced in this study for more extensive comparison purposes. These two additives were anti stripping agents in the form of liquid and powder, Wet Fix and hydrated lime respectively.

The laboratory findings would lead to the selection of a suitable foam mix either without or with additives as a modified road base material such that the pavement performance has to be equivalent or better than with conventional road base materials.

1.6 RESEARCH BENEFITS

The introduction of locally available materials in road construction has many potential benefits, which lead to lower cost when they are utilised in an optimal manner by adapting innovative and effective applications to build a durable pavement. The potential benefits can be expected as follows:

- Consequential stretching of funds for road development
- Fees paid to the local contractors to supply material to be used in the construction or maintenance of the pavement assists the local economy
- Reduction in the transport costs incurred in material cartage, in the form of fuel, vehicle wear and vehicle maintenance and depreciation, due to the ability to use wider range of materials close to their sources.
- Transportation of local materials involves less haulage therefore preserving the new or existing pavement from high axle loads of haulage equipment.
- Savings made by use of local materials can permit a more extensive rural road network including access roads to residential properties.
- More sustainable solution from global environmental point of view

1.7 THESIS LAYOUT

The thesis presents the methodology, results, analysis and discussion obtained from literature reviews and an extensive laboratory investigation. The thesis is divided into fifteen chapters, following the methodology described above. An overview of the execution of the tasks and corresponding thesis chapters is illustrated in the flowchart in Figure 1.3 and a brief description of the contents of each chapter is presented below:

Chapter 2 concentrates on review of relevant literature that focusses on the geological features to identify common rock and identify the current or unexploited sources of aggregates and current practices in road pavement construction.

The selection of candidate stabilisation methods to enhance the properties of the local aggregates i.e sandstones is covered in Chapter 3. The evaluation of these candidate stabilisation methods or agents includes the construction of a weighted matrix and scoring table to aid in the decision-making process.

In Chapter 4, an extended literature review on foamed bitumen mixtures and a review of their applications utilising a wide range of granular materials in different countries and the mechanical properties including the moisture susceptibility are discussed.

The analysis of materials used in this study are discussed in Chapter 5 consisting of aggregates, bitumen, additives such as cement, coir fibres and anti-stripping agents such as hydrated lime and wet-fix. It was carried out to select the available materials in the U.K that closely represent the common Brunei Darussalam pavement materials with respect to aggregate type and gradation, rheological properties of bitumen and availability of the materials.

Chapter 6 reviews the experimental techniques employed in the study and the curing strategies adopted by numbers of researchers are presented. Therefore the conditions had to reflect closely to the local environment. Three curing conditions considered in this study are discussed including a brief description of the humid curing tank developed for this study.

In Chapter 7, the descriptions on the production process of foamed bitumen using two different plants, WLB10 and WLB10s, and mixers, Hobart and twin shaft types are presented including sample preparation and mix design procedures. Chapter 8 presents an experimental testing on foam mix treated with or without cement to evaluate its stiffness behaviour or response at selected mixing moisture content (MMC) levels. Tests results from various foam mixtures are presented to assess the effect of foamed bitumen content and cement on foam mix at selected MMC levels.

Chapter 9 presents the preliminary experimental investigation of coir fibre reinforced foam mix to evaluate the mechanical properties in terms of stiffness modulus and resistance to permanent deformation.

As humidity is one of the significant climatic conditions in Brunei Darussalam therefore, Chapter 10 presents a study to evaluate the development of stiffness modulus of foam mix under humid curing condition, short and long term. Two foam mixes were selected for this laboratory study evaluation.

In Chapter 11, the foam mix is further evaluated in terms of its climatic durability by investigating the effect of dry and wet curing cycles on its stiffness modulus. Selected additives were added in the foam mix so that their effects could be seen in the experimental results.

Chapter 12 presents the rutting performances of selected foam mixes to investigate their resistance using the Wheel Tracking test (WTT).

In this study, the microstructure of foam mix treated with cement and coir fibres was studied to aid in understanding the macro behaviour. The micrographs analysis was described in Chapter 13.

A pavement analytical design study using multi-layer elastic analysis is presented in Chapter 14, to investigate the relative influence of foam mix in Brunei Darussalam's typical pavement structure and predict the pavement life in terms of numbers of traffic ESALs.

Chapter 15 covers the conclusions, implications for field conditions and recommendations for future research.

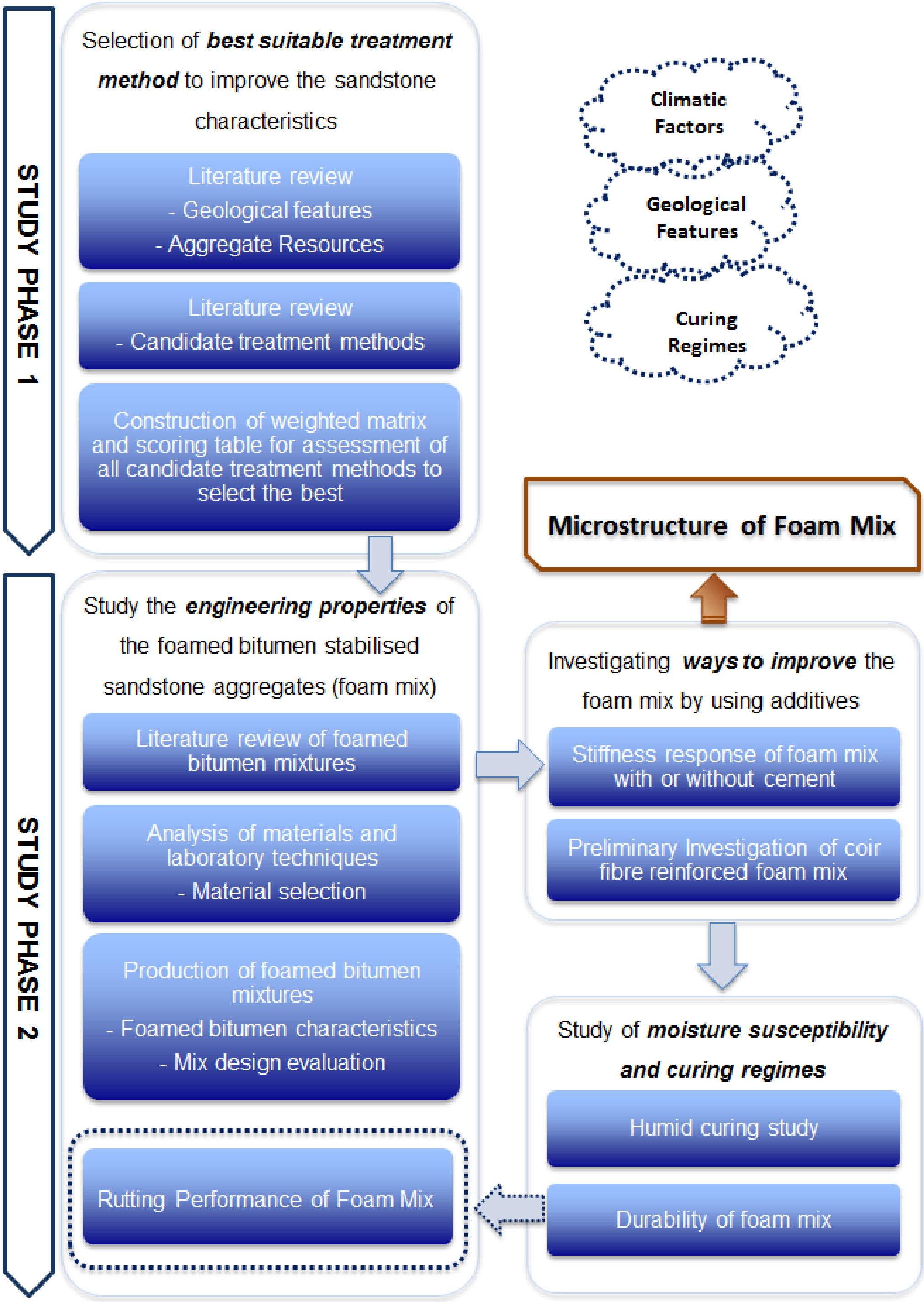


Figure 1.3 Flowchart of the research methodology

CHAPTER 2 REVIEW OF RELEVANT LITERATURE

2.1 INTRODUCTION

For road pavement to be economic, the engineering studies and designs carried out before the construction of the road pavement must take into account the environmental factors that are responsible for pavement deterioration. The susceptibility of pavement materials to environmental conditions must therefore, be properly evaluated to ascertain its reliability before and during its service life. This chapter discusses some important factors that are unique to the typical Brunei Darussalam's condition such as follows:

- Climatic conditions
- Geological features
- Pavement construction practise

2.2 CLIMATIC CONDITIONS

Climatic conditions particularly temperature and rainfall are a basis for large variations in the prevailing conditions of road pavement in different geographical regions in the world. Therefore, it is important to address them in this study.

Brunei Darussalam sits about 443 kilometres north of the equator, within the tropical equatorial climatic region, hot and wet with high humidity. The climatic pattern is generally hot days followed by cool nights all year round. The average air temperature is notable for its uniformity around 30°C all the year around but it may reach 40°C in a hot mid-afternoon in a typical day. The temperature may fall to about 21°C to 24°C at night. The region is also exposed to extensive sunshine in which the annual average period of sunshine per day is about 7 hours (Malik and Roslan, 1996). It was also stated that the months of March and April receive sunshine for about 8 hours.

Figures 2.1, 2.2, 2.3 and 2.4 are taken from data supplied by the Brunei Meteorological Services (BMS, 2013). Figure 2.1 and 2.2 give a general guide to the mean yearly and monthly rainfall statistics. The average recorded annual rainfall for the period from the year 1966 to 2013 was 2951.2mm. The highest rainfall recorded within this period was 4443.1mm in 2010. However, in January 2009, as shown in Figure 2.3, there was unusual and prolonged very wet weather with a significant recorded rainfall of 977.0mm, which badly affected the country causing flooding and landslides.

Generally Brunei Darussalam has rainfall throughout the year in which the recorded mean monthly rainfall was above 200mm. The weather is particularly wet from October to January, during monsoon season, with December being the wettest month. Heavy rainfalls could also occur in the months of May to September. The lowest recorded rainfall is in February to March.

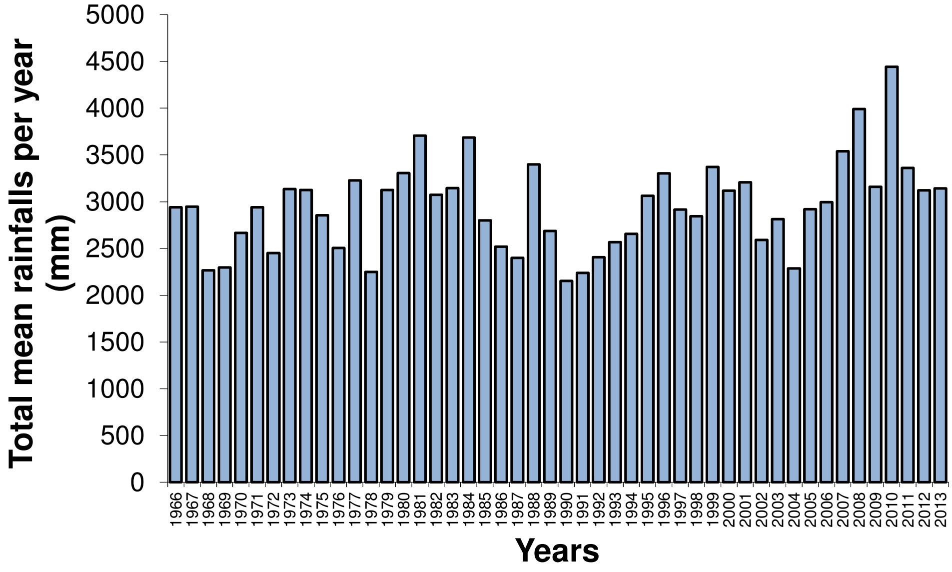


Figure 2.1 Total mean annual rainfall in Brunei Darussalam 1996-2013 (BMS, 2013)

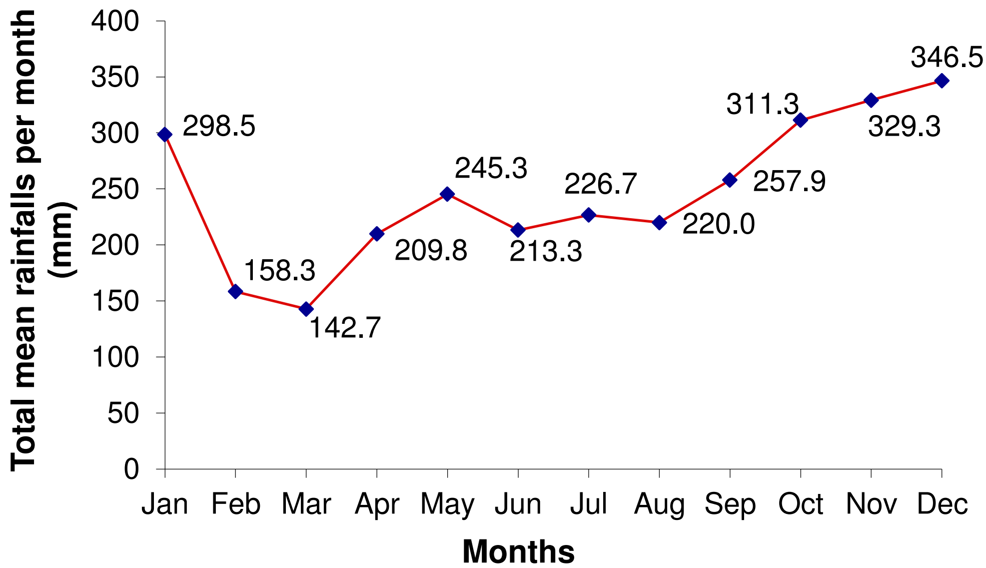
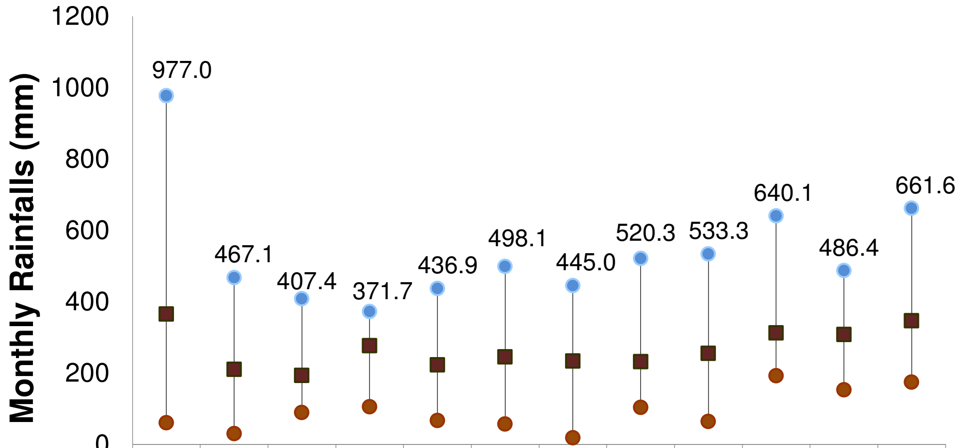


Figure 2.2 Mean monthly total rainfalls in Brunei Darussalam from a period of 1966 to 2013 (BMS, 2014)

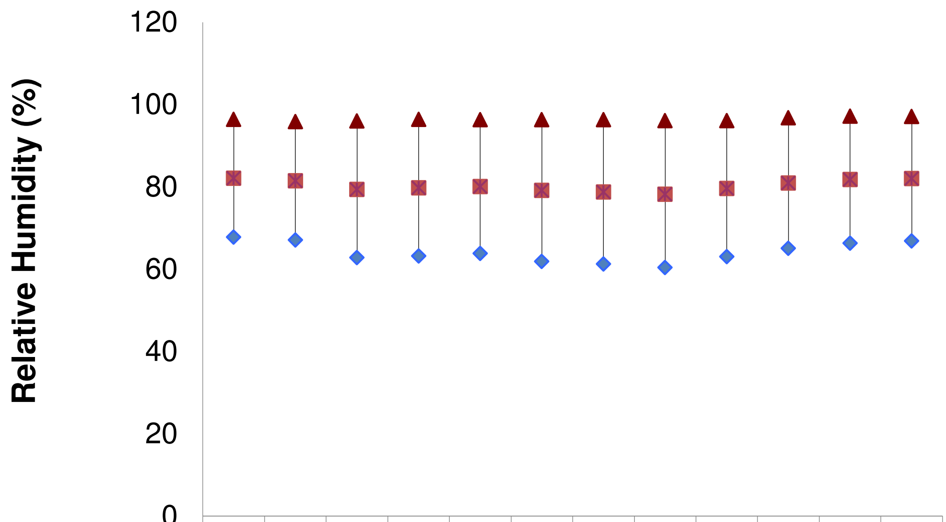


	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Minimum	60.0	29.5	88.5	104.5	66.2	56.1	17.8	102.9	63.5	192.1	152.1	173.7
Mean	365.0	209.5	193.1	275.7	222.2	245.1	232.9	231.6	255.0	312.0	307.4	346.2
Maximum	977.0	467.1	407.4	371.7	436.9	498.1	445.0	520.3	533.3	640.1	486.4	661.6

Months

Figure 2.3 Rainfall data from the period of 1996 to 2009 (BMS, 2009)

Another significant climatic condition in Brunei Darussalam is its humid environment. Humidity is defined as the amount of water vapour already in the air. Relative humidity is a measurement of the actual water vapour in the atmosphere expressed as the percentage of the amount that could exist under the same given temperature (Anderson, 1936). The humidity level is very high all year round due to the country's proximity to the sea and equator which is also experienced by most countries in South East Asia. The average daily value of relative humidity has a diurnal range of above 90% to around 60% as shown in Figure 2.4.



	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
◆ Average Minimum	67.73	67.07	62.82	63.18	63.79	61.89	61.21	60.36	63.04	65.07	66.29	66.82
■ Mean	82.07	81.47	79.39	79.77	80.06	79.11	78.76	78.21	79.57	80.92	81.73	81.96
▲ Average Maximum	96.40	95.87	95.97	96.37	96.33	96.33	96.30	96.07	96.10	96.76	97.17	97.10

Figure 2.4 Relative humidity in Brunei Darussalam from the period of 1979 to 2009 (BMS, 2009)

2.3 GEOLOGICAL FEATURES

The lack of adequate knowledge about the origin and characteristics of the pavement materials particularly aggregates, which are the interest of this study, can be the main cause of the poor selection and utilisation of local aggregates. Therefore, the geological features of Brunei Darussalam are reviewed in this section to understand the structure and history of origin of the bedrock. These factors allow identification of the rock for the potential source of road aggregates, either exploited or unexploited reserves. The local preferences on the pavement structural type would normally be based on the types of these road aggregates.

2.3.1 Geology

Brunei Darussalam forms part of the centre of the Northwest Borneo Geosynclines and lies within one of the most complicated areas of plate activity in the world (Fitch, 1960). A simplified geological map can be seen in Figure 2.5. It lies over the bedrock of Tertiary sedimentary rocks (Wilford, 1961). Sedimentary rocks mainly form from the breakdown products of older rocks that are subsequently transported normally by water but also air or glacial action to deposition sites such as on foothills, river valley floodplains and beaches as sediments (Smith et al, 2001) and later lithification occurs.

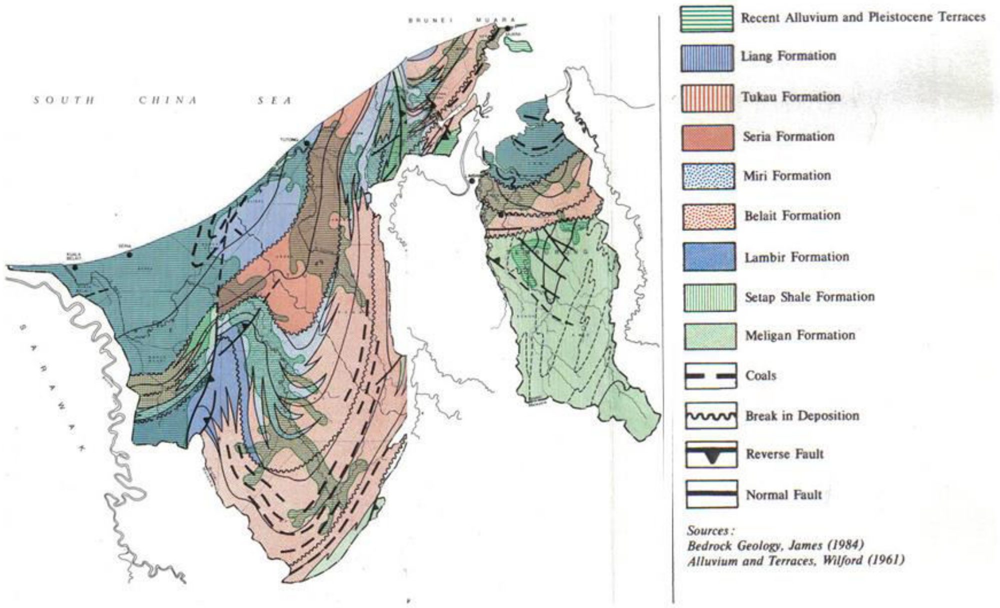


Figure 2 5 Geological map of Brunei Darussalam (TCP, 1986)

Table 2.1 presents the lateral and vertical variation of sedimentary rock types in terms of their geological formations and deposit environments. It can be observed that the sedimentary rocks are from marine sediments.

Table 2.1 Geological Sequences (Hull and Jackson, 1981 but some modified thicknesses after James et al, 1984)

Deposit/Formation	Rock Types	Depositional Environment	Maximum Thickness
Quaternary			
Holocene (Recent)			
Alluvium	Sand, Gravel, Clay & Peat	Fluvial, estuarine & shallow-marine	80m
Pleistocene			
Terrace Deposits	Sand, Gravel, Silt, Clay & Peat	Fluvial. Estuarine & shallow-marine	6m[a]
Tertiary			
Pliocene and Miocene			
Liang Formation	Sand, Clay, Lignite, Gravel & Tuffaceous beds	Deltaic, estuarine and shallow-marine	600m
Seria Formation	Sandy clay, sand, clay and some lignite	Shallow-marine	2,000m
Miri Formation	Sandstones and mudstones	Shallow-marine	1,900m
Lambir Formation	Sandstone, mudstone, clay and marl with some limestone	Marine, shallow-marine	2,100m
Belait Formation	Sandstone, sand, mudstone and clay with coal and lignite	Marginal marine, partly deltaic	6,400m
Setap Shale Formation	Mudstone and clay with sandstone and siltstone	Marine	5,700m
Meligan Formation	Sandstone with some mudstone	Shallow-marine, deltaic	3,000m

[a] Locally 20m thick in coastal terraces

2.3.2 Mineral Resources

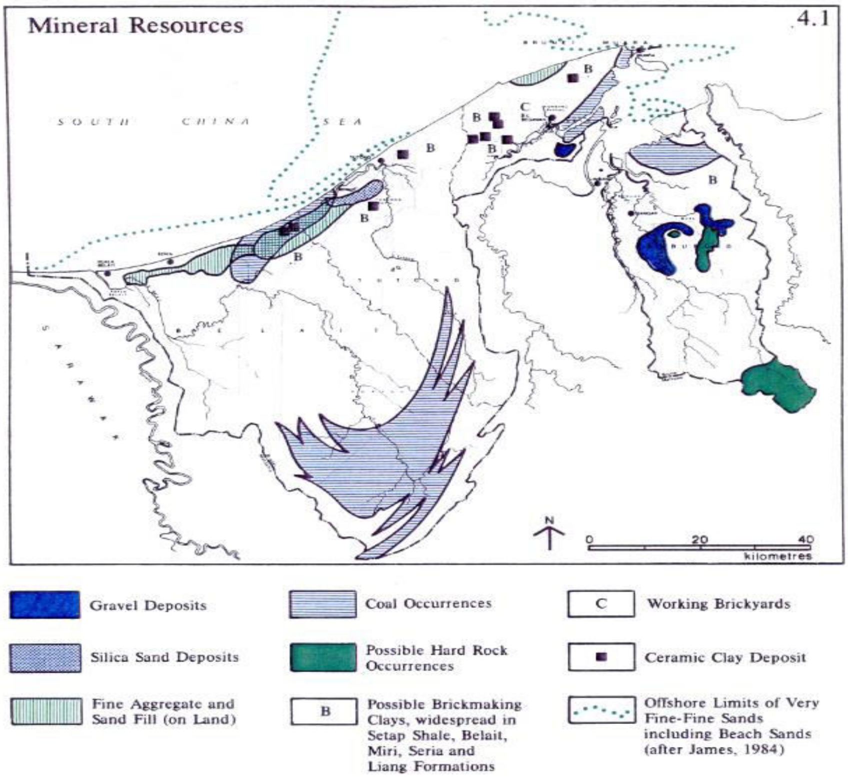


Figure 2.6 Mineral resources found in Brunei Darussalam (TCP, 1986)

In 1953, a five years development programme (now known as National Development Programme) was drawn up for the first time and envisaged the major road construction of 290 miles linking every district in the state. Since then the exploitations of mineral resource has been studied by the British Geological Survey (BGS) team. Being a small country, the available mineral resources are limited and the distribution of various minerals was mapped as shown in Figure 2.6. It can be concluded that the hard rock occurrence can only be found in Temburong district. Therefore for the purpose of the study, the following sections concentrate on a review of geology of Temburong District that will led to the identification of the common rock and available aggregate resources.

2.3.3 Geological formations in Temburong District

As seen in Figure 2.6, the known gravel and hard rock deposits are only located in the Temburong district and are the remaining source of aggregates in Brunei Darussalam. The district has four geological formations, as shown in Figure 2.7, containing workable deposits of sedimentary rock types.

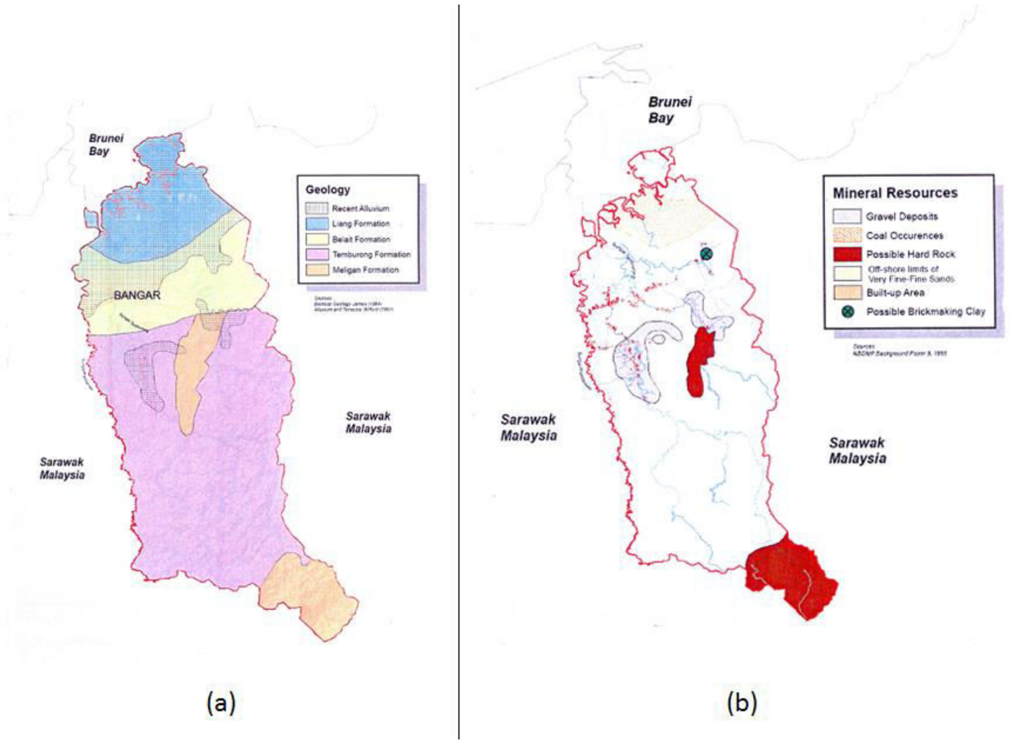


Figure 2.7 (a) Geology of Temburong (b) Rock Resources in Temburong (JT, 2006)

The geological formations are as follows:

- Setap Shale/Temburong Formation (Middle Oligocene to Early Miocene)

  The Setap Shale Formation, also reclassified as Temburong Formation due to the more indurated and argillaceous succession below an unconformity, occupies a large area of the Temburong district. It consists predominantly of hard, dark shale with some intercalations of sandstones and occasionally limestone.

- Meligan Formation (Middle Oligocene to Early Miocene)

  The formation consists of thick-bedded, white grey, well-cemented medium to coarse sandstones less than 3km thick.

- Belait Formation (Early to Late Miocene)

  This formation can be found in Labu area and consists of thick interbedding of sandstones and clays. The sandstones form prominent ridges.

- Liang Formation (Late Miocene to Pliocene)

  The formation occurs in the Lumut Hill, in Berakas syncline and near Limbang and consists predominantly of sands with occasional conglomerates and clays which are abundant in lignite beds of about 0.6 kilometres thick.

2.3.4 Aggregate Resources

It was reported that the mineralogical composition of the sandstone was mostly quartz with specific gravity of 2.3 to 2.5.

The engineering properties of the local rock were first investigated by British Geologist, Tate R.B (1968) and since then, there has been no investigation on the properties of the current quality of the rock.

2.3.4.1 Gravel Deposits

In 1954, the first gravel deposits were discovered in the river valleys at the coastal areas in Brunei-Muara and Temburong District. Unfortunately the exploitable deposits in Brunei-Muara district were depleted in 1960. In the quest of searching for more gravel reserves, the British Geological Survey (BGS) team led by Tate (1974) undertook a major exploration that identified a substantial amount of 21.2 million m^3. The gravel is composed almost entirely of Meligan sandstone which outcrops in the headwaters of the Temburong river between Tudal hill and Lesong hill. The parent rock is grey, fine-grained sandstone, with occasional interbedded conglomerates. During the Pleistocene period, gravel was transported downstream by the Temburong river and deposited in terraces over the relatively flat ground between Telugong hill and the Pendaruan river. It was classified as fine or medium grained sandstone and Quartz was found to be the dominant mineral (Tate, 1968). It formed as single crystal grains from 0.5 to 1mm across or as multi-crystalline grains of similar shape.

In 2003, the gravel reserves had reduced to about 3.8million m^3 within the Temburong valley area (TECA, 2003). Inefficient extraction processes might cause further reduction and a reserve at the lower reaches of the Batu Apoi river was already exhausted. However, to the present, Temburong district still remains as the major source of aggregates. The aggregate is almost entirely from the alluvial gravel deposits where these are confined in the floodplain of the Temburong River, the most accessible reserves. Hull and Jackson (1981) reported that the gravel was composed almost entirely of Meligan sandstone which outcrops in the headwaters of the Temburong river. Figure 2.8 presents the location of gravel deposits and quarries identified by TECA Consultant in 2003 and Temburong Land Department in 2006. In general, three different types of gravel deposits have been identified namely:

i. Alluvial gravels in the river
ii. Low terrace deposits between 13 and 35m above sea level
iii. High terrace deposits between 45 and 80m above sea level but these were too highly weathered for construction materials.

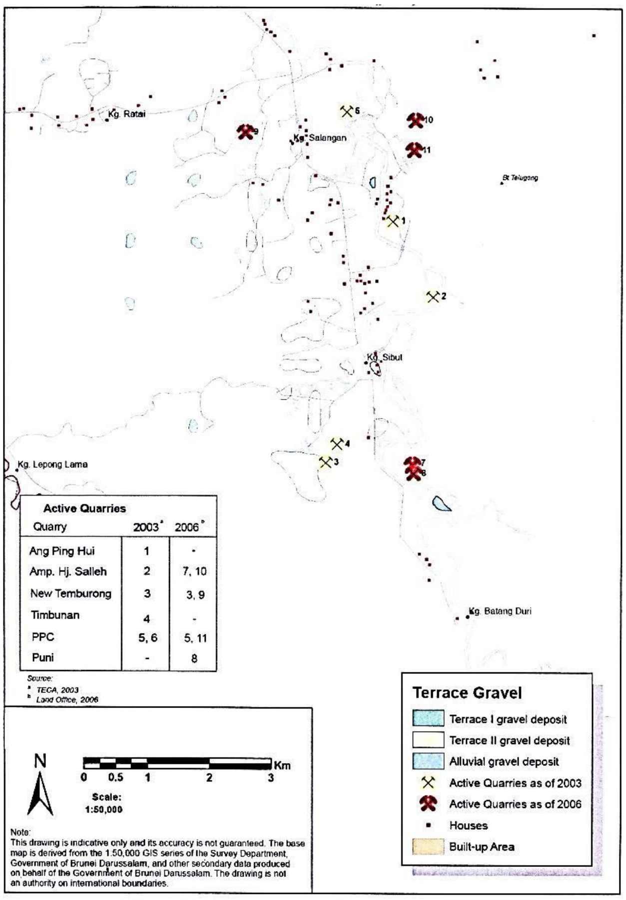


Figure 2.8 Gravel deposits and quarries in the Temburong valley (Teca, 2003 and TCP, 2006)

2.3.4.2 Hard Rock Deposits

The hard rock deposits are only located in the Temburong district as shown in Figure 2.7 (b) and were reported at four identified areas.

- The extreme remote area of hard rock source can be found in the far southeast corner of Temburong district. It consists of hard and very resistant sandstones from the Meligan Formation, which is believed to be the main source of local gravel deposits. However, the area has an accessibility problem due to its location within the Ulu Temburong National Park hindering the exploitation of the source.
- Another source of hard rock may lie within the faulted outcrops of the Meligan formation in the central part of Temburong district.
- The presence of hard sandstone beds was identified in the middle reaches of Temburong river. The bedrock succession upstream gives way transitionally to the dominantly sandstone sequence of the Meligan formation (Tate, 1974).
- The Belait formation in the Biang Ridge is reported to contain silicified hard rock (Tate, 1974).

2.3.5 Aggregate Statistics Data

From the Aggregate statistics data, (CPRU, 2009), the aggregates supply to the local market at a price of B$33.70 (£16.35) per cubic metre is cheaper than imported aggregates at a price of B$44.47 (£22.24) per cubic metre. Table 2.2 shows the statistical figures of the estimated volumes of local and imported aggregates for the period of three years (2006 to 2008) representing their annual consumptions for the main construction activities in the country (Aggregates Statistics Data, 2009). These data are sourced from the Ports Department so the figures of the imported materials are only concerned with the transportation by sea and do not include the materials that are transported over land. Because the Temburong district is not contiguous with the areas of Brunei-Muara district where the construction activity is centred, most Temburong-sourced aggregate does in fact, travel through Malaysian territory and, hence, is measured in Table 2.2.

Although there is no documented evidence of the consumption of aggregates dedicated to road construction activity, it is known that the imported sources of high quality aggregates dominate the road pavement construction activity to meet the required specifications. In addition, the road contract documents invariably specify imported aggregates for most major roads. Figure 2.9 presents a pie chart showing the estimated local and imported aggregates consumption by the overall construction sectors. Figure 2.10 compares the distribution of budgetary funds for each main category in construction sectors of National Housing, public buildings (such as schools, mosques, sports complex, clinics) and roads. It is assumed that when Figure 2.9 is superimposed on Figure 2.10, it indirectly suggests that the road construction sector predominantly used imported aggregates.

Table 2.2 Statistical Figures of Local and Imported Aggregates Volumes (Ports Department, 2009)

Year	Local Aggregates (m^3)	Imported Aggregates (m^3)	Total(m^3)
2006	359,640	75,510	435,150
2007	372,390	85,785	458,175
2008	337,991	67,164	405,155
Total	1,070,021	228,459	1,298,080

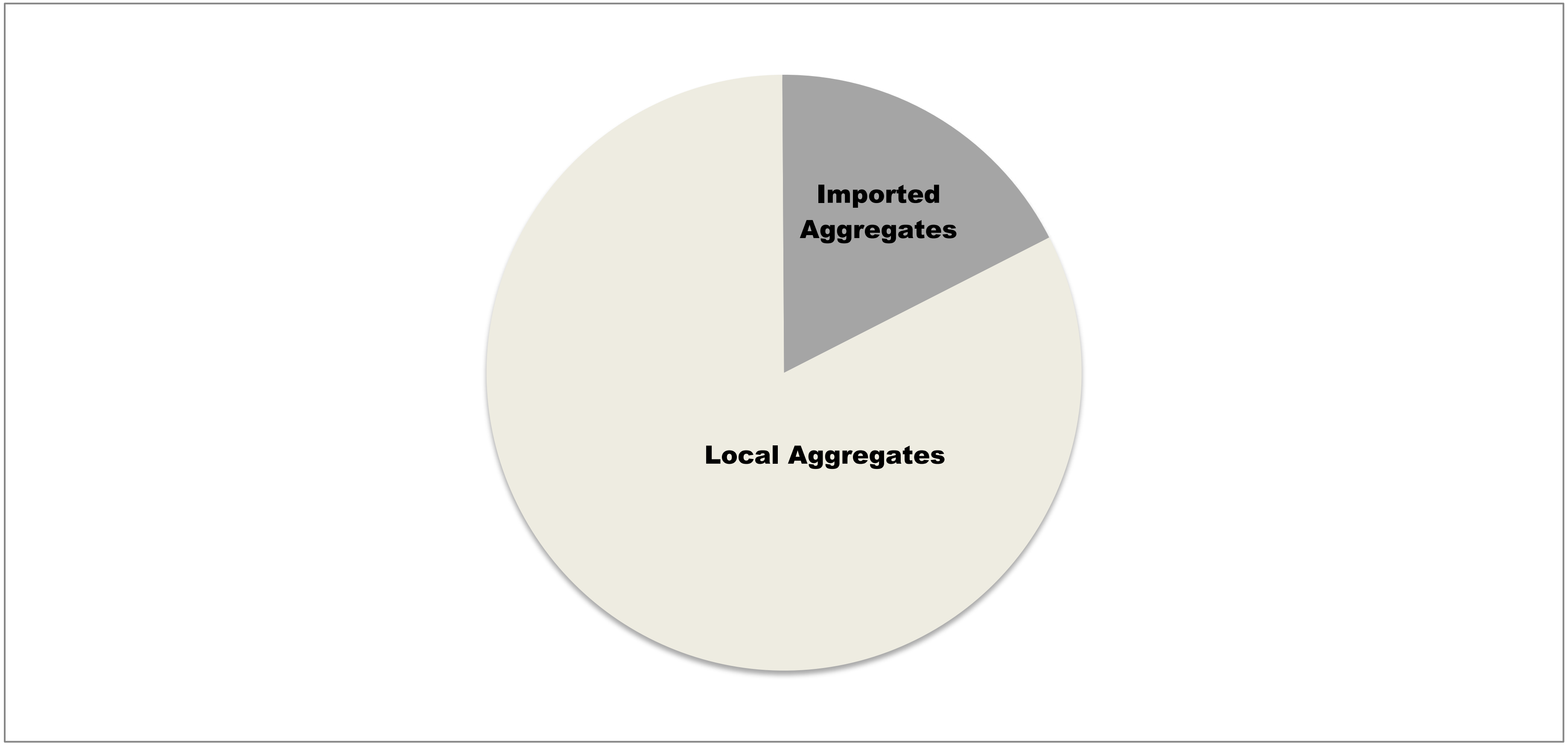


Figure 2.9 Consumption of Local and Imported Aggregates (Ports Department, 2009)

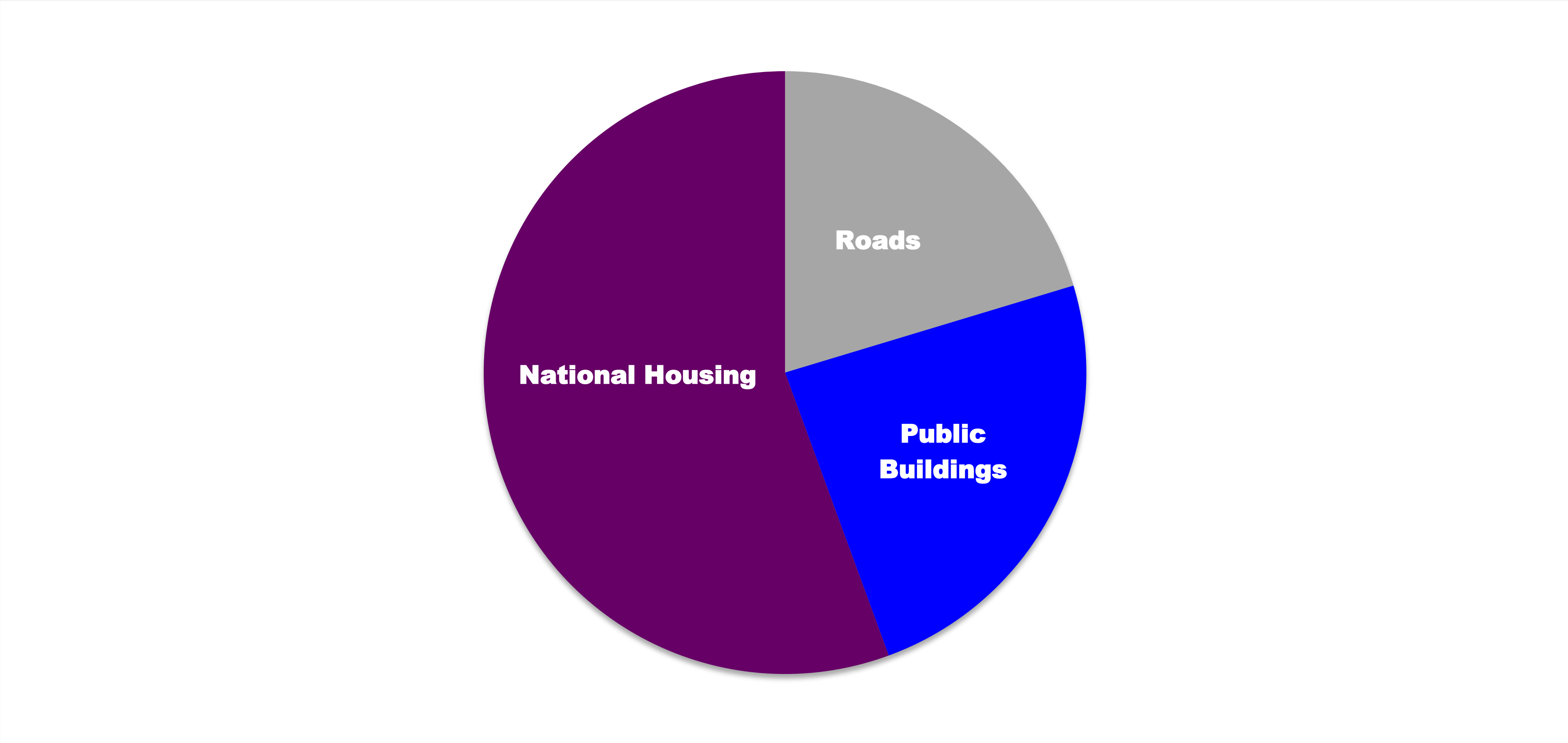


Figure 2.10 Budget allocation for different sectors in construction industry (BEDB, 2009)

2.4 ROAD CONSTRUCTION AND DESIGN PRACTICE

In addition to the climatic condition and geological features, the type of pavement structure is influenced by the local preferences, standards and traditions.

2.4.1 Pavement Specifications

Currently the specifications for road construction in Brunei Darussalam follows a 'recipe' approach that requires the pavement materials to qualify the basic physical properties as stated in GS1 (CPRU, 1998) such as gradation, aggregate crushing value (ACV), aggregate impact value (AIV) and Ten Percent Fines value (TFV), compaction density and moisture, California Bearing Ratio (CBR) and Marshall mix parameters. They control the quality of granular materials to be used for asphaltic concrete (i.e wearing course and binder course), road base, sub base and sub grade layers for the flexible pavement construction. The GS1 specification further asserts that the quality of crushed aggregates for road base and sub-base "shall be mechanically stable crushed gravel, or crushed rock or a mixture of crushed and natural aggregates which is hard, durable, clean and essentially free from clay and other deleterious materials".

The qualification of the quality of the granular materials for road base and sub-base stabilisation in GS 7 (CPRU, 1998) is the same as the ones stated in GS 1 (CPRU, 1998). Three types of stabilising agents are stated in GS 7 such as cement, bitumen emulsion and polymer. Again the strength of the stabilised products is mainly assessed based on the compressive strength and CBR tests. However, for any new materials and technology that are often accompanied by overseas technical guidelines, their applications have been on a trial basis and failures could occur unexpectedly as the guidelines do not meet the specific needs and conditions in Brunei Darussalam.

Both specifications emphasise more on the qualification of quality of pavement materials with strict quality control on site such as compaction density and CBR requirement. These requirements alone however, cannot assure the quality of pavement materials to produce durable pavement as they can turn out to be the cause of failure.

2.4.2 Pavement Design

At present, there is no pavement design standard developed for Brunei Darussalam's condition. Therefore, the pavement design practise is mainly based on the empirical pavement design procedures which are developed elsewhere. The empirical design method is mainly based on a design principle of limiting the pavement distress to a level that experience has shown to be acceptable. However the main reasons that the pavement design has not been substantiated in Brunei Darussalam are the limited road research and database of historical pavement performance and structural properties of pavement materials. The structural property data such as stiffness and strength are the main design input parameters in the mechanistic pavement design approach.

The input requirements of these empirical procedures do not make room for empirical information given local conditions. Climatic factors such as rainfall and temperature have significant influence on the properties of road pavement materials in a given geographic area. These are rarely taken into account in the existing pavement design methodologies; only the local conditions that applied during the empirical studies of observed road performance on which the designs were based.

There are two empirical design standard manuals currently still used in Brunei Darussalam. One is Road Note 29 (Road Research Laboratory, 1970) with Road Note 31 (Transportation and Road Research Laboratory, 1993), the other is the new revised Malaysian Pavement Design Standard (JKR Malaysia, 2013).

2.4.2.1 Road Note 29 Design Method

The Road Note 29 design procedure (Road Research Laboratory, 1970) contains design charts and tables derived from the results of the many full scale road trials have been used for UK conditions. The method allows a pavement to be designed for a life of a selected number of years by assessing the cumulative number of commercial vehicles to be carried. With CBR values for the sub-base, the thickness can be determined. The selection charts for the surfacing layer are limited to only two types of asphaltic concrete material; hot rolled asphalt and dense macadam asphalt categorised by number of standard axles. A standard axle is defined as axles applying a force of 40kN. For traffic over 11 million cumulative number of standard axles, the recommended bituminous surfacing layers for flexible pavement are wearing course of minimum thickness of 40mm and base course (binder course) of a minimum thickness of 60mm. This standard is no longer used in the U.K and it has been replaced by the current Design Manual for Roads and Bridges (DMRB) (UK Highway Agency, 2006). Road Note 29 is however, still familiar to many along with the Overseas Road Note 31 which is described in the followings section.

2.4.2.2 Overseas Road Note (ORN) 31 Design Method

This Note is based on research and experience in over 30 countries, located in the tropical and sub-tropical regions. The latest revision in 1993 contains a catalogue of pre-designed pavement structures covering a wider range of materials that cater for traffic up to 30 million standard axles. It gives a detailed procedure to estimate moisture content at which the bearing capacity of the subgrade should be determined in terms of CBR strength. The CBR strength is categorised into six subgrade classes, reflecting the sensitivity of design thickness to this quantity, catalogued ranging from S1 (2%) to S6 (30%). The material properties and design requirements for unbound granular road base and sub-base are specified in terms of gradation, Ten Percent Fines (TFV) and CBR strength. The catalogue is available for either a surface dressing or premixed bituminous surface of limited thickness, 50mm to 150mm, on different combinations and thicknesses of base and sub-base materials. Generally, the total pavement thickness for bituminous surfacing flexible structures is determined based on subgrade CBR and traffic volume.

2.4.2.3 Malaysian Pavement Design Manual

The Malaysian pavement design manual (JKR Malaysia, 1985) was first developed in 1985 based on AASHTO empirical design procedures that have undergone several revisions. Recently a new revised Malaysian pavement design manual (JKR Malaysia, 2013) has been developed in which design approach combines the data obtained from the field (such as CBR) and mechanistic analysis methods into a single tool. The tool is presented in a catalogue format of pre-designed pavement structures with traffic volume and sub-grade strength as primary input. Subgrade stiffness is quantified as CBR value that is then used to categorise the subgrade class such as 5% to 12%, 12.1% to 20%, 20.1% to 30% and more than 30%. The traffic is predicted by a number of ESALs over the design period and categorised into five classes: T1 (≤1 million ESAL), T2 (1 to 2 million ESALs), T3 (2.1 to 10 million ESALs), T4 (10.1 to 30 million ESALs) and T5 (>30million ESALs). The pavement structural types consist of a conventional flexible with granular base, a deep strength with stabilised base and a stabilised base with surface treatment.

2.4.2.4 Other Design Manuals

In addition to the Road Notes and Malaysian manual, pavement design has been greatly influenced by several consultants who use design methods developed by the American (AASHTO) and Australian (Austroads) authorities, regardless of environmental and climatic conditions, particularly when new pavement materials or technology is applied. The main reason is due to the lack of local guidelines and specification.

2.4.3 Flexible Pavement configuration

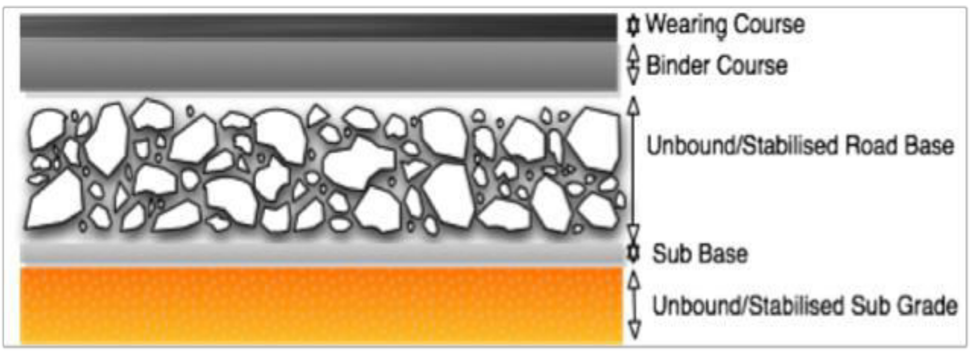


Figure 2.11 Typical conventional flexible pavement structure in Brunei Darussalam

Flexible pavement has been favoured in most road construction in Brunei Darussalam and a typical pavement structure is as shown in Figure 2.11. It is made up of two upper layers of bound asphaltic concrete wearing and binder course overlying road base, sub-base and subgrade layers. Road base mainly consists of unbound granular materials and in some weak ground areas, these materials are normally stabilised by using additives such as cement, dry powdered polymer and other agents to produce stiffer and moisture resistant material. Sub-base is normally made up of low quality aggregate materials which are able to protect the subgrade particularly during construction. Sub-grade is made up of cohesive soil as a foundation layer and can be stabilised where the ground is very weak.

2.4.4 Classification of Road

The roads in Brunei Darussalam are classified in GD15 (MOD, 1998) as follows using socio-economic considerations:

Class 'A' road provides a primary route among major centres in urban areas as well as for longer distance journeys between the major population centres in the city and other main towns in the country. It consists of major highways with dual carriageway to accommodate the high volume of traffic and is designed for high speed. It is often characterised by a U turn or at grade intersection or a flyover. Generally it provides uninterrupted flows in which interchanges, either overpasses or underpasses, are provided to permit traffic to pass through the junction without crossing any other traffic streams. Entrance and exit to the highway are provided at interchanges by slip roads that link to the secondary or distributor roads. U turn ramps are provided along major highways to allow the traffic to turn and re-join in the opposite direction.

Secondary road is the second in the hierarchy, classified as Class 'B', that forms traffic movements into and out of a town and links between the primary road network and the distributor road. Direct accesses to developments are not allowed, neither is parking at the roadside. However, partial control of access and smooth traffic flow are often necessary to accommodate their relatively high traffic volume. The flow of secondary road is usually controlled by signalised intersections and roundabouts to regulate the relatively high traffic volume. Main shopping areas, schools, hospitals can often be found on secondary roads.

Class 'C' road distributes traffic within towns and links the secondary roads to residential areas. It consists of small roundabout and intersections with local access roads that favour traffic movement on the distributor roads. Key community functions such as social amenities, outdoor recreational facilities, health centres and small scale commercial areas can often be found on this type of road.

Local access road falls under Class 'D' serving an important route to the individual residential properties or farming areas. It provides links to distributor roads where the public can gain access to health centres, markets, schools, recreational facilities or shopping centres. It carries the low traffic volume with the lowest speed limit.

2.4.5 Road Length Statistics and Construction Practise

Most of the roads in Brunei Darussalam are paved with bituminous surfacing known as asphaltic concrete for the upper wearing and the lower binder course layers. Table 2.3 shows the road length statistics which are classified by road class and pavement type.

Table 2.3 Road length statistics by pavement type and road class in Brunei Darussalam (DOR, 2009)

Road Class	**Pavement Type**				**Total Length (km)**
	Asphalt	**Concrete**	**Gravel**	**Earth**	
'A' (Primary)	430.8	-	-	-	430.8
B (Secondary)	248	-	-	-	248
C (Distributor)	780.7	9.3	167.3	-	957.3
D (Local Access)	663.1	206.6	257.6	1.8	1129.1
Total Length (km)	2122.6	215.9	424.9	1.8	2765.2

Some sections of road class C and D are still made up of gravel roads. These gravel roads were built in the early 1970s particularly the ones leading to the residential areas in villages and very rural communities (which are now classified as distributor and local access roads) under the jurisdiction of the District Office. These gravel roads were bound with fine gravel particles. During rainy seasons, these roads were impassable to traffic due to the severe erosion cause by the high intensity and volume of rainfalls. Meanwhile in dry conditions, the fines would form dust due to the moisture loss.

Over the years, these areas became more developed with many residential properties and had been taken under the jurisdiction of the Public Works Department, in the Ministry of Development Brunei Darussalam. Many of these roads were temporarily overlaid with milled asphalt to overcome the problem of dust and make them passable to traffic during rainy days. Since the 1980s, flexible pavements with bituminous surfacing layer as wearing and binder course were introduced, these gravel roads have been actively rehabilitated and upgraded to the standardised flexible pavement.

However, many of these roads sit on organic clay in low terrain areas. They are subsequently most at risk from flooding. Therefore, these roads are mainly designed to a new level higher than the water table and constructed with stabilised road base to increase the stiffness due to the weak ground and minimise the water induced damage. The concrete roads serve accesses to small numbers of the civil servant's residential areas and some commercial areas. They account for about 215km of the total road length.

The road statistics in terms of length in km are plotted in Figure 2.12 for each district in Brunei Darussalam. It shows that asphalt surface is the most preferred type.

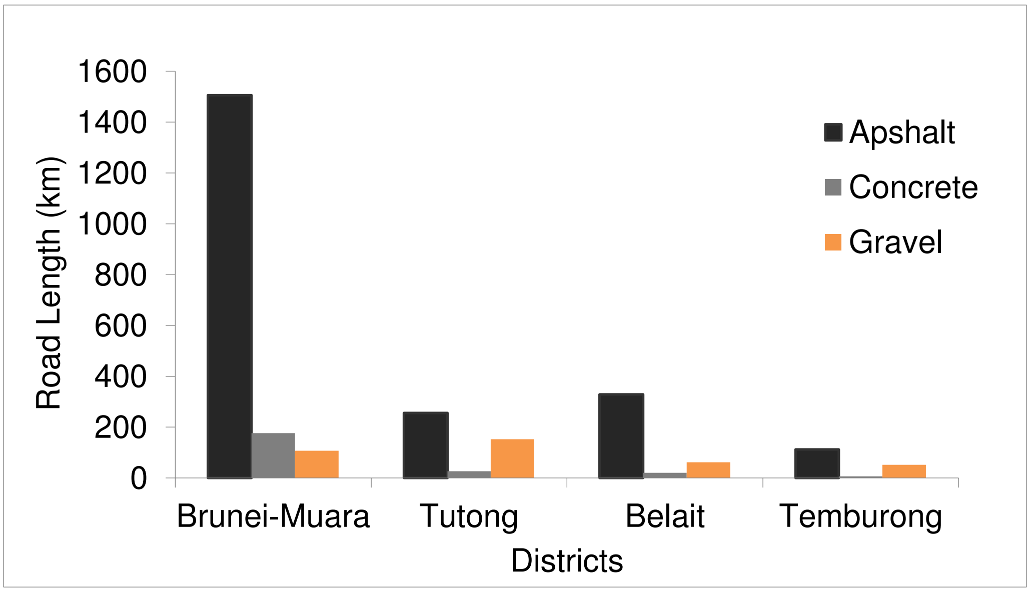


Figure 2.12 Statistics of the road pavement's surface layers for each district in Brunei Darussalam by lengths

2.4.7 Pavement Stabilisation Practise

Road pavement stabilisation has been practised in many parts of the world and is heavily influenced by the different factors experienced in each country. In Europe, stabilisation is mainly driven by environmental concerns by utilising the available waste products and combining them with selected stabilising agent to produce usable pavement materials. In Australia, the pavement stabilisation is primarily driven by the shortage of well-graded materials and low funding for its extensive road network (Wilmot, 2006). In Brunei Darussalam, stabilisation is mainly in the road base or subgrade layers and is driven by the weak ground condition and moisture susceptibility. However, the stabilised materials particularly in road base are mainly made up of imported quality aggregates similar to the unbound road base. Furthermore, any newly introduced stabilising agents apart from cement are imported and mainly accompanied with overseas design guidelines.

Stabilisation methods have been around for decades in Brunei Darussalam. The first stabilisation was carried out for the construction of the first main road, about 97 kilometres in length linking the capital to Tutong town. The road base was stabilised with cement about 8% to 10% (Myles, 1957) and the road has been standing till now.

Since 1995, stabilisation works have been making a 'come back' with updated technologies and the introduction of new stabilising agents such as Modified Polymer and/or Cementitious Chemical binders i.e 'Chemilink' and 'Renolith' and dry powdered polymer (Teck-Chin and Rahman, 2001). At the time of writing, stabilisation methods have become a common practise to tackle problematic construction sites such as those with poor bearing capacity or water susceptibility for either new road construction or rehabilitation.

The stabilisation is carried out mainly on the new laid subgrade, sub-base or road base materials depending on the pavement design. Therefore, the in situ stabilisation is conducted by overlaying new granular materials and then a selected stabilising agent would be spread onto them. It then follows with the pulverisation of the stabilising agent and the new granular materials before they are compacted. In road rehabilitation, the in situ stabilisation is carried out by raising the road level into a new designed level particularly at flood prone areas. Another stabilisation method is by mixing the stabilised materials in the central mixing plant. Then the stabilised materials are transported to the construction site for laying and compaction (Dong-Qing and Teck-Chin, 2004).

However, any new stabilising agents are accompanied by overseas test results with limitations on the type of materials. Even though the specification for pavement stabilisation was established in 1999 (CPRU 1999:GS7), it is limited to cement, cement/bitumen and polymer stabilisation. Despite the limitations, cement has still remained as the traditional binder in road stabilisation in Brunei Darussalam because the application process is familiar to many and it is readily available in the local market.

2.5 SUMMARY

In tropical equatorial regions, some of the most significant factors are the frequent rainfall or flooding leading to the prolonged exposure of pavement to water. Phenomena related to the high humidity environment and the alternate hot and wet rainy days can be significant factors affecting the pavement structure.

The review of geology gives a better appreciation of the nature and behaviour of the rock available in Brunei Darussalam and the potential exploitation for a new source of local aggregates. It can be concluded that there is no known evidence of igneous or metamorphic outcrops in Brunei Darussalam. Sandstones are the common hard rock types and can only be sourced in Temburong district.

The dependency on the supply of imported quality aggregates identifies the need to investigate alternate road base treatment methods by including the local sandstone aggregates of low quality. Although the current reserve of the local aggregates had shown depletion in production, it was reported that there are other sources of the aggregates found in Temburong district. Future exploitation to source aggregates at other new areas may be possible. Even though it is recognised that it is unlikely that materials of comparable quality to that of imported varieties would be found, it is desirable to locate sufficient local reserves to meet future unexpected demands for aggregates. Furthermore, some sections of the old roads are made up of local sandstone aggregates in the road base layer, therefore any future recycling works would anticipate the inclusion of these aggregates. Therefore, there is a need to improve the properties of these sandstone aggregates by treating them with any suitable treatment method or agent to make them as usable road base materials. This would reduce the dependency of the country on imported quality aggregates for road construction particularly in the road base layer.

The lack of a pavement design and specification standard does not allow the classification of other types of granular materials and alternative strengthening methods that can be reliable for their applications in Brunei Darussalam. Furthermore the limited research on the mechanical properties of pavement materials used is one of the main reasons pavement design has not been developed properly.

The current design guides in use in many tropical countries do not make allowance for an objective assessment of tropical local conditions where the road will be used.

CHAPTER 3 EVALUATION OF CANDIDATE STABILISATION METHODS

3.1 INTRODUCTION

This chapter reviews the potential stabilisation methods or agents for their possible use to improve the sandstone aggregates as usable road base materials in tropical equatorial climate regions. The objective was to find the treatment method or agent that seems to be suitable to be looked at in the future experimental research; the method must be reliable, good and cheap for its application in Brunei Darussalam. Therefore, a weighted matrix with selected criteria was constructed to produce a subjective ranking of treatment methods which is explained in the proceeding sections.

The study included the selection of significant criteria and key issues in order to create a scoring system that provides a ranking of the possible treatment methods. The selected method would be subjected to further investigations to identify the mechanical properties and performance mechanisms.

This study concerns the treatment of marginal aggregates with stabilising agents or binders:

- To find a feasible treatment method or agent that seems to be suitable to be looked at in future experimental research
- To ensure the candidate method would suit the typical conditions in Brunei Darussalam such as climatic factors
- To reduce the moisture susceptibility so as to provide more durable pavement performance.

3.2 REVIEW OF CANDIDATE STABILISATION METHODS OR AGENTS

There is a wide range of stabilisation methods or agents available across the world that can possibly improve the properties of sandstone aggregates. They have different characteristics and strength however, the benefits and limitations are dependent on the sandstone aggregates and particularly the climate of application. Therefore a desk review was conducted to omit those with uncertainties and list out the candidate stabilisation methods or agents as shown in Table 3.2 to be considered in the ranking evaluation. The candidates were categorised based on their types of bonding that they produced with the sandstone aggregates namely, mechanical, polymer, cementitious and bituminous bonding as shown in Table 3.1

Table 3.1 Bonding Group Categories

Bonding Category	Stabilisation Methods or Agents
Mechanical	Compaction, Gradation, Crushing Techniques, Reinforcement, Water Bound Macadam,
Polymer	Dry Powdered Polymer, Ligno-Sulfonates
Cementitious	Cement, Lime, Rice Husk Ash (RHA)
Bituminous	Bitumen Emulsion, Foamed Bitumen

3.2.1 Mechanical Bonding

Aggregate makes up a significant portion of unbound road base material in the pavement construction in Brunei Darussalam. A durable road base material can be produced when constructed with quality materials.

Aggregates are made up of different sizes and shapes of particles obtained by crushing rock whereby force is applied with sufficient energy to disrupt the internal bonds or planes of weakness within the rock (Barksdale, 1991). The fracture mechanisms are greatly influenced by the rate of energy generated from the crushing techniques, either compressive or impact. The mechanisms generate contact points of the aggregate particles. Compaction process arranges the aggregate particles closely together by means of a mechanical equipment and thereby contact one another at many “points” to form a stable aggregate skeleton (Marek, 1977). The stability of unbound road base is highest when the aggregate particles attain their maximum packing density.

Gradation is a key parameter to obtain the maximum density during compaction where a maximum number of particle contacts is developed. For example a well graded material is composed of particles with a large range of different sizes in which the fines are designed to fill the voids between coarser particles. The ideal grading may be defined by using Fuller’s equation:

$$P = 100\left(\frac{d}{D}\right)^{n} \qquad \text{Equation 3.1}$$

where,

P = % finer than the sieve size,d

D = maximum aggregate size in the gradation

n = the coefficient which adjusts the curve for fine or coarser zones (many researchers have observed that n=0.45 for a maximum density packing)

However, low quality aggregate tends to degrade under compaction, thus they generate more fine particles. When the particle size distribution becomes finer, the fines will eventually dominate the aggregate matrix in the behaviour of a road base structure. The coarse aggregate particles would "float" in the fine matrix and lose the particle contacts leading to a less stable road base.

If good aggregate particle interaction is not achievable by compaction, another form of mechanical bonding treatment is by introducing a reinforcement agent. Geosynthetic materials, such as geo-grid, are commonly used in pavements. They may reinforce layers of granular materials providing lateral confinement or they may provide a filtration or separation layer to minimise the migration of fines. However their implementation over large areas would be complex and/or costly.

Fibres are alternative reinforcing agents that are commonly used in concrete works and sometimes, soil. Natural fibres such as coir fibres are cheaper than any synthetic fibres and have potential to reinforce soil and, perhaps, pavement layers. Coir fibre is a common waste product obtainable from coconut husks. It is composed of a highly lignified form of cellulose. It was first introduced into the local market in 2009 on the basis of its ability in slope reinforcement. Its first pilot project was carried out on a slope where the main hospital is located in the capital city of Brunei Darussalam. They are sourced from a neighbouring country in an attempt to minimise the abundance of these waste materials in the region by using them for more beneficial functions. It has a high tearing resistance and retains its property to some extent in wet weather conditions (Sivakumar Babu and Vasudevan, 2007). It is a biodegradable material hence it can be a feasible solution in applications where it is meant to serve only during initial stages and where final strength is attained by other means (Subaida et al, 2009).

3.2.2 Cementitious Bonding

Cementitious methods involve the utilisation of binders such as cement, lime and pozzolanas like fly ash and rice husk ash (RHA). Amongst all of these, lime is one of the oldest binders, along with cement, in the treatment or stabilisation of aggregates for use as road base. Both have self-cementing properties whilst pozzolana, a siliceous or aluminous material, has no binding or self-cementing properties unless mixed with activators such as cement or lime, where, in the presence of water, it forms a cementitious bond.

Lime has been a successful binder in pavement stabilisation particularly in Australia (Vorobieff and Preston, 2004) and is known to effectively stabilise clayey soil, which contains silica and alumina, when a pozzolanic reaction can take place.

Cement is a common material used to stabilise road base construction over decades in Brunei Darussalam since 1950s. It has been utilised in pavement stabilisation aiming to reduce moisture susceptibility and, hence, to reduce the rutting problem. However, shrinkage cracking has become a concern in cement-stabilised bases. Over time, the crack could propagate to the asphalt surface forming reflection cracks thereby causing further pavement distress. Cement stabilised mixtures can be found almost in all established pavement design manuals and standards across the world, including Overseas Road Note 31, Austroads, U.K DMRB and Brunei Darussalam, GS 7 (CPRU, 1999).

Asia is the leading region in the world in the production of rice and therefore, rice husk is extremely prevalent. Rice husk can be burned kilns to make various things for example generate fuel for the production of fired bricks and the waste product is a useful pozzolana material called rice husk ash (RHA). RHA contains 90% silica that gives it potential for use in soil stabilisation. However, as a pozzolana material, it needs an activator to develop the strength of the road base materials (Hodgkinson and Visser, 2004) such as lime or cement.

Although cementitious bonding agents have good track records for pavement stabilisation, in the old days particularly Portland cement, but in recent years premature shrinkage cracks have often appeared after construction and in some cases, transverse cracks appeared after a short period of service life. The variations in material sources and properties as well as the local site conditions and construction might influence the performance of cementitious stabilised mixes.

3.2.3 Bituminous Bonding

Bituminous bonding in pavement structures has always been favoured because it forms a material that is ductile and flexible under traffic loading. There are two types of treatment agents, namely bitumen emulsion and foamed bitumen that generate bituminous bonding and are commonly used to stabilise road base materials.

Bitumen emulsion consists of two immiscible liquids, bitumen and water, which are temporarily stabilised by an emulsifier. It is known to have a good adhesion and is popular in the treatment of aggregate base (Thanaya, 2003). However, the ‘breaking’ process of the emulsion has to take place to develop the adhesion. The ‘breaking’ process involves the removal of a substantial amount of water by evaporation or drainage so as to release the bitumen to form a continuous adhesive film that holds the aggregate particles together. The benefits of bitumen emulsion as a treatment agent standout more in hot and dry weather conditions that would accelerate the evaporation process and hence, produce mixes with good adhesion. Good adhesion is evident when the bitumen emulsion mixtures perform well in a soaked condition (Hodgkinson and Visser, 2004). The selection of bitumen emulsion is crucial and dependent on the types of aggregates. However, it is often ruled out due to the long 'breaking' process of bitumen emulsion particularly when humid and wet climate are the issues as these would slow down the evaporation process, hence slow adhesion. In this case, cement is normally added to overcome the issue and gain the early strength rapidly. The selection of cationic and anionic bitumen emulsion type should be considered as good adhesion depends on the aggregates to be utilised, either alkaline or acidic types. Alkaline aggregates are the basic type such as limestone whilst acidic aggregates are the siliceous type such as sandstone or granite. Application of emulsions cannot be carried out in wet weather, or when rain is expected during the curing period; the emulsion may then be washed off the aggregates.

Foamed bitumen is a fine mousse formed by the instantaneous expansion of bitumen to about 15 times its original volume through the injection of small quantities of cold water into a stream of hot penetration grade bitumen. It can be mixed with mineral aggregates at ambient temperature and at in-situ moisture contents (Jenkins et al, 2000). Upon compaction, the foamed bitumen-stabilised base can gain adequate strength so that it can be trafficked immediately (Jenkins, 2000). Foamed bitumen typically contains 97% of bitumen, 2.5% of water and 0.5% of additive (Ramanujam and Jones, 2007). It is commonly used in combination with additives such as lime, cement and Pulverised Fuel Ash (PFA) in order to develop early-life strength and so as to improve the durability of the mixtures (Khweir, 2007). A good adhesion of bitumen and aggregate also contributes to a more durable foam mix. When using higher quality aggregates like an inherently strong crushed limestone, an additive may became unnecessary due to its minimal effect as compared to utilising low quality aggregates that may gain much improved durability (Castedo et al, 1985).

3.2.4 Polymer Bonding

Lignosulfonate, a waste product of the paper pulp industry, has a long history as a dust suppressant agent and surface stabiliser for unpaved roads in U.S.A and Sweden (Lennox and Mackenzie, 2008). It contains lignin, one of the most abundant polymers in the world. However, this waste product is currently non-existent in Brunei Darussalam.

Another promising Australian commercial product is a dry powdered polymer. It has a proven, successful, track record. It encapsulates the soil particles to prevent water penetrating into the soil (Lacey, 2004). Based on the author's experience, the construction application was simple and the performance was a success. The drawback was that the polymer had to be imported from Australia which was costly and uneconomical for a continuingly applicable solution.

3.2.5 Summary of the Binders

From the above literature surveys, Table 3.2 summarises the list of the available possible binders and their shortcomings.

Table 3.2 Potential Binders Availability and Shortcomings

Binder	**Availability**	**Shortcomings**
Cement	Available in locale	Prone to shrinkage cracks
Lime	Unavailable	Longer curing process to allow mellowing
FlyAsh Class C & F	Unavailable	No binding property on its own and it requires an activator such as lime or cement for a pozzolanic reaction to take place for the development of required strength
Rice Husk Ash (RHA)	Limited due to a small usage of rice husk for the production of fired bricks.	No self-cementing property therefore, it requires another agent such as lime or cement for a pozzolanic reaction to develop the strength
Coir Fibres	Available in the local market	Biodegradable
Foamed Bitumen	Available	A standard bitumen type is normally use for its production and it contains less water than bitumen emulsion but the high humidity and sudden rainfall might be a challenge to achieve a complete curing.
Bitumen Emulsion	Limited as its production requires an emulsifying agent	Leaching of binder may occur if sudden rainfall occurs during construction.
Ligno-sulfonate	Unavailable	Leaching may occur if sudden rainfall occurs during construction.
Dry Powdered Polymer	Unavailable	An Australian commercial product and costly to import.

3.3 RANKING EVALUATION OF CANDIDATE STABILISATION METHODS OR AGENTS

Thirteen candidate stabilisation methods or agents were considered and reviewed in the previous section. Due to the time limitation and material availability, therefore, on the basis of the literature surveys, a subjective weighted matrix was constructed to conduct a ranking evaluation of these stabilisation methods or agents. This is explained in the subsequent sections.

The assessment and evaluation methods presented in this section are selected to identify the most feasible treatment methods or additives (or combination of additives) for long term and cost-effective stabilisation of sandstones in tropical equatorial climate regions. The methods involved assigning weighting factors to each of the criteria in which the value measures the relative importance or impact of each criterion on two categories of roads namely rural and urban roads. Then in addition, the selection should concentrate on the use of local materials and of possible cheap regional products for reasons of economy and of sustainability.

The objectives of this ranking evaluation were:

- To decide between the candidate treatments for a suitable stabilisation method or agent that seems to be suitable to be looked at in future experimental research
- To select criteria for the weighted matrix that are tailored to the conditions in Brunei Darussalam in terms of climatic factors, traditions and practise, economy and local resources.

3.3.1 Weighted Matrix

Weighting Factor

Two main categories of roads were considered namely rural and urban roads. Therefore, the weighting factor (W.F.) is a value to quantify the level of importance of each criterion to each category of road. The relative importance of each criterion in the decision making was quantified in terms of weighting factors. Ten weighting factors were assigned to each criterion based on its level of importance to each category of road as shown in Table 3.3. Ten criteria were chosen to recognise the significant conditions in Brunei Darussalam in terms of climatic factors, standards and traditions, local preferences in structural types, economy and resources. In this case, the ranking evaluation was subjective.

Some criteria were weighted more for rural roads than for urban roads or vice versa. This was due to the fact that traditional urban road's planning and standards often incorporate high implied levels of service that are not appropriate for rural roads and that, when used in rural areas, result in unnecessarily expensive solutions. Moreover, the focus of mass infrastructure investment is toward high volume roads.

The cost criterion was assigned the highest weighting factor relative to other criteria for both roads as it was the key criterion in this evaluation. The reason was to promote the use of local resources for the reduction in the dependency of imported pavement materials which has been the main issue in the road construction industry in Brunei Darussalam.

However, for rural road, time to build criterion is weighted more than the urban road because this road is considered as the main access to the residential areas. Therefore, this criterion is crucial for rural road as compared to the urban road which is normally constructed in a new area.

Table 3.3 Weighting factor (W.F) value for each criterion

Criteria	W.F Values	
	Rural Roads	**Urban Roads**
Cost of Materials	10	10
Moisture	7	8
Aggregate Degradation	4	5
Reliability	5	4
Time and Traffic Dependent Decay	6	7
Time to build	8	3
Maintenance	1	1
Sustainability	6	7
Ease of Construction	3	4

Scoring Factor

Independent of the weighting factor determination, a scoring factor (S.F.) was assigned to quantify the level of satisfaction for each candidate treatment method or agent in meeting each criterion. A high value of the scoring factor, positive two (+2), identifies as an excellent candidate whereas the negative two (-2) indicates as a worst candidate and the zero (0) value is as average, neither better nor worse. The meaning of the score values is shown in Table 3.4.

Table 3.4 Scoring factor (S.F) values

S.F Values	Level of satisfaction
2	Excellent
1.5	Very Good
1	Good
0.5	Fairly Good
0	Average
-0.5	Fairly Poor
-1	Poor
-1.5	Very Poor
-2	Worst

Weighted Scores

The weighted score (W.S) of each criterion for each road category was calculated by multiplying its respective weighting factor (W.F) by the assigned scoring factor (S.F) to each candidate. The total weighted scores of all criteria for both road categories were summed up to arrive as the grand total score of each candidate treatment method or agent.

$$W.S = W.F \times S.F \qquad \text{Equation 3.2}$$

$$Grand\ Total\ Score\ = Total\ W.S_{Rural} + Total\ W.S_{Urban} \qquad \text{Equation 3.3}$$

3.3.2 Descriptions of Criteria

Cost of Materials is the key criterion that dictated the ranking of the treatment methods or agents. As mentioned in Chapter 1, Brunei Darussalam is highly dependent on imported road materials particularly conventional aggregates and stabilising agents therefore the cost criterion is significantly important in evaluating the candidate treatment methods and decision-making in which the weighting factor was assigned to the highest value, equal to 10, for both road categories. The selected treatment method or binder should be relatively inexpensive. It should be able to optimise the utilisation of locally available aggregates by enhancing their properties through treatment on its own or using additives, preferably available locally. The treatment methods or agents that are readily available in the local market would be assigned at positive values. A high positive value of the scoring factor would be assigned to a treatment method that could contribute to low cost by optimising the utilisation of locally available aggregates. The scoring factor would be negative if the treatment method uses unreadily available or imported materials leading to high implementation cost.

Moisture is one of the climatic factors that needs to be considered to produce quality road base materials both during the construction and service life. Furthermore, moisture is associated with the typical Brunei Darussalam climate conditions (e.g. frequent heavy rainfalls, high humidity and also alternate wet and dry cycles). The construction activities usually face delays due to rainfalls. Some treatment methods use binders that may leach in the event of rainfall during construction causing hazards to the environment. This could lead to the reapplication of binder which would be uneconomical. In the pavement service life, the exposure to moisture is continuously inevitable.

Moisture is a primary cause of distress in pavement (Chen et al, 2011). Some treatment methods form base materials that are more susceptible to moisture than others, particularly the ones with no binders. Therefore, the moisture criterion was significant in choosing the suitable treatment method. It was assigned with quite high weighting factor for both road categories amongst the rest of the criteria. A moisture susceptible base can primarily cause pavement failure (Chen et al, 2011). When water infiltrates, the pavement becomes saturated. Some excess water may accumulate and become trapped in the pavement structure causing the pavement to further suffer a pumping action under repeated traffic loading. This pumping action would eventually cause the migration of fines from the matrix and de-bonding of aggregates from the binder resulting in the degradation of mechanical properties i.e loss of stiffness and strength which could lead to an unstable pavement structure. A positive score would apply to the treatment method that can be devised to be the least affected by moisture both during construction and service life. The weighting factor for urban roads was assigned a relatively higher value than the rural roads as the urban roads appear more important as they serve the major centre of activities. Moisture effect can thus have a great impact on the selection of the treatment method, as it could cause premature pavement failure and hence results in increased rehabilitation work and maintenance costs.

Aggregate degradation is an important criterion to consider in the road base construction. Treatment methods that showed the ability to take traffic loadings without or with minimal disintegration would be assigned a positive score. In unbound pavement, aggregate becomes the main structural element whereby the quality of the aggregate must be high to resist degradation under repeated traffic loading. Negative scores are assigned to the treatment methods whereby the utilisation of less durable aggregates that tends to degrade under loading would cause rapid deterioration resulting in a less durable pavement.

The ***reliability*** criterion is evaluated based on the common treatment methods or agents whereby research, design, construction guidelines and attested field performances are available. Hence, positive values would be assigned. However, their performance can be varied between the treatment methods or agents depending on their compatibility with sandstone aggregates and weather conditions. Negative scores were assigned to a treatment method or agent that is associated with uncertainties due to limited information on design, construction and poor historical field performance. The reliability criterion is scored based on the existence of guidelines and specification documents on the treatment methods that can be found. A wide of range of literature and standards on the conventional pavement construction can be found widely across the world. Few traditional treatment agents such as cement and lime have been around over decades with many established standards. For bitumen stabilised materials (i.e foamed bitumen and bitumen emulsion design), the most complete manual was the Wirtgen Cold Recycling Manual (2004), later TG2 based largely on the experiences from South Africa. Other manuals can also be found in the Transport Research Laboratory reports (Milton and Earland, 1999; Merrill, Nunn and Carswell, 2004), Australia and New Zealand standards (Austroads, 2012). Polymer stabilisation, particularly dry powdered polymer, can only be found in Australian standards,

Time and traffic dependent decay occurs when pavement materials ravel due to cracking or weathering of binders. Unless binders are strong, cement stabilised materials are more prone to such damage under traffic, as they are brittle and/or are prone to shrinkage cracking. This behaviour can be compared with that of bituminous treatments which are usually favoured due to their ability to provide flexibility. Hence, a positive score would be assigned. Bituminous mixture or asphalt is known as a time dependent material under dry and wet conditions (Cho and Kim, 2010). The weighting factor of the criterion was assigned quite high for each road category because it is relatively important to select a treatment method that would form a flexible pavement. Hence, a positive value was assigned to the scoring factor of the treatment method.

Time to build is one of the criteria that dictate the choice of treatment method. A faster time to build the road means an immediate road opening to traffic and is the most preferable situation. The delay of road opening would create more traffic chaos and increase the user's costs. The delay could be due to a long curing process in order to develop early strength particularly for mixtures containing high amounts of water. Bituminous treatment using the bitumen emulsion would not be ideal as the curing process would be longer due to the time needed to evaporate the water and thus, strength could develop slowly. The selection is biased towards a shorter curing period for immediate road openings. Rural roads mainly serve the access to the residential areas that are located a long way from the major centre of activities, main health centre, big shopping areas etc. Therefore, the time to build became a very important criterion in evaluating the candidate treatment method and the weighting factor was assigned very high, 8. The treatment method should have the ability to be constructed and opened to traffic without any necessary delays such as long curing process. Whereas for urban road, the criterion became less important due to there being many alternative routes that exist like many urban areas in the world. Moreover, the category of urban road would focus on new road construction rather than rehabilitation works. For example, bitumen emulsion agent requires a long curing process to allow for the breaking of emulsion, lime requires a long mellowing process so that it could efficiently mix with the road materials to gain strength.

Maintenance is a criterion dependent on the serviceability life of the pavement. When the pavement degrades then the serviceability becomes poor and hence, maintenance work has to take place such as patching, resurfacing or major rehabilitation. The criterion was assigned a weighting factor of 1 because it is considered less important in this evaluation relative to other criteria. Some candidate treatment methods or agents could require frequent or regular maintenance works, between three or six months, particularly using low quality aggregates for unbound base or rigid cementitious pavement. The scoring factor was assigned positive value for each candidate treatment method or agent that would anticipate less maintenance works, within 2, 5 to 10 years period after completion particularly the ones forming flexible pavement.

Sustainability has become a strategy in the implementation of many construction projects around the world. The key sustainability issue relating to this evaluation study is to use the readily available materials in the locale. The stabilisation method or agent should be compatible with a wide range of aggregates, particularly of low quality, and the product could be remixed if necessary for future use. Utilising these materials would decrease the haulage of materials from imported sources, therefore it would be a cost effective solution and at the same time preserve the existing road network. As a result, the stabilisation method or agent was assigned a positive scoring factor in this evaluation. A negative scoring factor was assigned to a candidate treatment method or agent that relates to the non-optimisation (or wastage) of aggregates, more haulage, and utilisation of non-environmentally friendly and non-local resources.

The ***ease of construction*** is a criterion that evaluates the candidate stabilisation method or agent in term of the machineries, construction processes or sequences and labourer's skills. Some of the candidate treatment methods are carried out in simple and conventional construction processes such as compaction, cement stabilisation. Therefore, positive scores are given to a candidate treatment method or agent that requires a simple and conventional process of construction. The scoring factor was assigned to negative values to candidate treatment methods or agents that require special mixing, strict moisture control or highly skilled workers or high hazard risk to the workers, public and environment and so on.

The typical plant for road construction and stabilisation works are asphalt paver, compactors, miller, bitumen tanker, water tanker, pavement profiler, spreader and conventional soil or aggregate stabiliser. Most of these plants can be found in the majority of places in the world. A treatment method or agent was assigned a high positive value if it satisfied the criterion of ease of construction due to its need for traditional plant for conventional road construction process.

However, a lower scoring factor value would be assigned to a treatment method or agent if it requires intensive construction process for example mellowing process for lime stabilisation or breaking of bitumen emulsion. One candidate treatment agent in the list, foamed bitumen, requires a state of art technology equipment for its production and application on site. Using this technology would require initial investment cost for its specialised equipment and trained skilled labours.

3.4 SCORES AND RANKING

Table 3.5 summarises the scoring factor of each criterion explaining the reasons contributing to the positive and negative scoring factor values. The overall scoring and ranking of the candidate stabilisation methods or agents are shown in Table 3.6.

From this ranking evaluation, the foamed bitumen treatment has the highest grand total score of 132.5. Therefore, it has been shown that the utilisation of sandstone aggregates as usable road base materials could be best achieved with foamed bitumen stabilisation. Apart from producing a flexible pavement, foamed bitumen was produced using the standard penetration bitumen that is one of the conventional pavement materials in Brunei Darussalam, it has compatibility to a wide range of granular materials and rapid curing process.

The second candidate in the ranking evaluation is the bitumen emulsion polymer which is rated the next highest with a score of 50. This is mainly due to its potential in producing a promising flexible pavement, a preferable type of pavement structure, similar to the foamed bitumen treatment method. However, the potential leaching of bitumen emulsion is considered a disadvantage particularly during inclement weather conditions such as rainfall during field application. In addition, the limited availability of bitumen emulsion in the local market would not lessen the country's reliance on imports.

Cement ranked the third amongst the candidates because it is a conventional stabilising agent in Brunei Darussalam. It is also known to produce high strength and water resistant mixtures. However the cement stabilised road base requires a long curing process to develop strength. Shrinkage cracks usually appear after construction and to avoid these cracks propagating to the top layers, a layer of unbound crushed aggregates is normally laid on the cement stabilised road base. In this case, the pavement may require a very thick layer.

It should be noted that the main deciding factor in this ranking evaluation was the consideration of cost by potentially using the local materials to lessen the country's reliance on imports.

Therefore, extensive laboratory experimental research proceeded with the candidate, foamed bitumen. Prior to laboratory experiments, more detailed literature survey on foamed bitumen was carried out and reviewed in the following chapter to identify gaps in the current mix design and application standards. Any weaknesses derived from the design mixtures should be improved by the inclusion of a suitable co-treatment agent.

Table 3.5 Summary of the reasons for the scoring factor in positive and negative values for each criterion in the evaluation of candidate treatment methods or agents

Criteria	**Positive (+)**	**Negative (-)**
Cost of Materials	Readily available materials	Imported materials
Moisture Effect	Less leaching of binder during construction in the event of rainfall and low moisture susceptible materials	Leaching of binder during construction in the event of rainfall and high moisture susceptibility
Aggregate Degradation	Still stable over time	Easily de-bonded materials, untreated aggregates
Reliability	Past performances indicated promising	Not widely established technology, non-standard

	solution, international establishment of guidelines and standards	methods
Time and traffic dependent decay	Flexible and ductile pavement materials	Rigid pavement materials and known to be prone to cracking
Time to build	Could be immediately open to traffic upon completion	Long delay to traffic openings due to the requirement of a long curing period
Maintenance	Expected to take place between two, five or ten years period	Anticipated for a frequent or regular maintenance works
Ease of Construction	Traditional roadwork plant, conventional road construction process.	Specialised plant, intensive construction process, highly skilled labour, high risk agents
Sustainability	Optimum utilisation of aggregates, local available resources less haulage and low energy usage	Imported materials including aggregates, treatment agents, long haulage, non-optimisation of aggregate usage

Table 3.6 Ranking evaluation and scores of candidate stabilisation methods or agents

Category			Mechanical														
Candidates			Crushing			Compaction			Gradation			Reinforcement			Water Bound Macadam		
Criteria	W.F Rural	W.F Urban	S.F	W.S	W.S	S.F	W.S	W.S	S.F	W.S	W.S	S.F	W.S	W.S	S.F	W.S	W.S
Cost of Materials	10	10	1	10	10	1	10	10	1	10	10	0.5	5	5	1	10	10
Moisture	7	8	0	0	0	0	0	0	0	0	0	0	0	0	-1	-7	-8
Aggregate Degradation	4	5	-2	-8	-10	-1	-4	-5	-1	-4	-5	-1	-4	-5	-2	-8	-10
Reliability	5	4	1	5	4	1	5	4	1	5	4	0	0	0	1	5	4
Time & Traffic Dependent Decay	6	7	-1	-6	-7	-2	-12	-14	-2	-12	-14	-0.5	-3	-3.5	-1	-6	-7
Time to Build	8	3	2	16	6	2	16	6	2	16	6	1.5	12	4.5	2	16	6
Maintenance	1	1	-1	-1	-1	-1	-1	-1	-1	-1	-1	0	0	0	-1	-1	-1
Sustainability	5	6	-1	-5	-6	-1	-5	-6	-1	-5	-6	0	0	0	-1	-5	-6
Ease of Construction	3	4	1	3	4	1	3	4	1	3	4	1	3	4	1.5	4.5	6
Total W.S				14	0		12	-2		12	-2		13	5		9	-6
GRAND TOTAL			**14**			**10**			**10**			**18**			**2.5**		

Category	Polymer						Bituminous					
Candidates	Dry Powdered Polymer			Ligno-Sulfonates			Foamed Bitumen			Bitumen Emulsion		
Criteria	S.F	W.S	W.S	S.F	W.S	W.S	S.F	W.S	W.S	S.F	W.S	W.S
Cost of Materials	-2	-20	-20	-2	-20	-20	2	20	20	-2	-20	-20
Moisture	1	7	8	-2	-14	-16	1	7	8	1	7	8
Aggregate Degradation	1	4	5	-1	-4	-5	1	4	5	1	4	5
Reliability	1	5	4	0	0	0	1	5	4	1	5	4
Time & Traffic Dependent Decay	2	12	14	-1.5	-9	-10.5	2	12	14	2	12	14
Time to Build	1	8	3	-0.5	-4	-1.5	2	16	6	1	8	3
Maintenance	1	1	1	1	1	1	1	1	1	1	1	1
Sustainability	-1	-5	-6	0.5	2.5	3	1.5	7.5	9	1	5	6
Ease of Construction	1	3	4	1	3	4	-1	-3	-4	1	3	4
Total W.S		15	13		-45	-45		70	63		25	25
GRAND TOTAL	**28**			**-89.5**			***132.5***			**50**		

Category	Cementitious											
Candidates	Cement			RHA			Limes			Fly Ash		
Criteria	S.F	W.S	W.S	S.F	W.S	W.S	S.F	W.S	W.S	S.F	W.S	W.S
Cost of Materials	0	0	0	-1	-10	-10	-2	-20	-20	-2	-20	-20
Moisture	2	14	16	0	0	0	1	7	8	0	0	0
Aggregate Degradation	2	8	10	1	4	5	0	0	0	1	4	5
Reliability	2	10	8	0	0	0	2	10	8	1	5	4
Time & Traffic Dependent Decay	-1	-6	-7	1	6	7	0	0	0	0.5	3	3.5
Time to Build	-2	-16	-6	-2	-16	-6	-2	-16	-6	-2	-16	-6
Maintenance	0	0	0	1	1	1	1	1	1	1	1	1
Sustainability	0	0	0	1	5	6	0	0	0	0	0	0
Ease of Construction	1	3	4	1	3	4	0.5	1.5	2	1	3	4
Total W.S		13	25		-7	7		-17	-7		-20	-9
GRAND TOTAL	**38**			**0**			**-23.5**			**-28.5**		

CHAPTER 4 REVIEW OF FOAMED BITUMEN MIXTURE

4.1 INTRODUCTION

The chapter presents a literature review on the application of foamed bitumen stabilisation in many countries around the world. The application aims to modify and improve low quality road pavement materials so that they can perform similarly to the conventional materials. The materials come from different origins of different geographical regions. Thus, the details pertaining to the mix design criteria of foamed bitumen mixtures are discussed. This is done to aid in proposing an appropriate strategy in the mix design protocol of foamed bitumen mixture specific to the Brunei Darussalam's condition. Finally the review focuses on the mechanical performance of foamed bitumen mixture based on laboratory and field evaluation to understand its behaviour and weaknesses that could be overcome.

4.2 MIX DESIGN CRITERIA FOR FOAMED BITUMEN MIXTURE

Foamed bitumen has been employed to treat weak soil, road base and sub-base materials in most temperate regions like Australia, Europe, United States and China and also in hot arid areas like Africa and Saudi Arabia. It has recently gained recognition in South East Asia regions like Thailand, Hongkong, Indonesia and Malaysia. However, most of its design guidelines and specification are established in South Africa, Australia, United States, United Kingdom and other European countries. Foamed bitumen mixture is made up of granular materials such as crushed rocks, recycled asphalt pavement and other industrial waste products (i.e incinerator ash), and foamed bitumen.

This section reviews the mix design criteria recommended by researchers. Based on previous research work, the type of granular materials, foamed bitumen content, the characteristics of foamed bitumen as well as the mixing moisture content are thought to be the keys to success in designing foamed bitumen mixtures.

4.2.1 Granular Materials

Foamed bitumen stabilisation has been recognised for its contributions to sustainable construction. The reason is because of the compatibility of foamed bitumen with a wide range of granular materials, which have been successfully treated with combinations of additives if required under different conditions. The materials include substandard aggregates or waste products such as marginal virgin aggregates, recycled asphalt pavement (RAP), crushed concrete from concrete pavement and other salvaged materials (Muhammad et al, 2003 and Angsanam et al, 2008). The blend of RAP and crushed concrete treated with foamed bitumen exhibits a good stiffness modulus value (Muhammad et al, 2003). Dune sands and calcretes were treated successfully with foamed bitumen and utilised as road pavement materials (Millar and Nothard, 2004). In the case of waste product such as incinerator ash, the combination of foamed bitumen and lime were proven to improve the property of incinerator ash to make it usable as road base pavement material (Mallick and Hendrix, 2004).

A study of foamed bitumen treated road bases was conducted in Saudi Arabia (Al-Abdul Wahab et al, 2012) using local marginal materials like marl and reclaimed asphalt pavement (RAP). The road base was successfully improved when compared to using the conventional aggregate road base.

In regions like Norway, the treatment of low quality natural gravel with foamed bitumen was advantageous in the summer season as they develop a favourable strength faster.

4.2.2 Foamed Bitumen Binder Content

Another important mix design criterion is the amount of foamed bitumen binder needed to produce the mixture. The high binder content would require more fines to mobilise the bituminous bonding effect. Therefore, the optimum foamed bitumen binder content must be determined so that the best mix can be obtained. The guidelines (Asphalt Academy, 2009 and Muthen, 1999) recommended that the optimum foamed bitumen binder should be selected based on the relationship between Indirect Tensile Strength (ITS) and foamed bitumen content. Some researchers determined the optimum binder contents in terms of stiffness modulus under both dry and wet curing conditions. The optimum binder content of foam mix is selected at its maximum stiffness modulus (Huan et al, 2012). It is important to note that no consistent values or trends could be identified for the variation in foamed bitumen binder content (Jitsangiam et al, 2012).

4.2.3 Characteristics of Foamed Bitumen

Foamed bitumen forms instantaneously by injecting hot bitumen and a small amount of ambient water into an expansion chamber. The chamber is where the conversion of ambient water into steam occurs due to the hot bitumen and hence, the bitumen foams and is injected into the mixer. In this foamed state, the bitumen is able to mix with aggregates without heating them. However, the foam tends to collapse quickly hence, a vigorous mixing is required to disperse the foamed bitumen throughout the aggregates. Hence, it is essential to determine the characteristic of foamed bitumen, that measured by two key parameters, expansion ratio and half-life, which are defined below (Wirtgen, 2004 and Asphalt Academy, 2009) and illustrated in Figure 4.1:

- Expansion ratio is determined by the ratio of the volume of foamed or expanded bitumen at its maximum volume to the volume of initial fresh bitumen at its original volume. High expansion ratio of foamed bitumen indicates that it is at low viscosity producing more workable liquid to coat the aggregate particles particularly the fines particles
- Half Life is the time taken from when the foamed bitumen reaches its maximum volume to when its volume collapses to half. It represents the stability of foamed bitumen providing effective surface interactions between the aggregate particles.

In this case, the optimum characteristics of foamed bitumen would be at a point where a set temperature of bitumen and amount of water injected would give the maximum possible expansion ratio and at the same time, the longest possible half-life of the foamed bitumen.

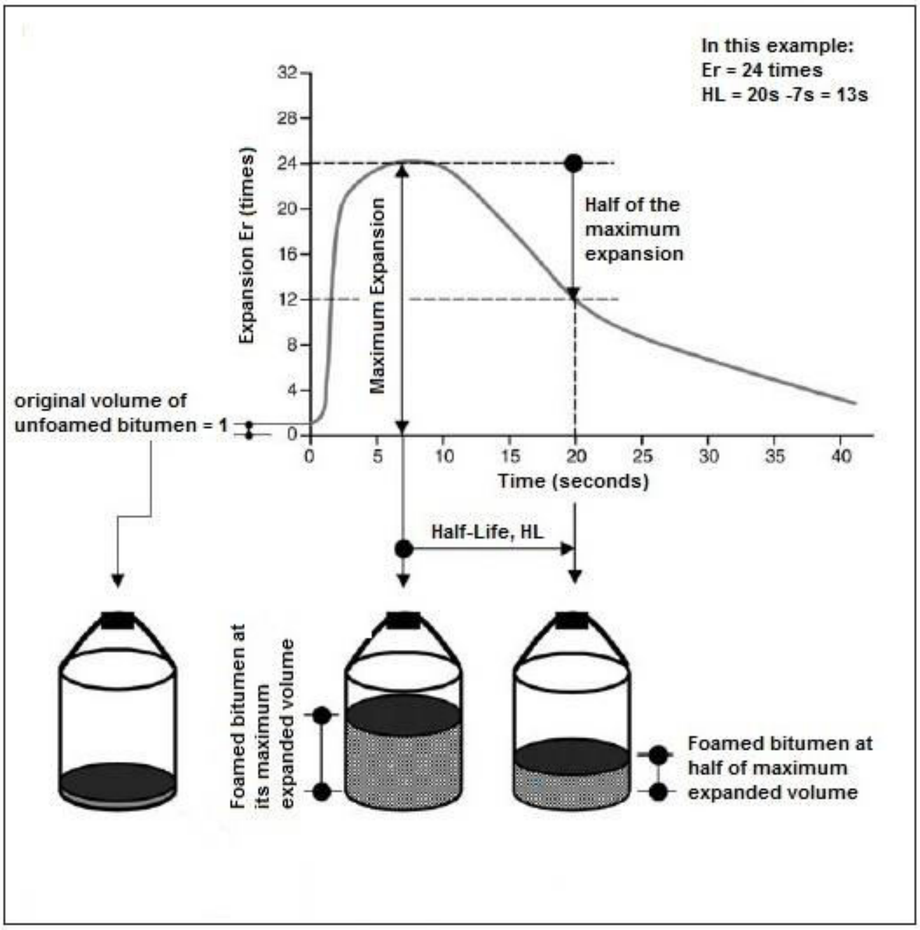


Figure 4.1 Illustrative graph showing the measurement of Maximum Expansion Ratio and Half-Life (Wirtgen, 2004 and Sunarjono, 2008)

The recommended minimum values of these two key parameters to achieve the optimum foamed bitumen characteristics are as shown in Table 4.1. They are influenced by the type of bitumen and the amount of water injected into the hot bitumen. An increase in either the amount of water injected or the bitumen's temperature normally increases the expansion ratio but simultaneously reduces the half-life of foamed bitumen (He & Wong, 2005; Wirtgen, 2004; Brennen et al, 1983). This can be attributed to more water being available, thus more steam being produced, which leads to the formation of more bubbles. This results in an increase in the expansion ratio of the foamed bitumen.

Table 4.1 Minimum values recommended for foamed bitumen characteristics (Wirtgen, 2012)

Foamed Bitumen Characteristics	**Values**	
Aggregate Temperature	10°C to 15°C	>15°C
Expansion Ratio, E_R (times)	10	8
Half-Life, HL (seconds)	8	6

However, if the foamed bitumen has low optimum values of E_R and HL, it can still be used providing the temperature of the aggregates is more than 15°C as shown in Table 4.1. Aggregate temperature can influence the bitumen dispersion during the mixing process, therefore warm aggregates would allow for stronger coating to the bitumen (Van de Ven et al, 2007; Jenkins, 2000 and Gaudefroy et al, 2007).

4.2.4 Mixing Moisture Content

Mixing moisture content (MMC) is the amount of water to be pre-added to the aggregates prior to mixing them with the foamed bitumen. It is an important mix design criterion for the mixing process of foamed bitumen mixtures because of the following reasons (Jenkins, 2000):

- It acts as a medium for the distribution of foamed bitumen in the mixtures by separating the aggregate particles particularly the fines and creating pathways for binder to distribute within the mix

- It aids in achieving compaction so the moisture would allow the aggregate particles to move to rearrange their packing until reaching their target densification.

MMC is known to influence the mixing workability of the foamed bitumen mixture hence, its mechanical properties. When the mixing moisture content is less than the optimum, the mixing workability would be impaired resulting in mixes with discrete bitumen globules and less mastic being formed. When MMC is high or exceeds the optimum amount, the development of bitumen and aggregate bonding would be hindered. This is due to the agglomeration of the aggregates reducing the surface area per mass hence they become less exposed to the bitumen coating (Fu, 2009). A long curing period would be required to remove the moisture or otherwise the foamed bitumen mixture would be prone to premature distress.

The MMC is in the range of 65% to 85% of the OMC of untreated dried aggregates (Lee, 1981). Sunarjono (2008), for his mix design, calculated an MMC equal to 70% of OMC, in his mix design using the equation formulated by Wirtgen (2004) as follows:

$$W_{add} = 1 + W_{OMC} + W_{air-dry} \qquad \text{Equation 4.1}$$

where

W_{add} = water to be added to sample or mixing moisture content (% by mass)

W_{omc} = optimum moisture content (% by mass)

$W_{air\text{-}dry}$ = water in air-dried sample (% by mass)

The rational range of MMC for the production of foamed asphalt mix recommended from a recent study is as presented in Table 4.2 (Xu et al, 2012). It was evaluated by the effect of the percentage fines content where the optimal MMC increased with more fines addition. However, the rational range of MMC is generally more than 70% of OMC.

Table 4.2 Recommendations of MMC for foamed asphalt mix (Xu et al, 2012).

Fines Contents	**Rational Range of MMC (% of OMC))**	**Optimal MMC (% of OMC)**
5 – 10%	70-80	75
10-15%	70-80	80
15-20%	75-85	80

4.2.5 Other Factors

Method of mixing is essential to produce a good foamed bitumen mixture that is durable. Sunarjono (2008) studied the effect of mixing method on stiffness modulus of foamed bitumen mixture. Better binder distribution in the foam mix was observed when using a flat agitator (or K-beater) resulting in higher stiffness modulus value than the ones using a dough hook produced at different foaming water contents as shown in Figure 4.2. Having a single hook, the distribution of the binder within the mix is limited and on the other hand, a flat agitator might cause degradation of coarse aggregate particles, larger than 10mm (Sunarjono, 2008). Hence, it is important that the employed method of mixing has to the same throughout the research for consistent evaluation.

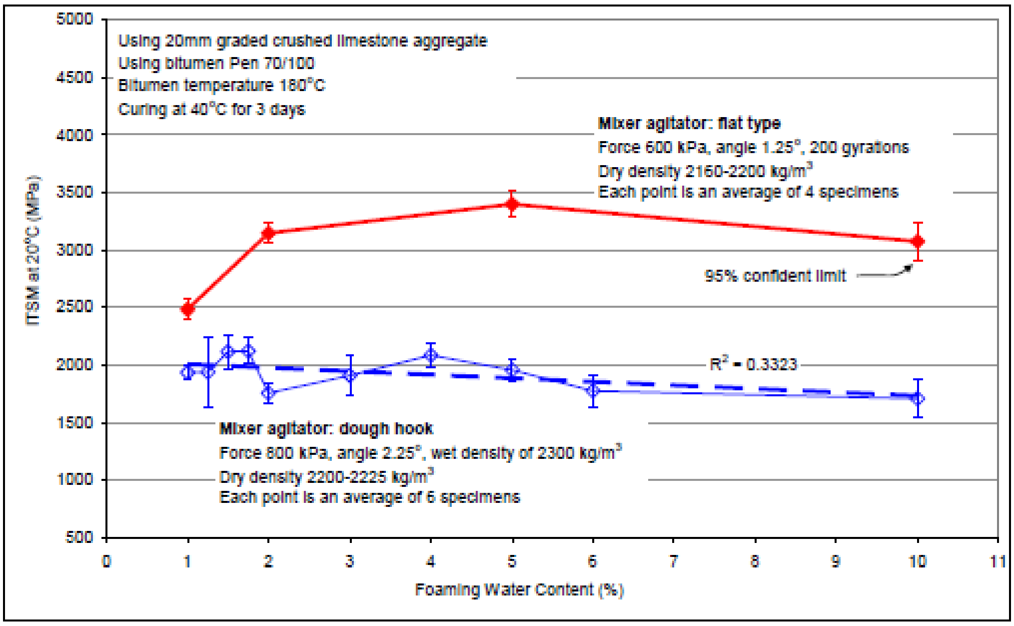


Figure 4.2 Stiffness modulus of foamed bitumen mixtures produced using flat agitator and dough hook types

Aggregate gradation is an important factor influencing the mechanical properties of foam mix. TG2 guideline (Asphalt Academy, 2009) provides the suitable gradation envelope for granular materials to be stabilised with foamed bitumen. Discontinuous gradation is not suited for the production of foamed bitumen mixture (Chao-Hui W. et al, 2011).

The foamed bitumen's affinity for finer aggregate particles makes this fraction primarily important in the composition of foam mixes unlike the coarser aggregate fractions. The coarser aggregates become hydrophobic to the foamed bitumen binder due to the rapid steam condensation, when they come in contact with the foamed bitumen bubbles, forming water onto the aggregate surfaces (Namutebi, 2011). In case of utilising RAP, the combination of RAP and dust stones treated with foamed bitumen exhibit better performance than the ones with RAP only (Angsanam et al, 2008). A minimum of fines (passing 75 microns) of between 5% and 20% is recommended (Asphalt Academy, 2009 and Wirtgen, 2004) whilst Chao-hui et al (2011) found that a fines percentage of between 5% and 12.5% proved to have better mechanical properties. The fines attach to the binder forming the foamed asphalt mastic that holds the coarser aggregate particles together.

4.3 MECHANICAL PERFORMANCE OF FOAMED BITUMEN MIXTURES

Foamed bitumen mixes are mainly dependent on aggregate interlock forces whereas hot asphalt mixes gain mechanical performance by the combination of the mastic cohesion and interlocking forces of the aggregates (He and Wong, 2007).

This section reviews previous research undertaken to evaluate the mechanical performance of foamed bitumen mixtures in terms of stiffness modulus, permanent deformation and fatigue resistance. In turn, the aforementioned measures of performance are reviewed to attain a better understanding of the effect of the different mixture variables on foamed bitumen mixtures. The mixture variables discussed include the mixing process, aggregate gradation, temperature, bitumen type and some additives.

Conceptually the behaviour of foamed bitumen mixture which is categorised as a cold bituminous mixture can be explained diagrammatically in Figure 4.3 (Asphalt Academy, 2009). The mechanical behaviour is evaluated based on the influence of the amount of bitumen and cement contents in the mixture. The resistance to moisture can be significantly improved with an increase in the bitumen content. However, the addition of high bitumen content would not serve the purpose of foamed bitumen treatment being economical. Although the mix would increase the flexibility of the pavement, it would develop deformation rapidly. On the other hand, the addition of cement in the foamed bitumen mixture would resist the deformation. But the excessive addition of cement content in the mix would turn it into a rigid cementitous mix that is prone to cracks because the cement would become the dominant binder.

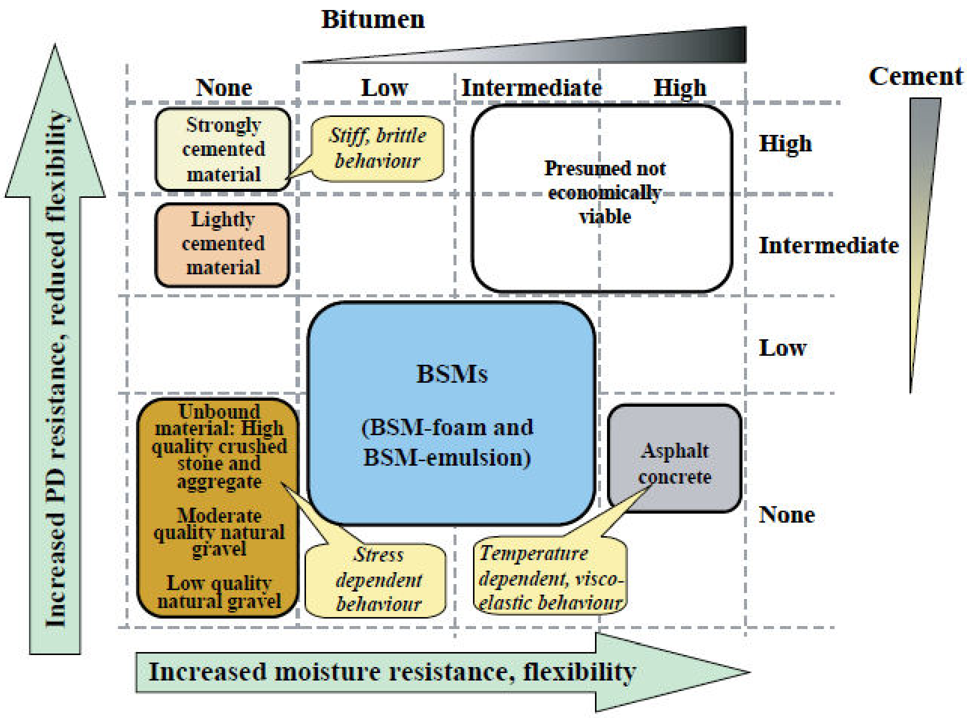


Figure 4.3 Conceptual behaviour of cold bituminous mixtures (Asphalt Academy, 2009)

In this case, it is deemed important to carefully evaluate the mix design of foamed bitumen mixtures particularly if the condition where it would be applied and the origin of material are different from the established concept as shown in Figure 4.3.

4.3.1 Stiffness Modulus

Since the study is interested to investigate the benefits of foamed bitumen application for a road base layer, therefore, stiffness modulus is an important property to be evaluated as it is influenced by the materials utilised in the mix and climatic factors. Stiffness modulus is also an essential input for the mechanistic pavement design. A stiff road base layer would result in a better load distribution and thus, it reduces the stress on the bottom layer such as subgrade which is normally made up of low quality materials. It also helps to reduce the likelihood of fatigue cracking on the top layers as the stress from the top layer is distributed to the road base layer.

Numerous studies of the stiffness modulus of foamed bitumen mixture have been conducted and evaluated using various testing methods such as Flexural Bending test, Tri-axial test and Uniaxial test under different test conditions and control parameters such as frequency, stress level and confined pressures. The choice of testing methods is governed by the availability of the test equipment and the objectives of the study. In this study, the Indirect tensile stiffness modulus (ITSM) test would be used to determine the stiffness modulus (see Chapter 6).

As one of the design criteria, the influence of foamed bitumen content can be seen in Figure 4.4 on the stiffness modulus of the mixture. Addition of a small amount of binder to produce a foamed bitumen mixture would result in a low stiffness value due to only a thin film of bitumen being available to coat the aggregates. As the binder content increases, the mix reaches its maximum stiffness modulus then eventually reduces due to high binder content that leads to high deformation capacity.

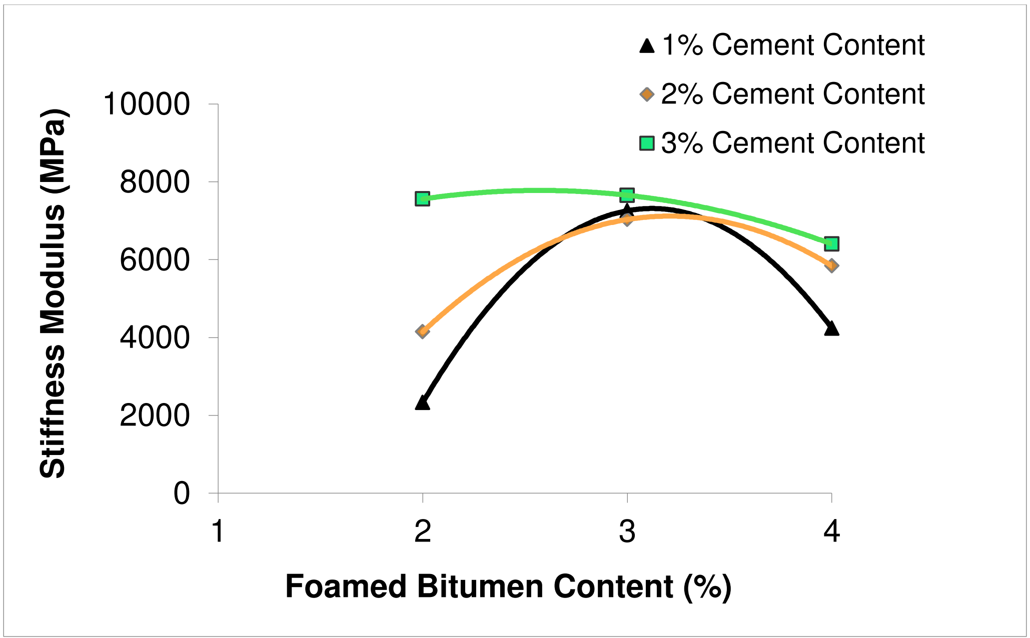


Figure 4.4 Stiffness Modulus against the foamed bitumen content (Bocci et al, 2005).

The presence of water in foamed bitumen mixture made its early state vulnerable to damage due to the low stiffness property (Sunarjono, 2008). In this case, temperature and curing are the influential factors in the development of stiffness modulus. The stiffness modulus develops over time as it cures allowing the loss of moisture. Foam mix has been found to have a better stiffness than that of unbound materials (Muhammad et al, 2003) as it cures. Curing temperature in the laboratory should be lower than the softening point of the bitumen. Unfortunately this is difficult to control in the field due to many environmental factors.

The development of stiffness modulus of foam mix with additives, such as cement and lime, can be seen in Figure 4.5 over the curing period at a temperature of 20°C (Jitareekul, 2009). The hot asphalt mix reached a stiffness modulus of 5000MPa approximately one month after it was laid. With hydrated lime, the foam mix experienced a constant stiffness modulus whereas the cement additions in the foam mix, 1% and 2%, showed the same rate of increase in stiffness modulus of foam mix (Jitareekul, 2009). The foam mix without the additives has the slowest development of stiffness modulus in which the fully cured stiffness modulus was about 2500MPa. The development of bonds between the foamed asphalt mastic and aggregate from the mixing process, compaction and through the curing process can be conceptually illustrated in Figure 4.6 (Fu et al, 2010).

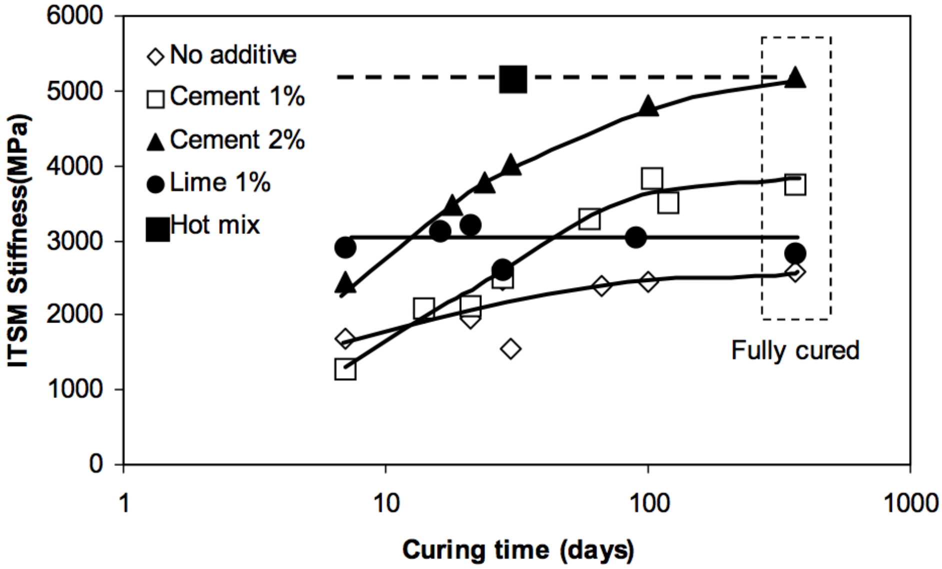


Figure 4.5 The stiffness modulus of foam mix with cementitous additives and hot asphalt mix (Jitareekul, 2009)

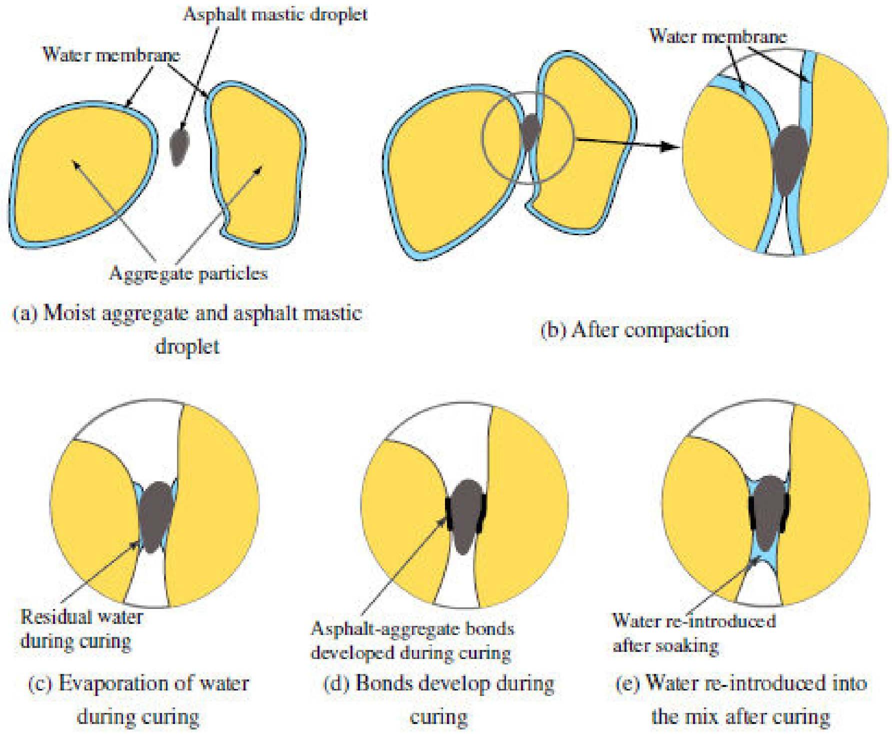


Figure 4.6 Conceptual illustration of the curing process for foamed bitumen mixture (Fu et al, 2010)

In foamed bitumen mixture, the mechanical performance, stiffness and plastic deformation, can be gained from unbound aggregate particles with high angularity (Li et al, 2011). Angular and rough materials improve the interlocking and shear resistance property between the aggregate particles (Pan et al, 2006) that contributes to the high stiffness modulus.

Although bituminous mixture is known to be sensitive to temperature particularly hot asphalt mix, this is not the case with foamed bitumen mixture (Fu and Harvey, 2007; Li et al, 2011). The effect of temperature on foamed bitumen mixture is minimal because of the lreduced bituminous bonding effect (Li et al, 2011). The change in temperature could only affect the stiffness at the bituminous mastic points within the mix whereas the remaining unbound aggregate skeleton is not sensitive to temperature changes as it is mainly dependent on the interlocking properties.

4.3.2 Fatigue

Fatigue can be defined as “the phenomenon of fracture under repeated or fluctuating stress having a maximum value generally less than the tensile strength of the material” [Whiteoak, 1991].

Its mechanism can be analysed by the law of displacement developed in three stages (Yan et al, 2010): a rapid displacement stage, a long plateau with low levels of displacement or an elastic zone; and a fatigue stage, or a plastic zone. Yan et al (2010) also studied the fatigue behaviour of both bitumen emulsion mix and foam mix. The bitumen emulsion mixes failed in three stages; however, the foam mix failed in only two stages indicating the material is relatively brittle similar to the semi rigid material’s behaviour as shown in Figure 4.7 (Yan et al, 2010).

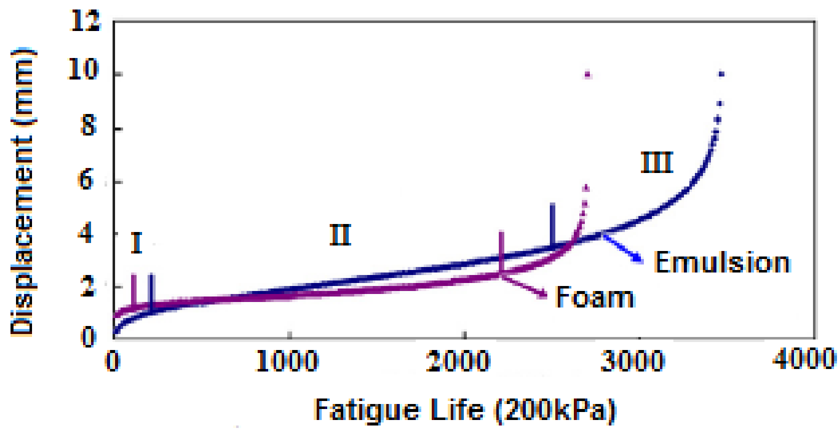


Figure 4.7 Displacement-Load cycles (200kPa)

The fatigue life of foamed bitumen mixtures is shorter at a given strain level than the fatigue life of hot asphalt mix as presented in Figure 4.8 (Sunarjono, 2008) in which the hot asphalt mix's result was taken from Read (1996). It showed that at 200 microstrain, the fatigue life of foamed bitumen was 900 cycles whereas it took 30000 cycles for the hot asphalt mix to fail. However, the fatigue performance of foamed bitumen mixture containing Recycled Asphalt Pavement (RAP) was found to be similar to that of hot asphalt mixture and to cement treated materials (He and Lu, 2004). This was due to the presence of residual bitumen in the RAP that contributes a thicker bitumen film coating on the aggregate particles to resist fatigue cracking. Also utilising marginal aggregate for foamed bitumen mixture could compromise its fatigue life, well below the good quality of emulsion stabilised materials (Little et al, 1983).

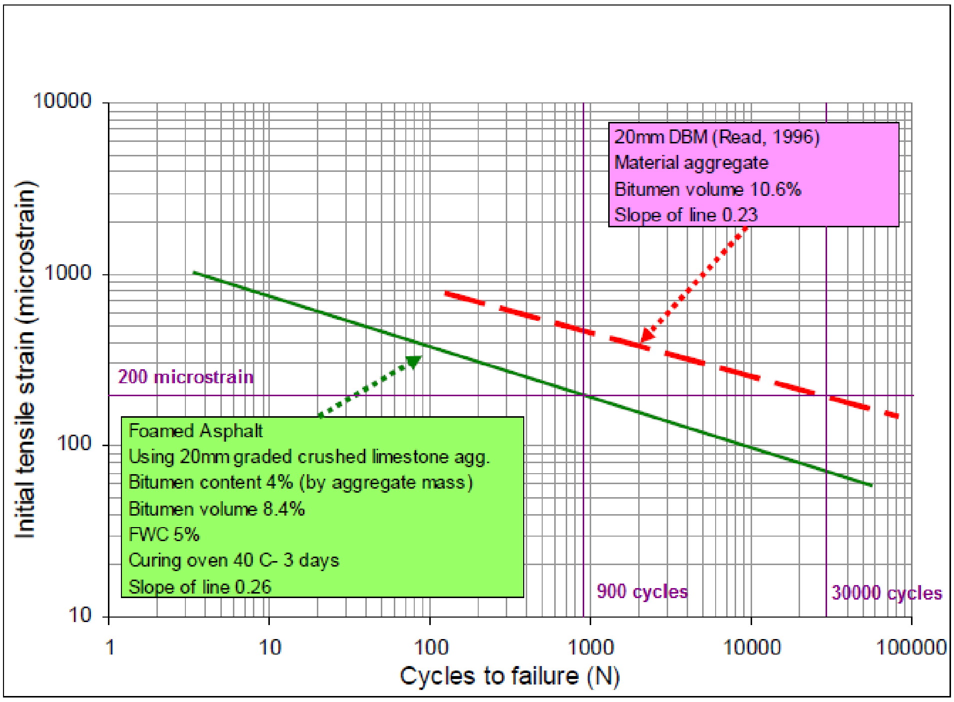


Figure 4.8 Fatigue lines of foamed asphalt and hot asphalt mix (Sunarjono, 2008)

The plotted graph in Figure 4.9 (Sunarjono, 2008) clearly shows that the fatigue property of foamed bitumen mixtures is not significantly influenced by the different foaming water contents (FWC).

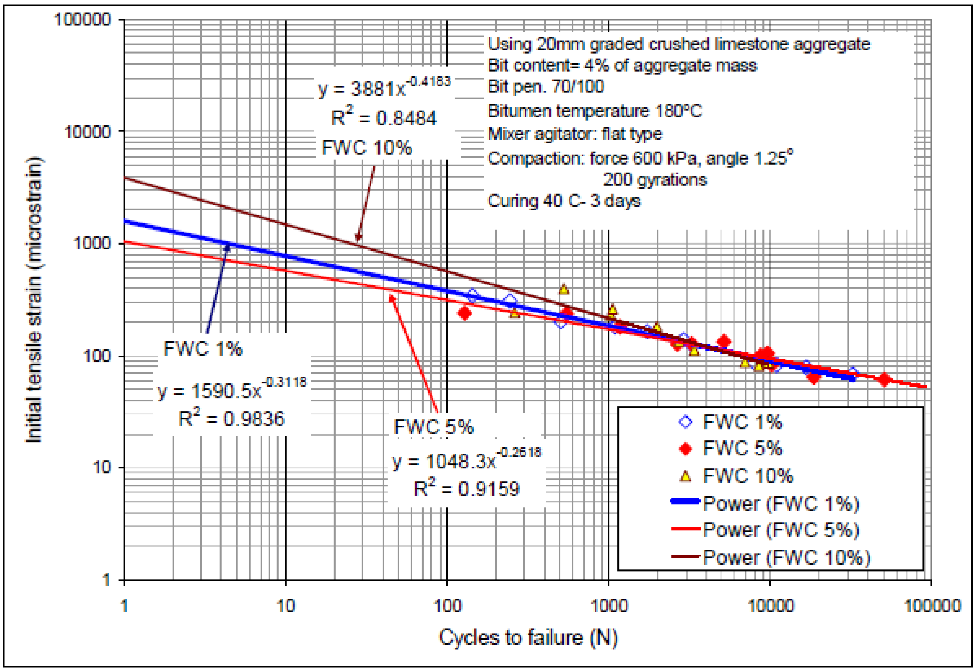


Figure 4.9 Fatigue characteristics of foamed bitumen mixtures based on strain for specimens produced at FWC of 1%, 5% and 10% (Sunarjono, 2008)

4.3.3 Permanent Deformation

Permanent deformation can be determined from the accumulation of permanent strain induced by repeated traffic loading. It can manifest in the form of rutting, surface deformation along the wheel tracks of the road carriageways. It is one the distress modes experienced by foamed bitumen mixtures (Vorobieff, 2005).

It can be greatly influenced by the content of the foamed bitumen in the mixture. Excessive bitumen content can lead to act as a lubricant that pushes the aggregate particles apart reducing the internal friction between them causing permanent deformation failure (Li et al, 2011).

The addition of RAP in foamed bitumen mixture increases its susceptibility to permanent deformation as compared to a hot asphalt mix (He and Wong, 2007). An increase in RAP content resulted in lower rutting resistance (Jitareekul, 2008). However, high rutting resistance could be experienced by foamed bitumen mixture utilising crushed aggregates with no RAP content (He and Lu, 2004)

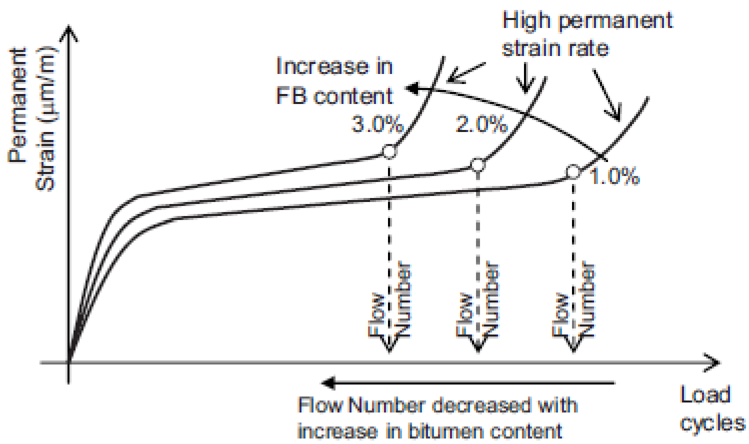


Figure 4.10 Permanent strain of Foam mix without active filler (Kim et al, 2008)

“Flow number” has been defined as the number of load cycles where the permanent strain starts to increase at a high rate. It was used by Kim et al (2007) to indicate failure of mix specimen as the foamed bitumen content increased as shown in Figure 4.10.

On a road with low traffic volume, the rutting resistance is greatly controlled by the mix design proportion of foamed bitumen as well as the type of bitumen being utilised particularly in its early life (Sunarjono, 2008). However, at high traffic loading, the rutting resistance is mainly controlled by the interlocking forces of the aggregate particles.

4.4 MOISTURE DAMAGE

Road pavements are exposed to many environmental factors such as temperature, air and water contributing to their deterioration. Among them water can be regarded as the key element that profoundly effects the durability of pavement materials because of its frequent occurrence in many climates. Moisture damage in asphalt mixtures can be defined as the loss of strength, stiffness and durability caused by the action of water that fails the bond between the binder or mastic and the fine and coarse aggregates (Airey et al, 2008). South African Interim Technical Guidelines TG2 (Asphalt Academy 2002) points out that aggregate particles are only partially coated in foamed bitumen mixes thus, moisture susceptibility is an important consideration. The presence of moisture in pavement layers has been regarded as the principal cause of their failure.

When water infiltrates the pavement, the damage due to moisture turns irreversible particularly when this moist pavement is in service under traffic. The progression of damage can be further influenced by the road pavement materials being used particularly when utilising low quality aggregates. Where good quality aggregates are utilised, the deterioration can be mainly caused by excessive traffic loadings and, therefore, may occur later in the pavement service life. However, regardless of the materials, a greater risk of rapid deterioration may occur particularly when climatic factors take their toll such as frost, floods and frequent rainfall.

In the tropical equatorial climate regions, moisture-induced pavement distress can be manifested as the follows (Fwa, 1987):

- Wet areas on roads leading to the formation of potholes or depressions on the surface
- Upward heaving of the pavement surface
- Separation and disintegration of various pavement layers or successive lifts of a given pavement layer such as ravelling
- Unevenness or undulation of the pavement surface

The physical nature of the foam mixes, having an unbound element in the coarse aggregate skeleton makes them more susceptible to moisture attack as compared to hot asphalt mixes. The various interactions of different phases in the microstructure of foam mix as mentioned in section 4.3 make it inherently sensitive to moisture (Jenkins, 2000 and Fu et al, 2010). The mechanical strength of the unbound part of the coarse aggregate skeleton depends of the frictional strength of the particles (Dawson and Kolisoja, 2004). In this case, by effective stress laws, the presence of water would lead to a reduction of the mechanical strength. The water gets further pressurised under traffic loading, reducing the contact stresses between the aggregate particles. This makes the mix more susceptible to moisture as compared to a hot asphalt mix. However, when the bonding provided by the foamed bitumen is fully developed, the occasional reintroduction of water can only partially damage the bond but if it is subjected to a long soaking period along with repetitive traffic load, the bond might be severely damaged (Fu et al, 2009). Utilising low quality aggregates may further impair the performance.

Engelbrecht et al (1985) found that foam mixes lost 50% of their strength when tested in a wet condition as compared to their strength when they were dried. The level of binder content could easily influence the moisture susceptibility of the foamed bitumen mixtures. An increase in foamed bitumen content would produce a mixture less permeable to water and less easily damaged by water's presence (Brennen et al, 1983) due to the low voids. However, foamed bitumen mixtures produced with low binder content could generate high void contents (Iwanski et al, 2014) and would be prone to damage by the action of water. However, generation of voids in the foamed bitumen mixture can be influenced by the type and size of the aggregate fraction which contributes to the adequate filling of the voids to ensure water tightness. Utilising recycled lean concrete road base for the production of foamed bitumen mixtures could generate low voids as compared to reclaimed crushed stone road base materials (Iwanski, 2014).

The infiltration of water together with high frequency loading of the foam mix would be likely to lead to extensive failure if the foamed bitumen content is insufficient or not close to optimal. Hence adequate foamed bitumen content is essential to effectively reduce moisture susceptibility even when cement is added.

4.4.1 Aggregate-Bitumen Interactions

Generally foam mixes provide excellent moisture resistance. However, different bitumen sources and grades can lead to different adhesion characteristics resulting in different retained modulus for every foam mix (Saleh, 2007). Thus, differing water resistance can be expected for different bitumens. The adhesive bond between the bitumen and aggregate is significantly affected by the physical and chemical properties of the aggregates at micro-scale (Bhasin and Little, 2006). The moisture susceptibility of foam mix can be influenced by the quality of aggregates being utilised. Basic aggregates such as limestone are easier to coat with bitumen than acidic aggregates (e.g, siliceous aggregates like sandstone and granite aggregates) containing a high silica content which have high concentrations of hydroxyl groups with greater affinity for carboxylic acid and water (Hefer and Little, 2005).

4.4.2 Treatment in Current Practise

Anti-stripping agents or active fillers have significantly evolved in the improvement of the moisture resistance of both hot asphalt mixtures and also cold bituminous mixtures.

Hydrated lime (HL) is normally used as a co-treatment agent in the application of foamed bitumen stabilisation in Australia (Vorobieff and Preston, 2004; Ramanujam et al, 2009). It is one of the active fillers that turns aggregates hydrophobic, generating effective adhesion with bitumen better than the anti-stripping agents do (Castedo et al, 1985). However, the effect becomes minimal when adding it in foamed bitumen mixture utilising crushed limestone aggregates due to the limestone's inherently strong adhesion to bitumen (Jitareekul, 2009). The stiffness modulus of foamed bitumen stabilised limestone mixture, with and without lime, is almost the same. When lime was added to pit run gravel and out wash sand (less suitable foamed bitumen materials) the resulting foam mix was stable even under vacuum saturation (Castedo et al, 1985). Incinerator ash in combination with lime additive (< 2%) was able to enhance the properties of foamed bitumen mixture (Mallick and Hendrix Jr, 2004).

Cement is known to improve the mechanical performance of foamed bitumen mixture more than any other filler such as lime, even if added in much smaller quantities (Huan et al, 2011, Hodgkinson and Visser, 2004). It generates hydration by reacting with water to form hydrated products such as C-S-H, CaOH and ettringite crystals. Hence, the hydration process beneficially helps to develop the early strength (Fu et al, 2008) that is most favoured by road engineers in order to open a newly built road to traffic without long delays. Delays have a great impact on the road users particularly due to long diversion and traffic congestion that lead to high fuel consumption.

The addition of cement also increases the fines fractions in the foamed bitumen mixture that promotes better stabilisation (Ruckel et al, 1983). It results in an increase in stiffness modulus (Jitareekul, 2009) and retains 80% of its dry stiffness modulus in wet condition. Generally foam mix with cement additive shows a good resistance to rutting failure as compared to foam mix with no cement, unbound materials and a cement-only treated mix (Gonzalez et al, 2009).

4.4.3 Coir Fibres as Reinforcment in Road Materials

Fibres have been used to reinforce concrete mixtures for decades. A review of literature revealed that various laboratory investigations have been conducted on natural fibre-reinforced materials for road construction. These investigations were limited in their scope to sand materials (Santoni and Tingle, 2001, Sarbaz and Ghiassian, 2007). In recent years, the study evolved by investigating the use of natural fibres in hot bituminous mixtures (Subramani, 2012, Thulasirajan and Narashima, 2011).

Some fibres are expensive to obtain, such as synthetic and steel fibres, and some can be obtained naturally, such as coir fibres, which are cheap. The advantage of natural fibre is that it is unpolluting, easily available and cost effective. The abundant quantities of coir fibres found mainly in South East Asia such as Philippines, Malaysia, Indonesia and Thailand would generate benefits if they were used as a form of ecological recycling.

Coir fibre is a common waste material obtainable from coconut husk. Natural fibres are generally lignocellulosic in nature, consisting of helically wound cellulose micro fibrils in a matrix of lignin and hemicellulose. Currently it is introduced in Brunei Darussalam to resolve the slope failure issue. It is sourced from a neighbouring country in an attempt to minimise the abundance of waste materials in the region for use as a form of ecological recycling.

As it is anticipated that there will be appreciable degradation of sandstone particles, and coir fibre could be introduced to overcome it. It has high tearing resistance and retains its property to some extent in wet weather conditions (Sivakumar Babu and Vasudevan, 2007). Although it is a biodegradable material, it can be a feasible solution in applications where it is meant to serve only during the initial stage before the strength develops to the ultimate (Subaida et al, 2009).

Fibres have also been found to improve the ductility behaviour of material particularly cementitious materials such as lime, therefore reducing shrinkage cracks (Ramesh et al, 2010). However, the strength of the reinforced materials depends on the distribution or orientations of the coir fibres. Randomly distributed fibres in a sand mixture exhibit higher strength than a mix reinforced by layered reinforcement (Venkatappa Rao et al, 2005). The same author also stated that long fibres are more effective than short fibres.

4.5 FAILURES OF FOAMED BITUMEN APPLICATION

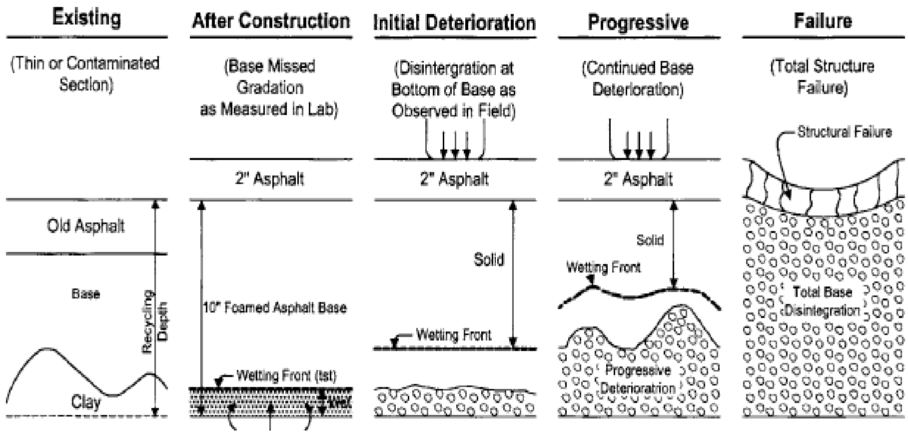


Figure 4.11 Failure mechanism of foamed bitumen mixture in road base application (Chen et al, 2006)

Chen et al (2006) conducted a forensic study on the cause of structural distress, mainly alligator and deep rutting, found in foam mix road base in Texas. Figure 4.11 hypothetically illustrates that when water infiltrates into the foam mix road base, the base layer became two distinct materials interfaced by a “Wetting front”. As the deterioration of the road base continued, it was no longer sufficient to withstand traffic loading. Ravelling also appeared to be an immediate problem after placement that might be due to the low binder contents, inadequate compaction and soft subgrade (Wijk and Wood, 1983). Careful monitoring ofconstruction variables and designing the mixture to suit local moisture conditions are vital to avoid premature distress.

4.6 SUMMARY

Owing to the wide distribution throughout most of the continents with different climatic conditions, the property of foamed bitumen as a stabilising agent is still dependent on the specific climatic condition under which the mix design is formed. Hence, the previous studies still have some limitations warranting additional study particularly in tropical equatorial climatic regions where frequent intense rainfall is very significant in combination with uniformly warm temperature of 30°C throughout the year.

The different strategies and approaches in the mix design of foamed bitumen mixture are influenced by the choice of materials, local practice and conditions. Therefore, the mechanical properties of the foamed bitumen mixture including the stiffness, tensile strength and rutting susceptibility obtained by researchers are varied due to many reasons that can be summarised as below:

- The utilisation of a wide range of granular materials depending on the geographical availability and sources such as RAP, granite, limestone, greywacke and so on.
- Variation curing conditions according to the geographical regions
- Different mixing, compaction process and testing methods

The large component of silica in sandstones may cause the foam mix to exhibit high moisture sensitivity because of the reduced chemical affinity of carboxylic acids in the bitumen to the surface of siliceous aggregates. In a region where heavy and intense rainfall is frequent, water can be considered as the main concern, having the most significant effect on the durability of foam mix. Therefore, a moisture sensitivity test is an important parameter in this laboratory evaluation study to investigate the potential application of foamed bitumen.

The selection of sub-standard or low quality granular materials to construct strong and durable pavement has the advantage of lower cost as compared to the hauling of imported materials which is the highest cost component. However, the trade off in cost savings by utilising sub-standard materials can be poor structural performance therefore, the laboratory evaluation of foamed bitumen mixture must take into accounts factors such as the significant climatic conditions and granular materials.

Although a wide range of granular materials have been utilised for the production of foamed bitumen mixtures with a history of successful performance, regardless of the low quality aggregates such as natural gravel, reclaimed asphalt pavement (RAP) and crushed concrete of different origin, it should be noted that the properties of the foamed bitumen mixture produced in the laboratory and field may differ (Angsanam, 2008).

Mixing method can influence the distribution of foamed bitumen binder hence, it is important to select the best suitable and available equipment to obtain a consistent mix throughout the study.

CHAPTER 5 ANALYSIS OF MATERIALS

5.1 INTRODUCTION

This chapter describes a simple analysis carried out on the main pavement materials, particularly the aggregate and bitumen. The aim was to select the types that closely match those conventionally used for road construction in Brunei Darussalam. The analysis methodology will be explained in the early sections. In the later sections, the potential co-treatment agents are considered and described briefly.

5.2 AGGREGATES

Sandstone was identified as the common rock type found in Brunei Darussalam as mentioned in Chapter 2. As was presented in Chapter 1, the aim of this research study was to optimise the utilisation of this local sandstone aggregate by means of a treatment method or stabilising agent to make it a usable road base material. Due to the transport and availability difficulties encountered when trying to utilise the local sandstone aggregates, it was decided to search for the closest match from sandstone aggregates available in the United Kingdom. In this case, the following requirements were selected and only a sandstone that satisfied these requirements was considered suitable for this study:

- They had to be the most similar to the properties of Brunei Darussalam's sandstone, or possibly have even poorer qualities, in order to be sure that the results obtained using the 'substitute' aggregate would represent a 'worst case'.

- The U.K sandstone aggregate gradation has to be designed to fit closely with the typical Brunei Darussalam's sandstone aggregates gradation.

- The designed gradation in compliance to the current Brunei Darussalam general specifications for flexible pavement, GS1 (CPRU, 1998) and pavement stabilisation GS7 (CPRU, 1999) must be achievable for road base aggregate materials.

5.2.1 Selection

In the U.K, sandstone quarries are more active in extracting dimensional stones for building walls, monuments and slab paving; their quality is generally considered too poor to use them as road aggregates. The selection of suitable quarries is based on the choice of the 'best' sandstone by looking at the supplied technical physical data. At the initial stage, sandstone from Barnhill quarry in the Forest of Dean, Gloucestershire, U.K was selected as a best match with the Brunei Darussalam type.

This sandstone originates from the Rhondda beds of the Pennant series of Carboniferous sandstone. It is a fine-grained type with predominantly blue grey colour with some brown sandstone tones. However, such large dimensional stones produced by Barnhill quarry would have been hard, if not impossible, to crush, without the right machinery thus, it was not selected for this study. Instead, Craig-yr-Hesg Quarry was recommended. Craig Yr Hesg Quarry is actively producing aggregate from a similar deposit to the one found in Barnhill Quarry for road surfacing works (high friction aggregate). It is situated at Pontypridd, Swansea in South Wales.

For the purpose of this assessment, only the two most representative quality properties were assessed namely the strength and wear of the stone rock. Furthermore, the constraint of time and limited mass of aggregate samples did not permit to conduct all the test properties for road aggregates. Hence the basic rock tests were carried out as follows:

5.2.1.1 Uniaxial Compressive Strength (UCS) Test

The strength of the sandstone sample was determined by the Uni-axial Compressive Strength (UCS) test. UCS is one the basic parameters of rock strength.

5.2.1.2 Cerchar Abrasiveness Index (CAI) Test

The CAI test, which is a common test used to determine the abrasiveness index of a stone, was adopted to measure the degree of fragmentation by taking the mass of the loss grains of a scratched stone sample.

The testing equipment consists of a sharp conical steel pin of cone angle 90 clamped in a holder which is subject to a 70N (7kg) dead load as shown in Figure 5.1. In this test, the pin is slowly drawn across the surface of the rock sample with a special sliding mechanism to cause a scratch against a rock specimen for a distance of 10mm as illustrated in Figure 5.1. The diameter of the worn flat area of the pin was measured using a micrometer reading method under a light reflected microscope as shown in Figure 5.2. Five readings were taken for each rock sample and the average value is defined as the Cerchar Abrasiveness Index (CAI).

Since the measurement involves the loss of rock grains resulting from the scratching of the steel needle, the method was adapted by also measuring the mass of loose grains, using the difference in mass of the specimen before and after tested. The lost mass result would further provide an estimate of the wear of the rock.

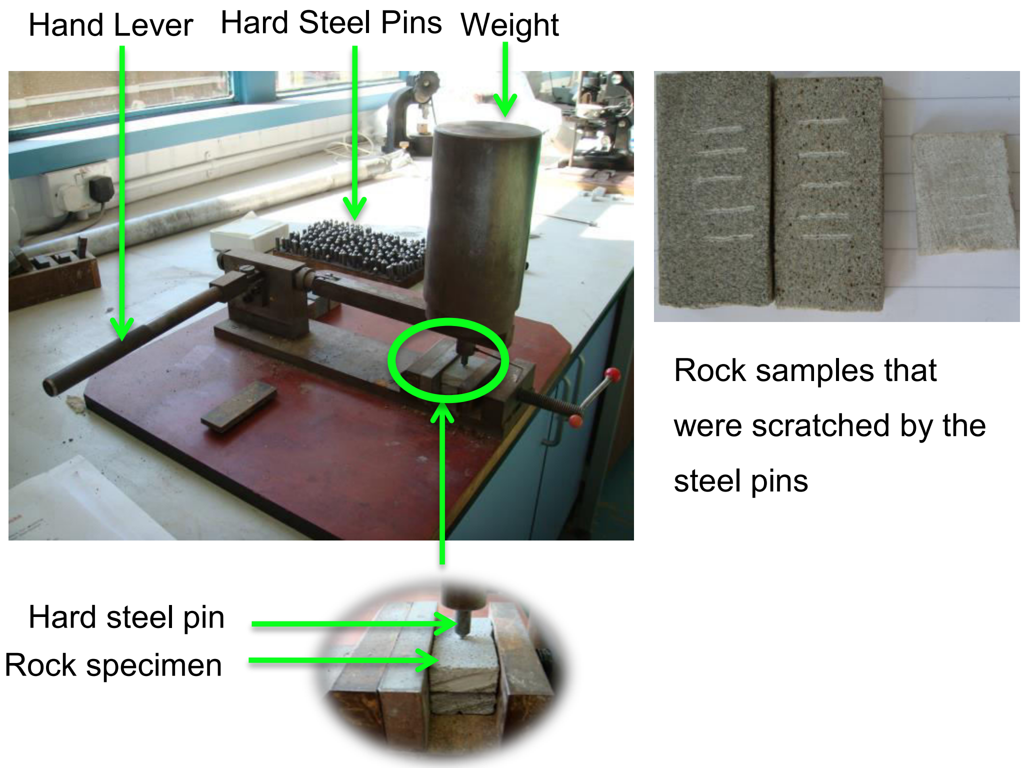


Figure 5.1 Cerchar Abrasiveness Test Equipment (Rock Mechanics Laboratory)

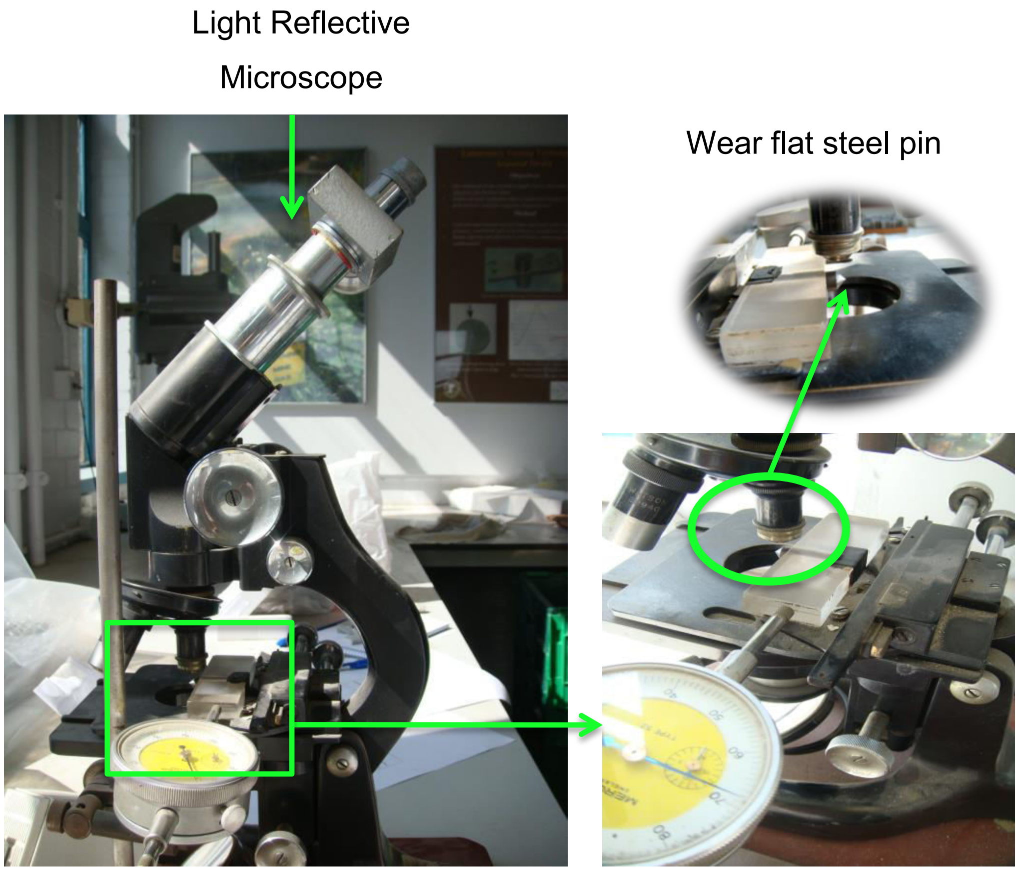


Figure 5.2 Microscopic Reading Method

5.2.1.3 Analysis of Test Results

Table 5.1 presents the UCS and CAI test results for the U.K and Brunei Darussalam sandstone aggregates. Both UCS and CAI results indicate the Craig-Yr-Hessg aggregate is somewhat weaker than the Brunei Darussalam material, thereby satisfying the first of the three selection criteria mentioned in Section 5.2.

Table 5.1 CAI and UCS Test Results

Sandstone Rock Source	**CAI**	**Mass of loose grains (g)**	**Degree of Abrasiveness (Cerchar, 1973)**	**UCS (MPa)**
Brunei Darussalam's Sandstone (Puni Quarry)	4.32	0.02	Highly	197
Sandstone (Craig-Yr-Hessg Quarry)	3.7	0.05	Moderate	111

For further confirmation, an X-ray diffraction analysis (XRD) together with an elemental analysis using Energy dispersive X-ray (EDX) was performed on the sandstone aggregate powder to find the main chemical compositions. Figure 5.3 represents the spectrum of chemical composition of the U.K's sandstone from the XRD analysis. The spectrum clearly indicates a silicon dioxide (SiO_2) peak therefore, the sandstone is dominantly composed of quartz which is the same as the Brunei Darussalam sandstones (Tate, 1968). Hence, it was decided to use sandstone from Craig Yr Hesg quarry as aggregate for the research study. If not specified otherwise, from now on all results will refer to tests based on this stone and this Craig-Yr-Hesg aggregate will be known as 'U.K. aggregate'.

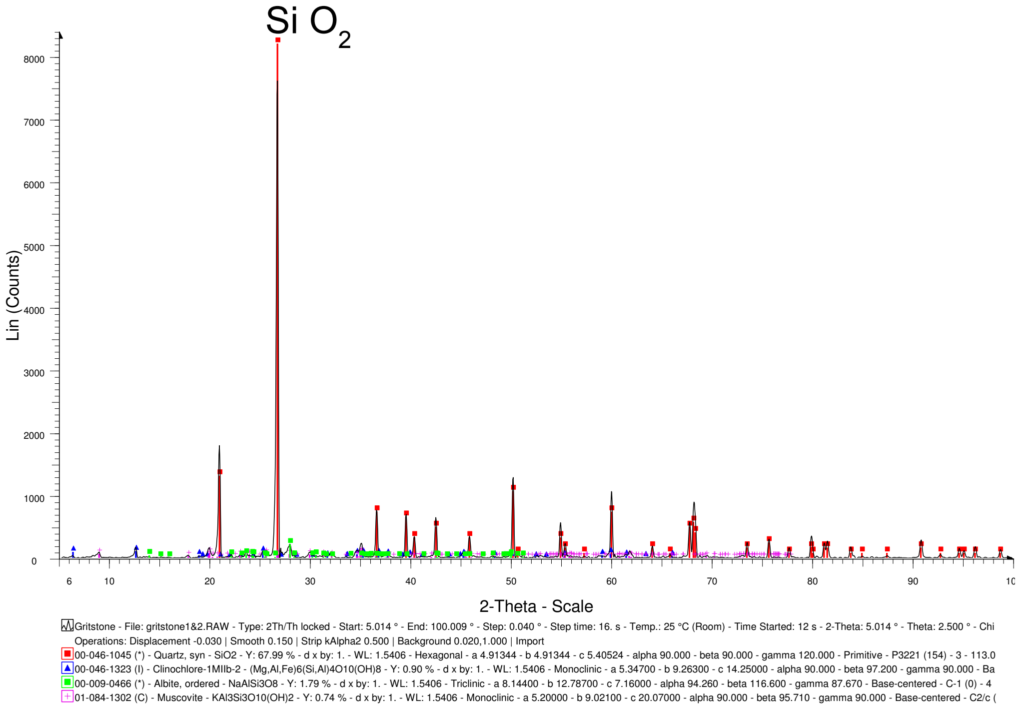


Figure 5.3 XRD analysis of sandstone from Craig-Yr-Hesg Quarry Swansea, U.K.

5.2.2 Gradation

The gradation of the U.K sandstone aggregate had to be designed to closely match the typical grading of the Brunei Darussalam sandstone aggregates. Figure 5.4 presents the typical gradation of the latter (produced by Puni Quarry) that lies at the mid-point of the road base gradation envelope as specified in Brunei Darussalam's GS1 (CPRU, 1998). It runs roughly on the lower boundary of the foamed bitumen stabilised grading envelope, within the ideal zone (Asphalt Academy, 2009). Hence, the gradation of the Brunei Darussalam sandstone aggregates was used as a target gradation design for the U.K sandstone.

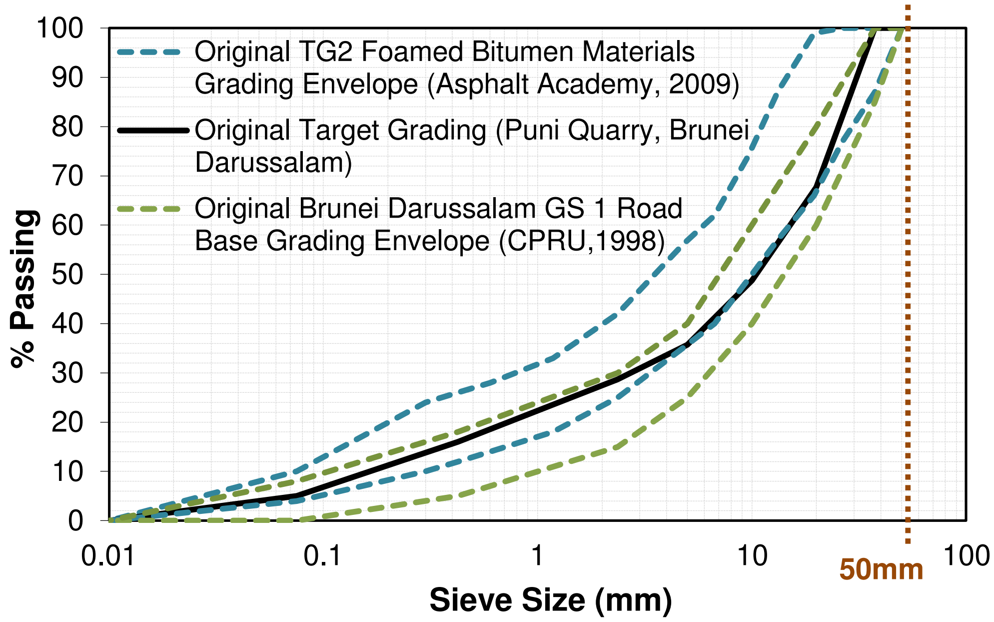


Figure 5.4 Typical Brunei Darussalam sandstone aggregate gradation compared with foamed bitumen stabilised material (TG2) and Road Base (GS1) gradation envelopes

The U.K sandstone aggregates used in this study were supplied in kind by the operator, Hanson Aggregates. They were supplied in accordance to the stock size fractions passing 20mm, 14mm, 10mm, 5mm and dust. Each stock bin's grading was obtained by fractioning materials received into the British Standard (BS) sieve sizes so as to compare with the Brunei's specified grading for analysis as shown in Table 5.2. As can be noticed in Table 5.2, the quarry is not producing aggregate sizes larger than 20mm. This is because the aggregates are mainly utilised for road surfacing which requires a nominal aggregate size of 14mm. It was therefore impossible to design the U.K sandstone's gradation to perfectly match the typical Brunei Darussalam sandstone gradation and also to fall within the GS 1 specified road base gradation envelope. Therefore, modification was made by adjusting the target gradation, the typical Brunei Darussalam sandstones gradation to allow the use of the available raw U.K sandstone aggregate sizes with a maximum particle size of 20mm (see Figure 5.5).

Table 5.2 Particle gradations for different aggregate batches or sizes (BS EN 933-1:1997)

	Percentage Retained (%)				
BS Sieve Size (mm)	20mm	14mm	10mm	6mm	Dust
20.0	5.05				
14.0	76.4	1.42			
10.0	14.76	64.06	6.68		
5.0	2.74	32.51	88.92	55.09	1.17
2.36	0.05	0.95	3.33	42.01	26.93
0.425	0.02	0.32	0.11	1.02	36.09
0.075	0.36	0.3	0.49	0.34	21.8
Pan	0.58	0.37	0.31	1.8	14.06

The adjustment was done by an interpolation method which was used to formularise an equation so that the target grading would be shifted horizontally at constant magnitude by making 20mm as the new maximum particle size. Therefore, the new sieve sizes are calculated using the formularised equation 5.1 below: (see Appendix 1 for detailed calculations for each grading):

Equation 5.1:

$$log_{10}(d_n) = \left[\frac{\{log_{10}(d_o) - log_{10}d_z\}\{log_{10}d_{20mm} - log_{10}d_z\}}{log_{10}d_{max} - log_{10}d_z}\right]/100$$

where

d_n = New particle size

d_{max} = Largest particle size in the original grading

d_o = Original particle size

d_z = Minimum sieve size in all gradations

It was a challenge to combine the bin sizes of the raw U.K sandstone aggregate so as to fit the adjusted target grading of the typical Brunei Darussalam sandstone aggregates. After several attempts, the closest possible fit for UK sandstone aggregates grading was designed as plotted in Figure 5.5 which is referred as the "actual grading". The figure indicates that the actual grading design has an excellent fit, this shifted target representing the Brunei Darussalam sandstone aggregate grading and fulfilling the grading requirements for the granular material to be used in this study. Although the shifted target grading and actual grading curves lie outside the GS1 road base gradation envelope due to the unavailable coarse aggregate fractions, both curves run roughly on the upper boundary of the TG2 foamed bitumen material grading.

Furthermore the particle sizes passing 0.075mm fall within an acceptable range of 5% to 10% as recommended for foamed bitumen stabilised materials (Wirtgen, 2012 and Asphalt Academy, 2009).

Hence this actual grading design of the U.K sandstone aggregate would be used throughout this study. The actual grading was consistently obtained by fractioning the aggregate particles received and reconstituting them accurately, by mass, according to Table 5.3.

Despite the missing coarser aggregate sizes particularly above 20mm (i.e 37.5mm and 50mm) which might mean that behaviour would not represent the behaviour of the real grading in the field construction, the inclusion of these coarser materials in small specimens would be likely to produce significant variation in the test results. This would also cause practical problems in small-scale laboratory mixing equipment. The recommended maximum aggregate size used for manufacturing a small specimen with a diameter of 100mm is approximately 19mm (about one fifth of specimen's diameter) (Di Benedetto et al, 2001), therefore the actual grading is expected to overcome these limitations.

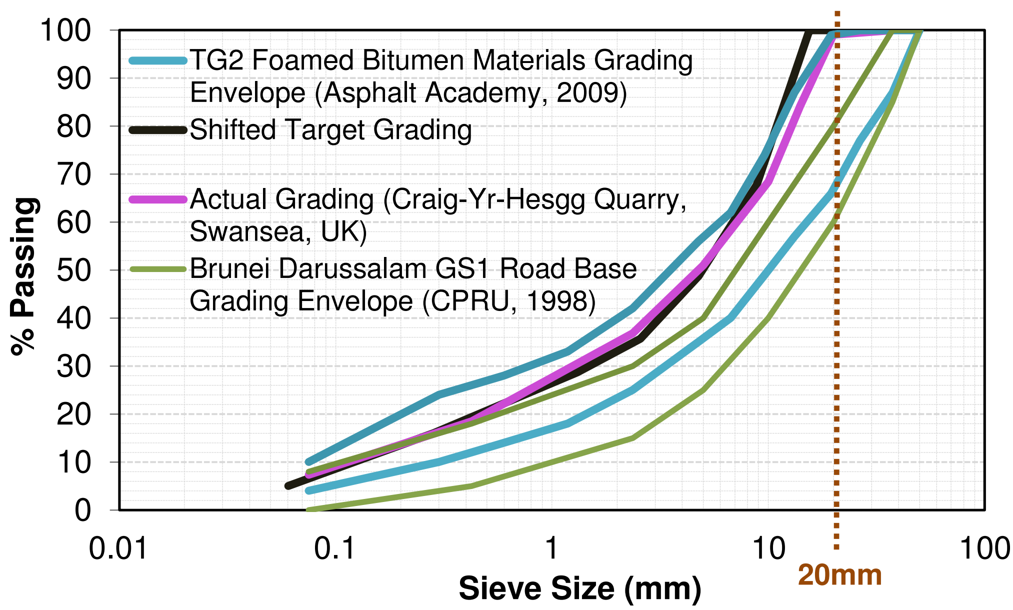


Figure 5.5 Shifted target gradations starting at new particle size of 20mm to which the actual gradation was fitted with asscoiated gradings.

Table 5.3 Proportion of actual aggregate gradation used in the study

BS Sieve Size (mm)	Percentage Retained (%)					Combined % Passing (Designed Gradation)
	20mm	14mm	10mm	6mm	Dust	
37.5	0.00					**100.0**
20.0	0.96	0.00				**99.0**
14.0	14.55	0.27	0.00			**84.2**
10.0	2.81	12.20	0.76	0.00		**68.4**
5.0	0.52	6.19	10.16	0.00	0.59	**51.0**
2.36	0.01	0.18	0.38	0.00	13.59	**36.8**
0.425	0.00	0.06	0.01	0.00	18.22	**18.5**
0.075	0.07	0.06	0.06	0.00	11.00	**7.3**
Pan	0.11	0.07	0.04	0.00	7.09	**0.0**
Grading Proportion %	**19**	**19**	**11.5**	**0.0**	**50.50**	**100%**

5.2.3 Compaction Methods

Optimum moisture content (OMC) of untreated graded aggregates is an important parameter to determine the recommended mixing and compaction moisture content in the production of foam mix. It is the value of moisture content at which the aggregates can be compacted to their maximum dry density. The ability of aggregates to be compacted is sensitive to both the total moisture content and the compaction method. Therefore, it is necessary to study the effect of compaction method with the available compactors - vibratory and gyratory compactor – to assess their viability to compact and fabricate the mixtures in the laboratory.

5.2.3.1 Vibratory Compaction

Vibratory compaction was recommended by the South African guideline, TG 2 (Asphalt Academy, 2002), to reflect the field compaction using vibratory plant for foamed bitumen stabilised materials. In this study, the vibratory compaction was carried out using an electrical vibrating hammer under controlled weight as shown in Figure 5.6.

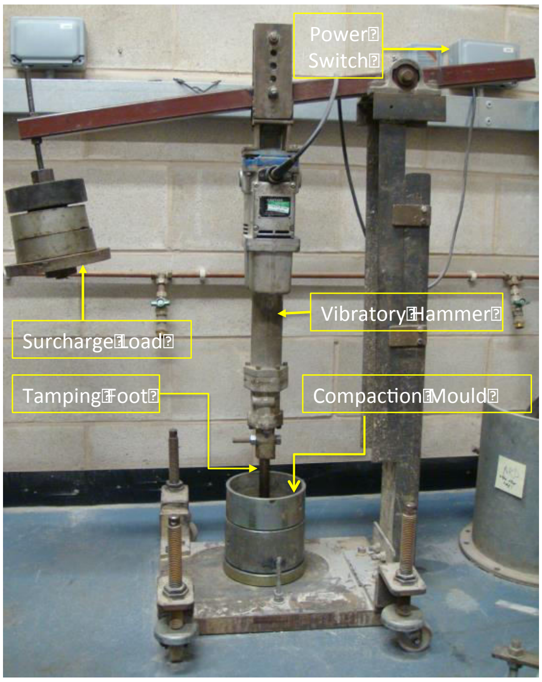


Figure 5.6 Vibratory compactor

The hammer was hung vertically in a loading frame to provide a steady force of 450±10 N in accordance with BS 5835 (BSI, 1980). The compaction mould was fixed securely to the provided base. However, the mould used in this study was slightly different from the standard vibratory compaction mould. The aggregates were compacted in three layers and each layer was subjected to vibration under a constant downward force for 60 seconds. A 150mm tamping foot was attached to the hammer. To flatten surface irregularities of the specimen faces at the end of the compaction, a smaller compaction foot (100mm) was used.

5.2.3.2 Gyratory Compaction

Gyratory compaction is a common laboratory compaction for manufacturing hot asphalt mixture specimens (BS EN 12697-31:2007). The compaction effort is controlled by the application of a vertical pressure (600kPa), at a certain angle of gyration (1.25°) and gyration speed (30 revolutions per minute) as seen in Figure 5.7.

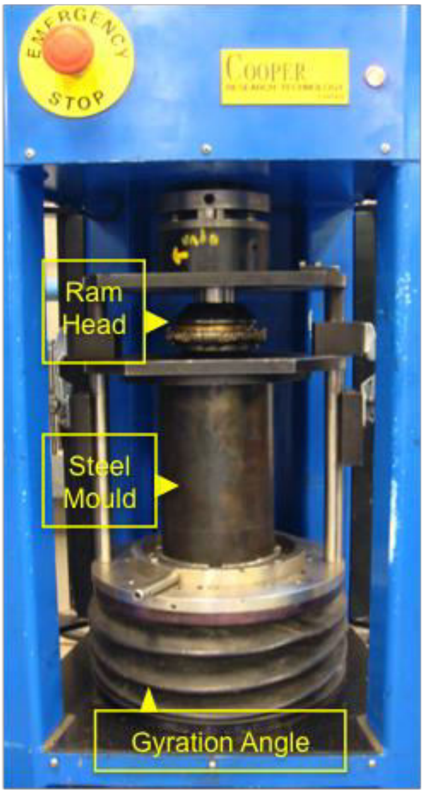


Figure 5.7 Gyratory compactor

In this investigation, all aggregate samples with varying water contents were compacted at the same number of gyrations. During compaction the height of the sample was automatically measured and the wet density calculated. The compaction data could be seen on-screen in graphical and tabular format as compaction progressed and these data can be saved in Microsoft Excel™ compatible format.

5.2.3.3 Moisture Contents and Dry Density

Normally all the aggregates delivered to the laboratory were dried for storage. Thus, no further drying was necessary prior to the compaction test. Target added water contents of 2.0%, 4.0%, 6.0%, 8.0% and 10.0% (by mass of dried aggregates) were used in the investigation and recorded as initial moisture contents. Immediately after compaction, the volume and wet mass of each specimen was determined. These data were used to calculate the densities.

The mass of each specimen was monitored before and after compaction for the determination of moisture contents. Each compacted specimen was dried in an oven at 100°C until a constant mass was obtained and the dry density was calculated. Residual moisture was calculated after the compacted specimen was dried.

5.2.3.4 Analysis of Compaction Results

It has been noted that as the water content increases, the compaction tends to be easier and hence, the dry density of compacted aggregates increases until reaching a maximum value at the optimum moisture content as shown in Figure 5.8. However, an increase in the water content beyond its optimum value results in a rapid decrease in the dry density due to the presence of excess water that tends to push the aggregate particles away from each other.

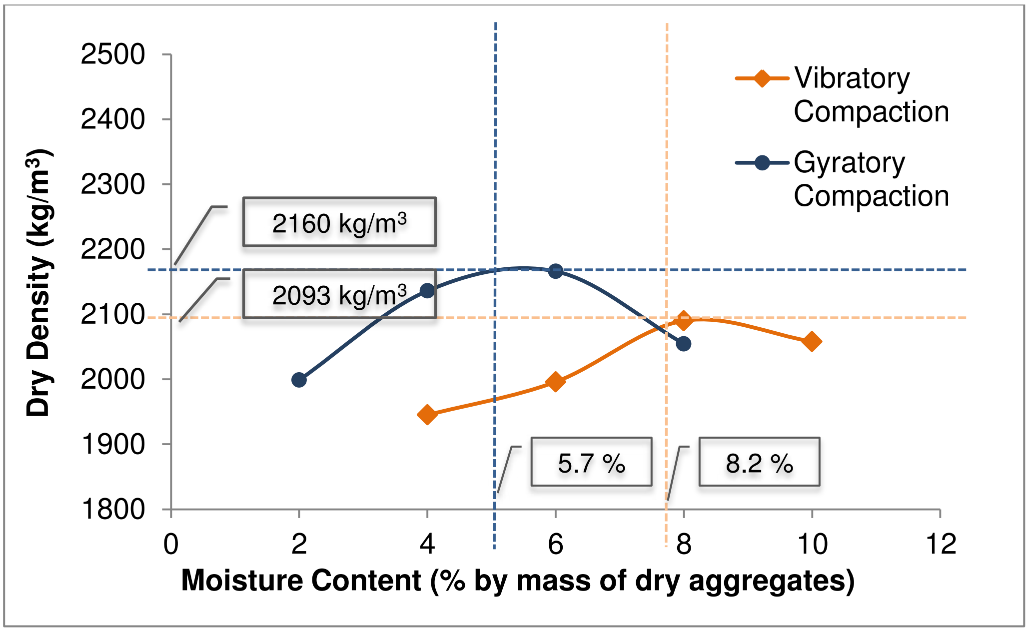


Figure 5.8 Dry density against moisture contents of dry aggregates compacted by vibratory and gyratory compactors

In the case of using vibratory compaction, the optimum initial moisture content was found to be 8% and the maximum dry density was 2095kg/m^3. On the other hand, in the case of gyratory compaction, the optimum initial moisture content was found to be 5.7% and the maximum dry density was 2160kg/m^3 as summarised in Table 5.4.

Table 5.4 Results of Maximum Dry Density and Optimum Moisture Content

Compaction Methods	**Vibratory**	**Gyratory**
Maximum dry density, kg/m^3	2095	2160
Optimum initial moisture content, %	8.2	5.7
Optimum residual moisture content, %	8.1	5.9

The densities recorded from the vibratory compaction were lower than the densities recorded from the gyratory compaction. This is reasonable because the vibratory compaction process delivers energy to the aggregate particles by vibration action applied monotonically via vertical stress, only to the top surface of the specimen. In contrast, the gyratory compaction benefits from a shearing action that induces reorientation of aggregate particles in the aggregates matrix in addition to the applied vertical static stress.

An important point that was also noted was that, at high moisture contents, particularly beyond the optimum initial moisture content, an amount of water was observed seeping out of the base of the mould during compaction which included fine aggregate particles. Large amounts of materials from the compacted layer in the mould also tended to stick to the bottom of the foot upon raising the compactor

The gyratory compaction process is more automated and simpler than the vibratory compactor. Therefore the gyratory compaction has less room for human error, and it can be accurately conducted with less training or experience. The use of a gyratory compactor would therefore also increase the repeatability of the test because the results are less dependent on the person.

Laboratory compaction may be considered light as compared to the field compaction. Field compaction is carried out using vibratory and pneumatic rollers; therefore their heavy weights and high amplitudes would result in higher density of the mix than the mix compacted in the laboratory compactor (Virgil Ping et al, 2002). Therefore, for the purpose of this study, the higher MDD and lower OMC of the two methods, given by the gyratory compaction, were selected for this study. Moreover, the gyratory compaction method consumes less time during the preparation and this would minimise the loss of moisture prior to compaction.

5.3 BITUMEN

The second material to be analysed was the bitumen. The first set of tests was aimed at investigating the general behaviour of the aggregate to be treated with foamed bitumen. In order to do that it was necessary to choose what type of bitumen should have been used as a 'starting point', find a supplier and, once obtained, characterise it with a series of tests. Also, it was important to be sure that the supplier would be able to supply the same source of bitumen for the whole duration of the research.

The objectives of the analysis were as follows:

- To choose what type of bitumen should be used so that the penetration grade would be within the range of the conventional bitumen type used in Brunei Darussalam; bitumen 80/100.
- Determine the rheological properties of the selected bitumen such as penetration, viscosity, and softening point.
- Investigate the changes in the penetration and viscosity of the circulated neat bitumen in the foamed bitumen plant and also the foamed bitumen.

5.2.1 Selection

Bitumen penetration grade 80/100 is a conventional type used as paving grade bitumen for the production of hot mix asphalt in Brunei Darussalam. However, the bitumen grade of 80/100 is uncommon in the U.K instead the closest available type is the bitumen penetration grade 70/100. In this case, the selection of bitumen was based on its individual penetration value. Two bitumen grades, 79 and 90pen, were sampled from two companies in the United Kingdom, Shell Bitumen and TOTAL Bitumen respectively. The rheological properties of both bitumen types are given in Table 5.5.

Table 5.5 Rheological properties of bitumen

Property	**Bitumen Penetration Grade**		**Test Specification**
	79	*90*	
Viscosities:			BS EN 13302:2003
120°C (mPa.s)	947.8	611.9	
160°C (mPa.s)	142.2	96.0	
180°C (mPa.s)	73.8	51.0	
Penetration at 25°C (0.1 mm)	73	87	BS EN1426:2007
Softening Point (°C)	49.8	47.2	BS EN 1427:2007
Penetration index (°C)	-0.152	-0.243	

The penetration test is expressed as the distance that a needle vertically penetrates a sample under known conditions of loading, time, and temperature in accordance to BS EN 1426:2007 (BS, 2007). It was carried out to confirm the bitumen's penetration value as supplied. From Table 5.5, bitumen 90pen is confirmed to fall within the range of penetration 80/100 and it has been used throughout this study period. The softening point test measures the temperature at which the bitumen has tendency to flow.

Viscosity measures the amount of force required to cause a spindle's movement in the bitumen fluid. The relationship between the viscosities of the two bitumen grades with increase in temperature is presented in Figure 5.9. A highly viscous bitumen requires a greater amount of force to shear than a less viscous one. It is also clear that the temperature has an effect on the viscosity of the bitumen.

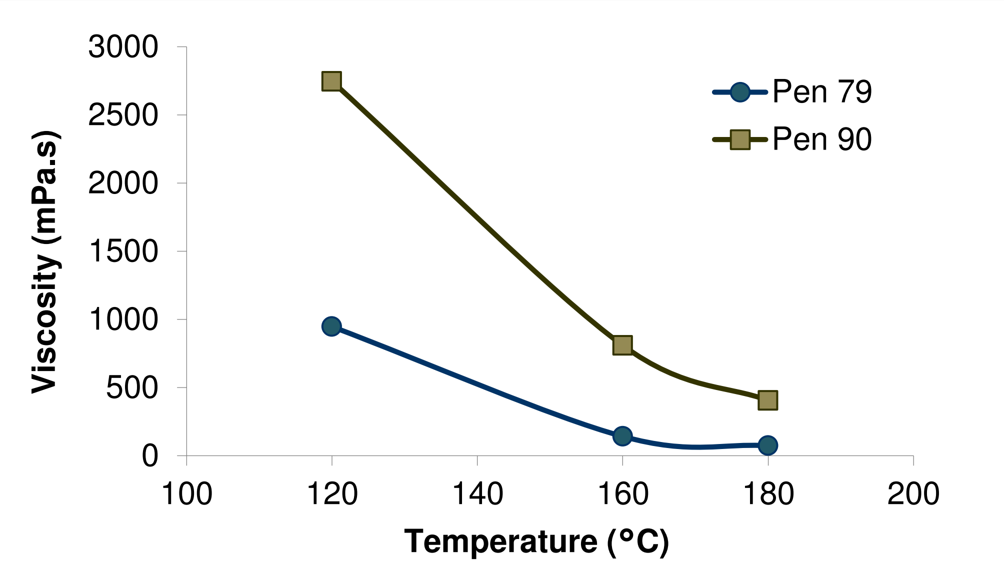


Figure 5.9 Viscosity of Bitumen Pen 79 and 90 at temperature of 120°C, 160°C and 180°C

5.2.2 Circulated Neat Bitumen and Foamed Bitumen

The production of foamed bitumen, which is explained in more detail in Chapter 7, involved the long exposure of neat bitumen to a high temperature; pre-heating of neat bitumen was performed for at least 3 hours before it was poured into the foamed bitumen plant's tank. Then it continued to be heated to allow for its circulation in the system throughout the production of foamed bitumen. The temperature of the bitumen in the plant was normally higher than 150°C to keep it fluid.

Since bitumen exposure to high temperature is known to cause aging of the bitumen (Lu and Isacsson, 2002) therefore it was decided to investigate and compare the effect of this high temperature on the properties of the circulated and heated neat bitumen, in terms of the penetration and viscosity values. The heated and circulated neat bitumen was sampled by ladling it directly from the plant's tank.

Foamed bitumen is produced with a small injection of ambient water into a stream of the hot bitumen (i.e. heated and circulated neat bitumen). The presence of water in foamed bitumen may change the initial rheological properties. Therefore, the investigation of the rheological properties was extended to the foamed bitumen. The foamed bitumen sample was taken as soon as it was produced and collected in a cylinder.

Both samples were collected at intervals of one and four hours after the bitumen was circulated. Then they were tested for penetration and viscosity. The results are as shown in Table 5.6. As can be observed in Table 5.6, the longer the neat bitumen was heated and circulated, its penetration value decreased, as did that of the foamed bitumen. There was quite a significant reduction in the penetration value of the neat bitumen after 4 hours of being heated and circulated in the plant, whereas the foamed bitumen still maintained a penetration value within the range of 80/100.

In the case of viscosity, the longer the neat bitumen was heated and circulated, the higher is its viscosity, indicating the aging of bitumen. The viscosity result for foamed bitumen was not possible to obtain. The Brookfield viscosity meter proved not to be the best to measure the viscosity of foamed bitumen. An attempt was made but due to the persistent spitting of foamed bitumen out of the test tube during the test, the test was aborted. This was perhaps due to the presence of water in the foamed bitumen which steamed at 160°C when obtaining the viscosity at that temperature. Although there are other viscosity tests that could be used to determine the viscosity of foamed bitumen such as a manual handheld viscometer, it was not available in the laboratory. Since this test was not a major part of the study so the penetration results were adequate to understand how the property of the foamed bitumen changed. In this case, the bitumen should not be heated for too many hours as the penetration would reduce hence, the rheological property could change producing different characteristics of the foamed bitumen.

Table 5.6 Properties of circulated neat bitumen in a foamed bitumen plant's tank

	Neat Bitumen			**Foamed Bitumen**	
	Bitumen Circulation Time (hr)				
Properties	**0**	**1**	**4**	**1**	**4**
Penetration at 25°C (mm)	87	88	77	89	80
Viscosity at 160°C (mPa.s)	96	96	99	-	-

5.3 ADDITIVES

Additives are usually considered in cases where a foam mix cannot perform optimally particularly due to the climate factors. As was introduced earlier in Chapter 1, Brunei Darussalam is governed by its hot and wet climate with high humidity all the year round. Therefore, the wet environment and high humidity might cause potential problems in the moisture susceptibility and curing process. Therefore it is desirable to find an additive that could accelerate the curing process to gain early strength without delaying the road opening to traffic upon completion. Such additives can be found in the group of hydraulic binders. The group hardens when it reacts with water and becomes impermeable to water upon solidification. Cement was selected because it can be easily available in the local market and is widely used as a stabilising agent in Brunei Darussalam.

It is known that foamed bitumen mixes fall in the category of lightly bonded materials where the binder tends to stick to the fines forming the mortar that then welds the coarser materials together. Therefore, reinforcement could be a way to increase the structural integrity of the aggregate skeleton particularly the coarser materials in the foamed bitumen mixtures. The selection of reinforcement must not compromise economic value where it is readily available in the local area as considered earlier in the evaluation and assessment of potential binders. The study therefore included a preliminary investigation into coir fibres as a potential reinforcement agent for foam mix. Coir fibres were selected because they are cheap and easily available in the local market. They have been increasingly researched as soil reinforcement but have never been used in foamed bitumen mixtures.

5.3.1 Cement

Cement is a hydraulic binder widely used in civil engineering applications. The most common type of cement is Ordinary Portland Cement (OPC). Nowadays there are many OPC blends with admixtures. In order to avoid the effects of other materials, high strength cement was selected, as it has no admixtures.

Cement has been recognised to give beneficial improvement to foam mix properties (Huan et al, 2011, Halles and Thenoux, 2009) and the functions can be outlined as follows:

- As an accelerant of the development of early stiffness of the foam mix due to the formation of cementitious bonds, therefore the curing process would be rapid. Since its benefit stands out more in bitumen emulsion mixes that originally have higher water content, about 30 – 40% (Jostein, 2000), this could improve the foam mix performance in wet weather conditions.

- The formation of hydrated products within the foam mix provides a more dense structure and less voids resulting in the reduction of moisture susceptibility (Muhammad et al, 2003, Fu et al, 2008 and Gonzalez et al, 2009), and in assisting the dispersion of bitumen and improving adhesion of bitumen and aggregate

In this study, cement was not selected due to the need to modify fines but due to the other stated functions. Moreover, the selected aggregate gradation in this study did not need fines modification as it has adequate fines to fulfil the requirement of foam mix materials as mentioned in Section 5.2.

5.3.2 Coir Fibres

Coir fibres have been widely applied as woven mats in soil reinforcement for erosion control. A mat was supplied by the Cocogreen Enterpise, a local company based in Brunei Darussalam. The entwined woven coir fibres were loosened by hand and later cut into selected lengths to hypothetically reinforce at least two maximum stone sizes of 20mm (see Figure 5.10). The utmost care has been taken to maintain the same cut-off lengths. Table 5.7 shows the properties of the coir fibres.

(a) (b)

Figure 5.10 (a) Typical entwined woven coir fibres (b) Cut coir fibres

Table 5.7 Property of Coir Fibres (Manufacturer , Cocogreen Company)

Property	**Values**
Specific Gravity (kg/m^3)	1177
Water Absorption (%)	93
Tensile Strength (MPa)	95 – 118
Modulus of Elasticity (GPa)	8

5.3.3 Others

The climatic durability study as an extension to the moisture susceptibility test included two common anti stripping agents as used in hot asphalt mix. They would be combined with foam mix to investigate and contrast the effects of dry and wet cycles on the combination mixtures with foam mix. The climatic durability study can be found and is further explained in Chapter 10. The two additives were as follows:

Hydrated Lime (HL) is a white powdered form of anti-stripping agent used in hot asphalt mixtures with the chemical formula $Ca(OH)_2$.

Wet Fix (WF) is a liquid anti-stripping agent that is commonly used in hot asphalt mix. It is an amine based liquid that would help to reduce the surface tension between the bitumen and aggregates to promote strong adhesion. Thus, it can be considered as a surface active agent (Selvamoha el al., 2007).

5.4 SUMMARY

From the analysis of materials to be used in this study, some considerations can be deduced and summarised as follows:

Aggregates

The sandstone aggregates were sourced only from Craig-Yr-Hesg quarry and one gradation to be used in this study was designed to closely represent the common sandstone aggregates found in Brunei Darussalam, albeit at a transformed grading with a reduced maximum stone size.

Bitumen

Bitumen 80/100 is a conventional type used in Brunei Darussalam. However, for the purpose of this study, a specific penetration value, 90pen was selected. The penetration of the bitumen should be consistently within the range of 80 to 100 and pen 90 was found the most suitable as it did not age drastically when it was heated and circulated in the foamed bitumen plant. However, it is advisable to use fresh neat bitumen every mixing day to maintain the same consistency in the quality of the bitumen. Moreover a significant reduction in the penetration of the neat bitumen whilst heating and circulating in the foamed bitumen plant could change the characteristics of the foamed bitumen that is to be produced and might cause it to become less workable.

The softening point of the chosen bitumen was 47.2°C that indicated that the bitumen could easily became fluid at this temperature. Therefore, the suitable curing temperature, particularly in the dry curing condition, should be selected below 47.2°C. A temperature of 40°C seems to be recommended by many researchers for dry oven curing. More discussions about the curing strategies can be found in Chapter 6.

Compaction

OMC of the untreated aggregates is the key parameter to calculate the mixing moisture content to be added in the production of foam mix. Hence, a compaction test was essential to determine the OMC and corresponding maximum density of aggregates.

Two methods were used, the vibratory and gyratory compaction methods. It was found that the gyratory compaction method produced specimens with a higher maximum dry density and of a lower optimum moisture content than the vibratory compaction method. The gyratory compactor was found to be easy to use; therefore the OMC of untreated aggregates was rounded off to 6% to compensate for any loss in moisture during the mixing and preparation process. Maximum dry density was 2160kg/m^3. Gyratory compaction was selected for use in fabricating cylindrical test specimens throughout the study. Immediate compaction is required for all specimens to avoid the loss of moisture.

Additives

Cement would be used in the experiments to tackle any shortcomings of the foam mix in terms of development of early strength and to process curing rapidly. This is thought to be important for the humid conditions prevalent in Brunei Darussalam.

Coir fibres, hypothetically, can reinforce the unbound coarse aggregates in the foam mix. Since these would be new to foam mix therefore, a preliminary experimental investigation would be carried out.

Other additives, anti-stripping agents such as hydrated lime and wet fix, are included to assess their potential to improve the climatic durability of the foam mix and to create a greater contrast between the effects of the dry and wet cycles as discussed in Chapter 11.

CHAPTER 6 RESEARCH APPROACH AND METHODOLOGY

6.1 INTRODUCTION

This chapter describes the experimental techniques used to investigate the engineering properties of foam mix in this study, namely the Indirect Tensile Stiffness Modulus (ITSM) test, Indirect Tensile Strength (ITS) test, Repeated Load Axial test (RLAT) and Wheel Tracking Test (WTT). It outlines the three extreme field curing conditions specifically selected to closely simulate Brunei Darussalam's local climatic condition. These specific conditions are not stated in any existing guidelines particularly in regard to the humid environment. Therefore, further details of the experiment design are also elaborated to characterise the properties of foam mix.

6.2 EXPERIMENTAL TECHNIQUES

The experimental techniques consist of various test methods. The main test method was an Indirect Tensile Stiffness Modulus (ITSM) in order to evaluate an important property of road base materials. Other tests such as Indirect Tensile Strength (ITS) and Wheel Tracking Test (WTT) were also included to measure the effect of reinforcement and rutting resistance respectively. In addition to these methodologies, imaging techniques were also used to quantify the characteristics of foam mix, but these are explained in Chapter 12.

6.2.1 Indirect Tensile Stiffness Modulus (ITSM)

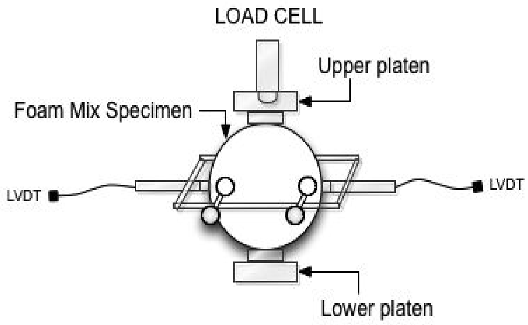


Figure 6.1 ITSM test equipment and its side view's cross-section

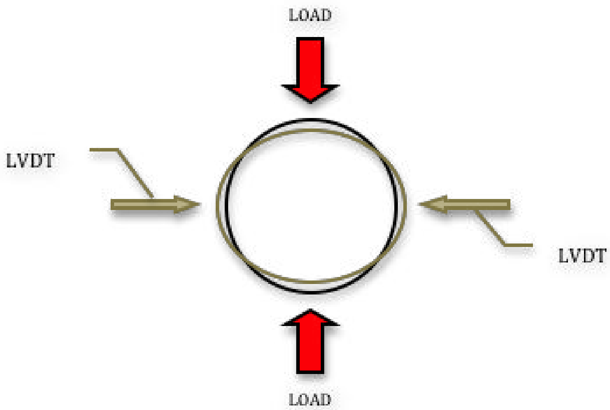


Figure 6. 2 The deformation induced by the ITSM test

The ITSM test used one of the Nottingham Asphalt Testers (NAT) to evaluate the stiffness modulus of the bituminous mix, in this case foam mix as shown in Figure 6.1, in accordance with BS DD213 (1993). It can be used to evaluate loss of stiffness of bituminous mixes after being subjected to dry curing followed by wet curing for a certain period of time.

The ITSM test is carried out in a controlled horizontal deformation mode. The typical target of horizontal deformation is 5 microns for 100mm diameter specimens or 7 microns for 150mm diameter specimens of hot asphalt mixes. The horizontal deformation is induced by the loads and measured by external Linear Variable Differential Transformers (LVDTs) lined up with the specimen's diameter as shown in Figure 6.2. In studies to evaluate bitumen emulsion mixes, the horizontal deformation was controlled at a lower value than for hot asphalt mixes and that was 2 microns at a test temperature of 20°C (Santagata et al, 2009 and Oke, 2010). In this study, it was decided that the recovered horizontal deformation should be 3 microns to allow for the slightly higher test temperature of 30°C.

6.2.2 Indirect Tensile Strength (ITS) Test

The ITS test is conducted by applying a compressive load to a cylindrical specimen along a vertical diametrical plane. A uniform tensile stress is developed perpendicular to the direction of the applied load along the central vertical plane causing the specimen to fail by splitting. This test is also otherwise known as the splitting test. The objective of the test in the present study was to observe the effect of coir fibre reinforcement on the tensile strength of the foam mixes.

Figure 6.3 ITS test equipment

The specimen was positioned in an Instron uniaxial loading device. A seating load was applied to the specimen to ensure a satisfactory zero load position, i.e when the upper platen was just touching the specimen as shown in Figure 6.3. Then the device was set to the initial condition and followed by the test sequences. Each specimen was compressed until the specimen reached 13mm vertical deformation or collapse. The specimen normally reached the peak load and collapsed before it reached the 13mm deformation.

6.2.3 Repeated Load Axial Test (RLAT)

The RLAT was conducted on the foam mix with and without the reinforcement of coir fibres to determine the resistance to permanent deformation, in accordance with British Standard DD 226 (1996). The test uses a cylindrical specimen with a diameter of 100mm or 150mm and thickness preferably between 40mm and 100mm. The test simulates the slow moving traffic that leads to the most deformation in a real road. A load cycle consists of a vertical and axial stress application of 1s durations followed by a 1s rest period. According to BS DD 226, a standard test consists of 1800 load cycles with a maximum axial stress of 100 kPa at a test temperature of 30°C. The test is initiated by a stress of 10 kPa for duration of 10 minutes (the conditioning stage) to ensure that the loading platens are properly seated onto the specimen prior to running the testing. The vertical deformation of the specimen is measured by two LVDTs mounted on the upper loading platen, as shown in Figure 6.4. The test output consists of vertical deformations of the specimen plotted against number of load cycles.

Figure 6.4 RLAT test equipment

Since the objective of the test in this study was to investigate the effect of coir fibre reinforcement on the foam mix under a wet environment, therefore a plastic membrane was used to wrap the wet test specimens in attempt to simulate the wet environment and minimise the loss of moisture.

6.2.4 Wheel Tracking Test (WTT)

The WTT, under the EN 12697-22 standard method, was used to determine the susceptibility of bituminous mixes to deformation that might form by repeated traffic loading at a fixed temperature.

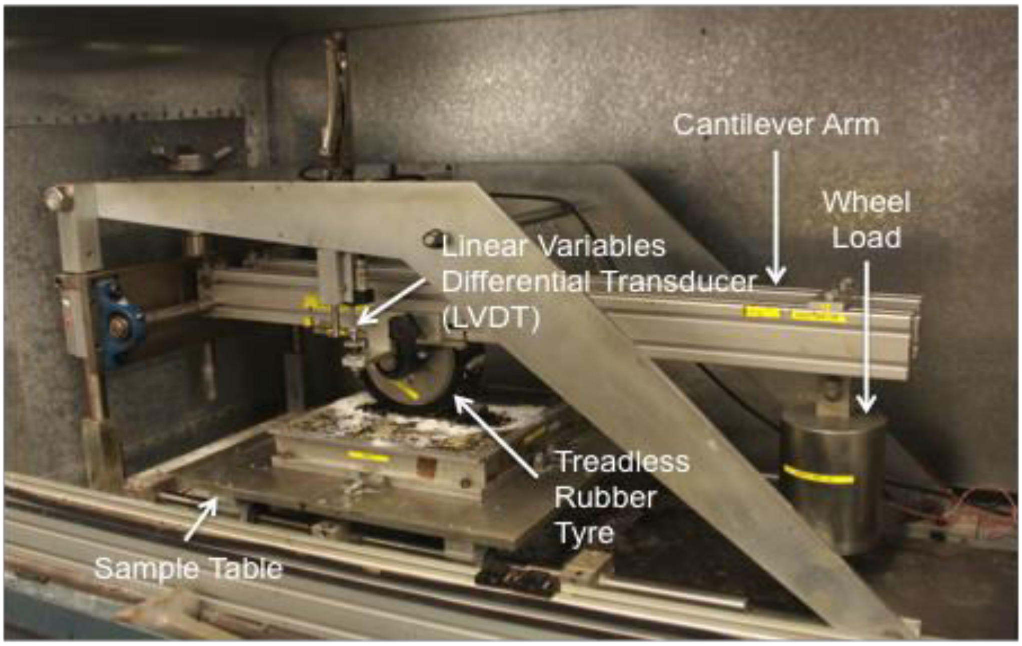


Figure 6.5 Wheel Tracking test (WTT) device in an insulated temperature control cabinet

The WTT device, as shown in Figure 6.5, is made of a solid aluminium frame supporting a stainless steel temperature control cabinet fitted with a double glazed window. It simulates the action of traffic by rolling a tread-less rubber tyre wheel of thickness 20mm, which is loaded with 520N, on top of the specimen. The test can be carried out on either a core specimen of 200mm diameter or on a square slab of 300mm x 300mm x 50mm thick manufactured by a laboratory roller compactor.

The LVDT transducer measured the rut depth at the centre of the test specimen along the longitudinal tracking profile. The rut depth was measured every pass by the LVDT transducer at about 25 points, approximately equally spaced over a length of 50mm along the wheel path. The tracking is finished after the required number of cycles or if the rut depth exceeds the maximum rut depth (equal to 15mm). The development of rut depth was displayed on an on-screen graph in terms of rut depth versus time, along with the thickness profile and temperature. The mean value was considered as the rut depth at that number of passes and was recorded in millimetres.

6.3 CURING PROCESS

Curing can be described as the process of water molecules escaping the foam mixes during the period immediately following placement and compaction. The water molecules move to the surface of the applied foam mix through pore pressure flow that is induced by the action of the compactor during construction and/or the action of traffic and eventually evaporates into the atmosphere (Jenkins, 2000). The presence of water in foam mixes is inevitable as it is a significant element in mixing and compaction as well as for the production of foamed bitumen. The water may be present in spaces between the aggregates in the mixes, in the individual aggregate's pore as well as in the foamed bitumen. Curing occurs gradually after a period of time until the foam mix attains its stable condition. When a satisfactory amount of water or moisture has been released into the atmosphere, a direct contact between the aggregates and foamed bitumen binder or the foam mastic would develop, generating adhesion and hence the mechanical competency of the foam mixes. However, the speed of moisture loss can be influenced by the temperature, humidity level (amount of water vapour already in the air) and local wind speed. The gradual development of the curing process makes the early phase of a foam mix's life critical. If the curing was inefficient or incomplete, it could lead to premature pavement distress.

6.4 REVIEW OF CURING STRATEGIES

The curing strategy is an important approach in the mix design and requires evaluation to simulate closely the expected field conditions from optimistic to extreme worst conditions. The conditions should consider the temperature, moisture and distinct climate patterns.

Literature findings from various researchers and agencies reveal a number of laboratory curing protocols designed to closely simulate the climatic regions where the case studies were developed. Although there is an established technical guideline developed by the Asphalt Academy, based in South Africa, many researchers have preferred to design their own curing protocols aiming to closely simulate the field condition of their country. The designated curing period was chosen by the respective researchers to consider the climate, adverse weather condition, construction practices and the representation of different stages of pavement life. A collection of curing conditions studied can be seen in Table 6.1 and 6.2.

Table 6.1 Reviews of dry curing conditions

Dry Curing Condition	Simulated Field Condition	References
Cured in the oven for 3 days at 60°C	Driest field condition	Bowering (1970)
Cured in a mould for 1 day with both exposed ends at 23± 8°C	Short term curing within 24 hours after construction (dry and temperate climate regions)	Ruckel et al (1983)
Cured in a mould for 1 day at ambient temperature followed by oven drying for I day at 40°C	Between 7 to 14 days	Ruckel et al (1982)
Cured in a mould for 1 day at ambient temperature followed by the oven drying for 72 hours at 40°C (or 48 hours may be adequate)	Long term	Ruckel et al (1982) Kavussi and Hashemian (2004) Sunarjono (2008)
Cured in the oven for 4 days at 60°C followed by 3 days at 24°C	Unspecified	Engelbrecht et al (1985)
Sealed curing at 40°C for 3 days	Unspecified	Weston et al (2002)
Unsealed cured for 1 day at ambient temperature then sealed cured for 48 hours at 40°C	Long term curing and simulate field Equilibrium Moisture	Houston and Long (2004)
Cured in a mould for 1 day at ambient temperature followed by unsealed curing	Unspecified	Hodgkinson and Visser (2004)

for 72 hours at 60°C		
Sealed curing at 20°C for 24 hours	First weeks of after construction	Halles and Thenoux (2009)
Cured at 40°C for 72 hours	Long term	Halles and Thenoux (2009)
Sealed curing for 48 hours at 40°C	Six months	Asphalt Academy (2009)
Immediately extruded from the mould after compaction followed by unsealed curing for 7 days at 40°C	Long term – optimistic for water evaporation representing field condition in dry season	Fu et al (2010)
Immediately extruded from the mould after compaction and then sealed curing for 1 day at 20°C	Short term – conservative for water evaporation simulating the first few hours after construction	Fu et al (2010)
Sealed curing for 7 days at room temperature (25°C)	Western Australia's Field Condition	Huan et al (2010)
Sealed specimens at ambient temperature, 20°C for 14 days	New Zealand Suboptimal Conditions – Shortly after construction with Intermittent or heavy rain	Gonzalez et al (2011)

Table 6.2 Reviews of wet curing conditions

Wet Curing Condition	References
Cured in the oven for 4 days at 60°C followed by 2 hours wet vacuum saturation then soaked for 3 days at 24°C	Engelbrecht et al (1985)
Cured in the oven for 3 days at 60°C then allowed for 8nos of freeze/thaw cycles: 4 cycles each day.	Joisten (2000)
Soaked in water bath at 25°C for 5 days (tested at 24 hours intervals)	Saleh (2006)
Soaked in water bath for 7 and 40 days at 20°C	Fu et al (2010)
Soaked in water bath at 25°C for 24 hours	He and Wong (2008) Kim et al, (2011)

The newly made laboratory compacted foam mix specimens were generally left in the mould for 24 hours to build up the cohesion of the foam mixes for easy handling before the specimens were extruded. Then the designated curing period was followed. Using a curing temperature of 60°C, Hodgkinson and Visser (2004) found that the dry foam mixes with cementitious binder exhibited higher strength than the ones with inactive fillers or without any filler. However, a curing temperature of 40°C and later at room temperature appears to be more accepted. Many authors agreed that using a higher curing temperature than the softening point of the binder would affect the rheology of the binder, which could introduce another variable effect into the test results.

The curing period of 72 hours at 40°C has been most commonly practiced by many researchers as it is argued to represent the long-term field conditions of foam mixes. Whilst for the simulation of the short term or early phase field condition, most laboratories adopted sealed curing to retain the moisture in the foam mixes. Sealed curing was also used to study the effect of active filler on foam mixes (Huan et al, 2008). Gonzalez et al (2011) considered that they could simulate the exposure of unsealed foam mix road base to rainfall by adapting sealed curing in the laboratory at ambient temperature for 14 days. Two extreme cases in California were studied to investigate the curing mechanism of foam mixes: i) a conservative condition for water evaporation which normally occurs a few hours after construction by sealed curing at 20°C for 24 hours, and ii) unsealed curing for an optimistic water evaporation representing long term field conditions in the dry season (Fu et al, 2010).

Foam mixes are known to be susceptible to water as they are partially bound materials. When they are soaked in water, their stiffness and strength drop enormously (Engelbrecht et al, 1985). To assess this shortcoming, many researchers adopted wet curing by soaking in a water bath at a temperature near to the room temperature. When foam mixes are soaked, the stiffness values may decrease by 40% (Fu et al, 2010). To model a severe condition, vacuum saturation has been applied and followed by soaking (Ruckel et al, 1983).

In South Africa, the prediction of the equilibrium moisture contents in the field led to the development of a laboratory curing protocol by sealing the specimen in a plastic bag (Jenkins, 2000) which later adopted in the Technical Guidelines 2 (Asphalt Academy, 2009). However, the sealed curing did not work well for all specimens as some became too wet to be tested (Long and Ventura, 2004). In Perth, a laboratory study on foam mixes adopted a sealed curing method after initial curing in the mould for 72 hours at 40°C before specimens were extruded and later wrapped around with a rubber membrane to represent the consolidated and drained condition in the field (Huan et al, 2010).

In most countries in Europe and America, adequate curing of road base on site is indicated by measuring the level of the moisture contents in the field. When a satisfactory moisture level has been reached, the road base would be ready to be sealed with hot asphalt mix as its surface layer. In Iowa, the moisture content of the foam mix layers which have been cold in-place recycled with old asphalt surface is required to reach less than 1.5% (Kim et al, 2011). However, this practice can be challenging in inclement weather conditions. Furthermore, it may not be best applied to regions where humidity is high and rain occurs frequently. It would require a long exposure of unsealed foam mixes to achieve the required moisture content and curing in a humid environment has a major impact on the strength of foam mixes (Ruckel et al, 1982).

6.5 CURING AND CLIMATE

Different climatic regions exhibit different curing behaviour. The water escapes to the surface and is lost into the atmosphere, from its liquid state into a gaseous one, through evaporation. The evaporation process can be considered as the main mechanism to progress curing. However, the speed of evaporation is affected by some important climatic factors such as temperature, humidity (amount of water vapour already in air) and local wind speed. A hot climate with dry environment is a favourable weather for evaporation as it allows the water molecules to move about rapidly and eventually escape.

In a cold climate, the evaporation process slows down because the air can hold less water vapour than in a hot climate and there is less energy in the water to promote evaporation. When the environment is humid, the evaporation process would slow down because the additional moisture that can be held in the air is small as it already contains much water vapour.

The climatic conditions in Brunei Darussalam reviewed in Chapter 2 indicate that the humidity level is very high all year round due to the country's proximity to the sea and equator which is also experienced by most countries in South East Asia. It has a relative humidity on average of about 90%. Due to this high humidity environment, the process of moisture loss from the foam mixes would be slow resulting in an increase in the curing period and, thus, the development of the best properties of the foam mix are slowed down.

In Brunei Darussalam, the climatic pattern is generally hot days followed by cool nights all year round. This indicates that the pavement experiences a cycle of diurnal and nocturnal temperature. The temperature variations on most road pavements could be significantly larger than the variation in air temperature. Moreover, the pavement is also exposed to extensive sunshine all year round.

Rainfall commonly occurs intermittently during a day, or is frequent and heavy during monsoon periods. Such climate does not appear to favour the curing process of road constructions in Brunei Darussalam. As the foam mixes are proposed as the road base materials and it is common for road base to be left exposed or unsealed for days until the next layer is placed, therefore, the unsealed foam mixes are exposed to moisture which may remain in the material during its early life when it is sealed. There is no pronounced dry or wet season where one can assure curing would be more favourable.

Additionally, although road base is not opened to traffic immediately until it is properly sealed, some road users may attempt to use the roads to avoid long diversion routes or traffic queues. These common cases do not favour the slow curing process. Instead it is desirable to obtain rapid performance of foam mixes and allow for early road openings – something which has always been a demanding task for engineers.

Other factors such as subsurface or surface water infiltrations, variations in the underlying moisture conditions, drainage and human activities in the vicinity of the road construction areas may also influence the curing process.

6.6 CURING CONDITIONS FOR TROPICAL EQUATORIAL CLIMATE

Most of the previous simulations of field conditions do not appear to be representative of Brunei Darussalam's applications. It is essential to evaluate the properties of foam mixes based on the local field conditions where they are applied even it is not feasible to predict an accurate model for the curing period. However, the simulation of field condition in a standard laboratory would be a difficult task and take time. It is unrealistic to develop different procedures for each real field condition. Therefore, this study recommended three laboratory curing procedures that represent three extreme field conditions, in Brunei Darussalam. Test temperatures were also selected to closely represent field condition.

6.6.1 Determination of Curing and Test Temperature

Using simple algorithm equations (SHRP, 1994), the pavement's surface temperature and temperature at any depth can be calculated with the input data of average air temperature and latitude data of the country's location.

$$T_{surf} = T_{air} - 0.00618Lat^2 + 0.2289Lat + 24.4 \qquad \text{Equation 6.1}$$

where

T_{surf} = Temperature at the surface (°C)

T_{Air} = Ambient Temperature (°C)

Lat = Latitude of the region (Degrees)

For temperatures at different depths, the relationship below is used.

$$T_d = T_{surf}(1 - 0.063d + 0.007d^2 - 0.004d^2) \qquad \text{Equation 6.2}$$

where

T_d = Temperature at depth (°F)

T_{surf} = Temperature at the surface (°F)

d = Depth from the surface (inches)

Using Equation 6.1 with the latitude of 4.8167°N, it was calculated that the temperature of the pavement surface in Brunei Darussalam could reach about 60°C. As the foam mixes are designed for road base layers, they may be exposed to this high temperature prior to surfacing works (such as wearing course and binder course). However, 60°C is higher than the softening point of the binder and this would induce the aging effect of the bitumen. This temperature was only used for WTT as it would allow more damage and a more reliable measure of the rutting resistance.

Road base is commonly constructed so that it lies about 100mm below the two typical asphalt layers (such as a 60mm thick binder course layer topped with a 40mm thick wearing course). Using Equation 6.2, the pavement temperature at that depth was calculated and it was about 40°C. Hence, 40°C was selected as the temperature for dry curing. However, for the purpose of this study, test specimens were unsealed to allow for rapid evaporation and strength gain simulating the long term (Ruckel et al, 1982, Kavussi and Hashemian, 2004 and Sunarjono, 2008).

For the test temperature, the standards normally state to test at ambient temperature which is 20°C. Therefore, having a local average temperature of 30°C all throughout the year, this was selected as the ambient temperature for all tests and also the temperature of the water for the wet curing condition.

6.6.2 Dry Curing

Dry curing represents an optimistic condition for the evaporation process. All compacted specimens were left to cure for 24 hours at 30°C in the mould to gain cohesion and prevent the specimens from falling apart when they were extracted from their moulds. Then they were dry cured in the oven for 3 days at 40°C and dry ITSM tests were commenced immediately.

6.6.3 Wet Curing

Wet curing was selected to investigate the effect of soaking the foam mix in a water bath for a certain period of time. The effect of soaking represents the worst condition in Brunei Darussalam simulating a prolonged exposure to rainfall. Water can easily seep into the pavement through surface cracks. Although a roadside drain is normally built as one of the requirements in construction of roads, the event of prolonged heavy rainfall may overflow the water in the drains onto the road causing flooding.

For the wet curing condition, the test specimens were all soaked in a water bath for 24 hours at 30°C and then immediately tested for Wet ITSM. It should be noted that all the test specimens that underwent wet curing were the same test specimens that were initially used for dry curing and tested for dry ITSM. This was due to the limited mass of materials available to manufacture the test specimens separately for these two curing conditions.

6.6.4 Humid Curing

Humid curing represents a conservative condition for water evaporation and is close to the field condition. The findings would give guidance for the timing of placement of surface layers such as binder and wearing course. It is a common practise and an important requirement that the road base has to be sealed soon after construction so the road can be opened to traffic, thus minimising traffic delays and impact to the road users.

In order to simulate the humid condition, a humid tank was developed in the laboratory with a fully closed lid and filled with water to a required level with water heated to maintain a temperature of 30°C using a fish tank's water heater (see Figure 6.6). The specimens of each mix were placed in separate containers. They should not be in contact with water to avoid the damaging effect of water infiltration which would introduce a variable effect to the humid cured specimens. Even though this tank condition might not perfectly represent the realistic field condition, yet under the accelerated curing, it would permit measurement of the degree of curing at the early age of foam mixes and over time. It would also indicate the period after which a satisfactory stiffness would be reached.

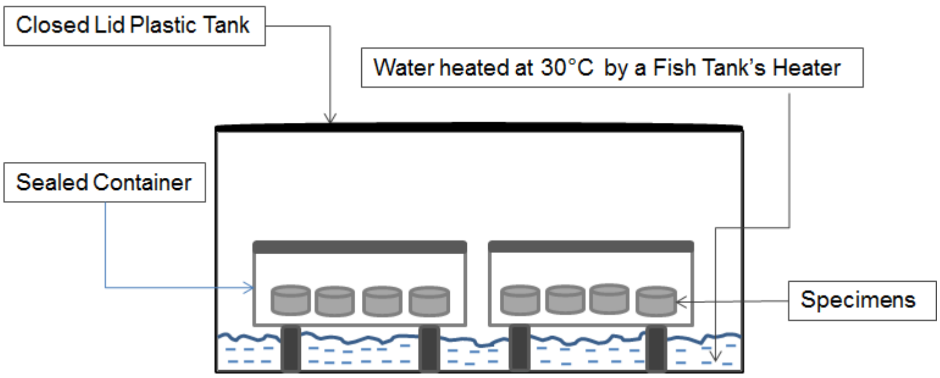


Figure 6.6 Schematic illustration of a laboratory humid tank

6.7 SUMMARY

Test equipment has been described in this chapter namely ITSM, ITS, RLAT and wheel tracker test, for determining the fundamental properties of foam mix. Since road base application is the interest of this study, the ITSM, amongst other test methods mentioned in this chapter, will be used as the main test method to determine the stiffness modulus which is an important fundamental property. Stiffness modulus controls the ability of a layer to distribute loads, to protect the subgrade and to support the surface layer (s).

Three curing conditions have been selected in this study, simulating optimistic, conservative and worst conditions for Brunei Darussalam, for the laboratory evaluation of the fundamental properties of foam mix, particularly stiffness modulus.

CHAPTER 7 PRODUCTION OF FOAMED BITUMEN MIXTURES

7.1 INTRODUCTION

In this study, foamed bitumen mixture is made up of a mixture of sandstone aggregates and foamed bitumen referred to now as “foam mix” in this thesis. In the initial stage of the production of foam mix, it is essential to determine the expansion ratio and half-life that could define the optimum characteristics of foamed bitumen. Then the next stage would be to determine the optimum binder content for the foam mix utilising the sandstone aggregates. The detailed process from the production of the foamed bitumen to the mixing and fabrication of foam mix test specimens will be explained.

The foamed bitumen is produced using specialised plant. Hence, this chapter introduces the types of laboratory-scale foamed bitumen plant and associated equipment used to produce foamed bitumen. Two types of plants were used, namely the WLB 10 and WLB 10s. The first plant was used early in the study but, due to a technical fault, it was replaced by the WLB10s. Each type is described in this chapter and their differences in terms of operations, bitumen calibrations and mixing techniques are highlighted.

7.2 LABORATORY FOAMED BITUMEN PLANT, WLB10

7.2.1 General Description

In the early stage of this research study, a laboratory-scale foamed bitumen plant, WLB10 was used to produce foamed bitumen. The plant is coupled with a Hobart mixer as shown in Figure 7.1 where the foamed bitumen is sprayed and mixed with the pre-wetted aggregates to produce bituminous foam mixes.

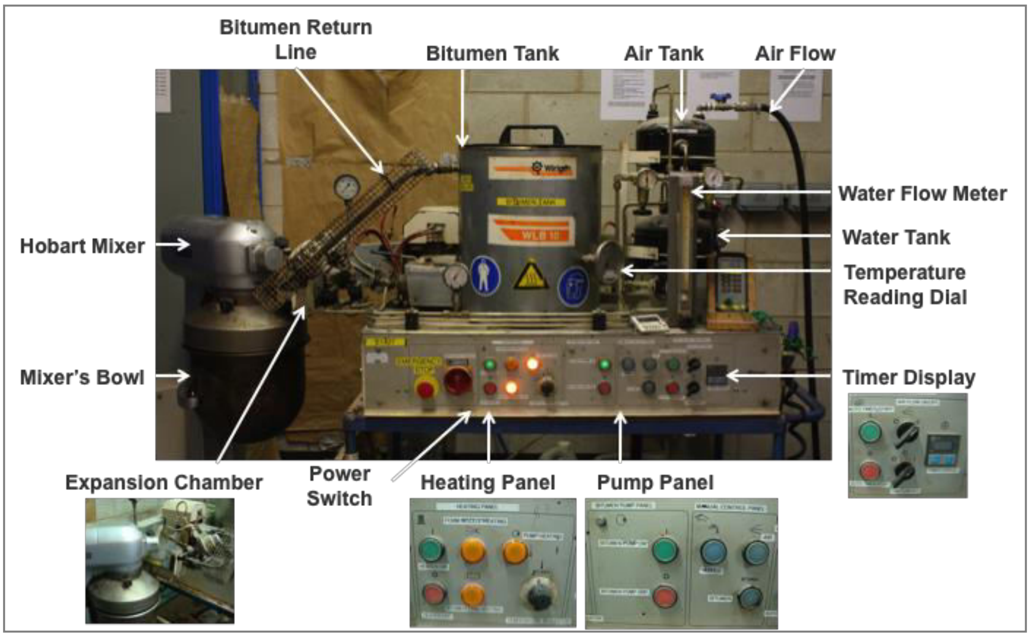


Figure 7.1 Laboratory Foamed Bitumen Plant, WLB10 coupled with a Hobart Mixer

It has three main control settings to produce different characteristics of foamed bitumen. These are described as follows:

A *temperature regulator* controls the temperature of the selected bitumen inside the rig's tank. The bitumen however must be pre-heated for at least three hours at a temperature of 160°C before pouring it inside the tank. The tank has a built-in temperature probe that displays the reading on a connected temperature dial gauge.

It is also essential to set the temperature regulator to at least 160°C to ensure the fluidity of the bitumen for pumping to start and circulate the bitumen in a pipe through the expansion chamber returning it back to the tank. If the temperature of the bitumen is not high enough to make it fluid or there is any hardened residual bitumen from previous mixing, the pump would suffer blockage. When blockages happen, a heat gun has to be applied to unblock the affected pipe and resume the circulation.

When the circulation has started, a desired bitumen temperature can be set from 150°C to 200°C. The temperature usually stabilises in about 1 to 2 hours. In order to ensure the temperature probe provides a true reading, the amount of bitumen must be adequate to fill the tank above the built-in temperature probe, otherwise the probe measures the tank's temperature rather than the bitumen. It is wise to use an external temperature probe to measure the bitumen temperature directly or when the bitumen volume decreases in the tank.

A *timer* functions to regulate the opening time of the 2.5mm nozzle valve of the expansion chamber. The desired timer setting is calculated according to the designed mass of foamed bitumen added onto the aggregates to form the foam mix. The standard flow rate of the rig is 100 grams per second. However, it is advisable to calibrate the discharge flow rate particularly when using a different bitumen grade or temperature.

A *water flow meter* is used to adjust and set the desired amount of water discharge (or foaming water content) into the hot bitumen in the expansion chamber at a desired level of air and water pressure. The water discharge is measured in litres per hour (l/hr). The appropriate setting for a desired amount of water, assuming 100grams per second of discharge of bitumen, is presented in Table 7.1.

Table 7.1 Foaming water content (FWC) in relation to the water flow meter setting

Foaming water content (% mass of foamed bitumen)	1.5	2	2.5	3	3.5	4	4.5	5
Water Flow Meter Setting (l/h)	5.4	7.2	9.0	10.8	12.6	14.4	16.2	18.0

7.2.2 Bitumen Calibration

The foamed bitumen plant normally comes with its own standard cylindrical metal container to collect the discharged foamed bitumen. The container is 280mm in diameter and holds a minimum capacity of 20 litres. It also has a red dipstick that is specially calibrated for it, with the volume occupied by 500g of neat bitumen being 1 unit measure. The dipstick has prongs attached to it at every 5 or 6 times this unit volume. It is used to measure the maximum height of the foamed bitumen, as can be seen in Figure 7.2, so that the expansion due to foaming can be measured. Unfortunately the standard container is not available in the NTEC laboratory. Hence, an alternative container was used which has a diameter of 275mm. For a non-standard container, a calibration to the bitumen volume for the container had to be made according to the ratio of 0.812g/cm^2/unit height (500g bitumen: 280mm diameter) (Wirtgen, 2002), therefore;

Mass/unit height of new cylinder (g) = 0.812 x Area of the new cylinder (cm^2)

Figure 7.2 A red dip stick with prongs measuring the maximum height of foamed bitumen

For the WLB 10, the calibration of the foam bitumen production was carried out by discharging a series of masses of bitumen into a weighed container as recorded in Table 7.2. The plant was set to discharge bitumen at the standard flow rate of 100 grams per second. However, the recorded flow rates are higher than the standard value. The extra mass is probably due to the amount of residual bitumen (shown in Figure 7.3) that was observed to still flow after delivery of the foamed bitumen was completed. Therefore, the discharge flow rate is taken as an average of 113.5 grams per second for the set employed.

Table 7.2 Discharge flow rate of bitumen at temperature of 160°C

Timer Setting (s)	Reading 1 (g)	Reading 2 (g)	Reading 3 (g)	Average (g/s)
1	116	117	120	118
2	229	231	234	116
3	338	340	343	114
4	449	441	450	112
5	542	540	549	109
Average of the discharge flow rate (g/s)				**113.5**

Figure 7.3 Residual bitumen

7.2.3 Hobart Mixer

All mixing, when the foamed bitumen was produced by the WLB10, was carried out using a 'Hobart' mixer. The mixer has three speed levels i.e 1, 2 and 3 and it is mounted under the nozzle of the foamed bitumen plant. A conventional wire whisk was used in this study as shown in Figure 7.4. The mixer's capacity is limited to produce a batch of less than 5 kg of material (ideally 4.5kg) each time the foamed bitumen is discharged to allow more room for the whisk to agitate the mixtures. It is also to avoid jamming of aggregate that would stop the machine causing delays that tend to result in the production of poor mixtures.

There are two other types of mixer paddles available such as a k-beater and a dough hook type (Millar, and Nothard, 2004). The former, according to Sunarjono (2008) is a better distributor of foamed bitumen into fine particles but its wide frame leaves a 15mm gap between the beater and the wall of the mixer's bowl, meaning that larger aggregates sizes may be degraded. The limited space may cause jamming of the mixer or the aggregates may be thrown out and hit the mixer's motor (Oke, 2010). Thus it was not recommended for use with graded aggregates that contain 20mm size fractions. The latter, a dough hook, was not preferred because of its single spiral causing segregation; hence it did not produce a better distribution of foamed bitumen (Sunarjono, 2008). Even though it was observed that the wire whisk easily clogged with foamed bitumen, this has been taken into consideration by applying a correction factor to the mass of required foamed bitumen to determine the mass of actual foamed bitumen that would mix with the aggregates.

Figure 7.4 A wire whisk type agitator used in the study

7.3 LABORATORY FOAMED BITUMEN PLANT, WLB10S

7.3.1 General Description

The laboratory foamed bitumen plant shown in Figure 7.5 is the new WLB 10s with improved operating system and electronic functions to adjust the temperature and water setting accordingly. It also offers a choice of using either of two types of nozzles:

- 2.5mm diameter for 100g/s bitumen flow
- 2.0mm diameter for 50g/s bitumen flow

This study used the 2.5mm diameter in order to have the same bitumen flow, 100g/s, as the previous plant. Therefore, the water setting rate is still the same as shown in Table 7.1. The WLB 10s was used later in the research study due to a technical fault in the WLB 10.

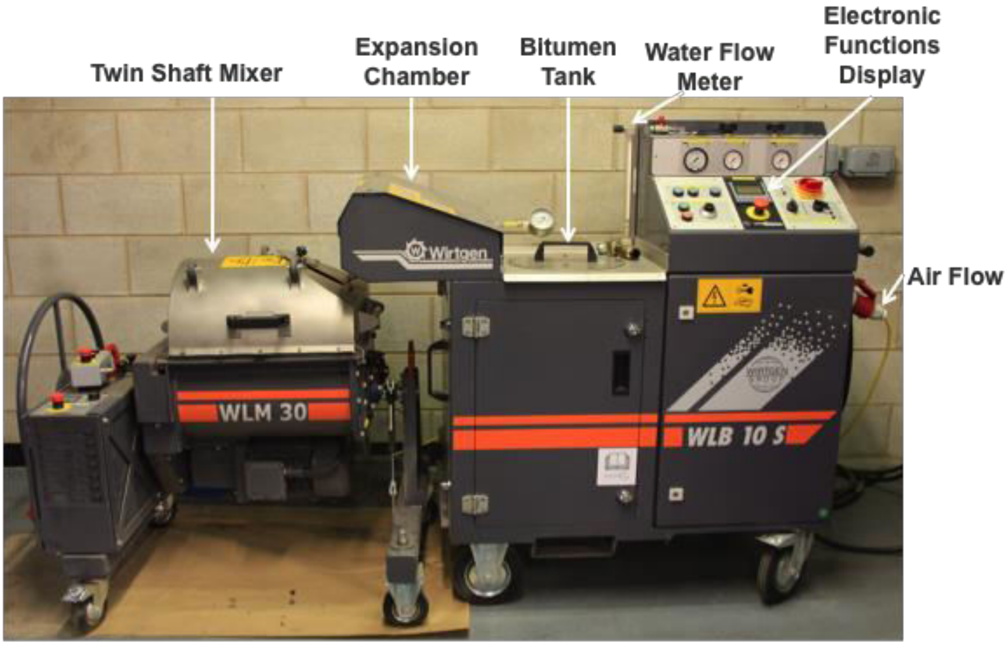


Figure 7.5 Foamed Bitumen Plant, WLB 10s, coupled with a Twin Shaft Mixer

7.3.2 Bitumen Calibration

The plant, WLB 10s, does not have a timer to regulate the nozzle's opening time for the production of the desired amount of discharged bitumen but instead a pump dial gauge is built into the system with adjustable speeds to control the discharged flow quantity of the bitumen. For this reason, the calibration was conducted by adjusting the pump speeds to produce the desired quantity of the discharged bitumen which then would be an input quantity value in terms of mass in the system. For example, using the 100g/s nozzle, the pump speed has to be adjusted to produce a 100g of discharged bitumen.

However, there were many times that the discharged mass of bitumen was not equal to 100g, although some values were close to 100g. Therefore, calculations were made to determine the apparent mass of the bitumen to input into the program so that it produced the correct mass (i.e the required designed mass).

For example, using the 100g/s nozzle, a pump speed of 270rpm produced a mass of bitumen flow of about 97g therefore, the mass of bitumen flow, x, to be input in the program would be calculated as follows:

$$x = \frac{mass\ of\ the\ required\ foamed\ bitumen\ x100}{97} \qquad \text{Equation 7.1}$$

Hence, the calibration is important to be conducted prior to the start of the mixing works and when using a new batch of bitumen or previously heated bitumen.

7.3.3 Twin Shaft Mixer

The twin shaft mixer offers a large volume of mixing with minimum capacity of 12 kg to a maximum of 30kg. Its paddles closely resemble the recycler machines as shown in Figure 7.6. It has a variable speed ranging from 1 to 100 rpm.

Figure 7.6 Twin Shaft Mixer

7.4 INVESTIGATION OF FOAMED BITUMEN CHARACTERISTICS

The foamed bitumen characteristics are determined at an early stage as part of the mix design. Therefore, it should be noted that this investigation was conducted using the old plant WLB 10. The expansion ratio and half-life were determined at temperatures of 150°C, 160°C and 170°C at different percentages of foaming water content at 1%, 2% and 3% to determine the optimum use of the foamed bitumen as can be seen in Figure 7.7 and 7.8 respectively.

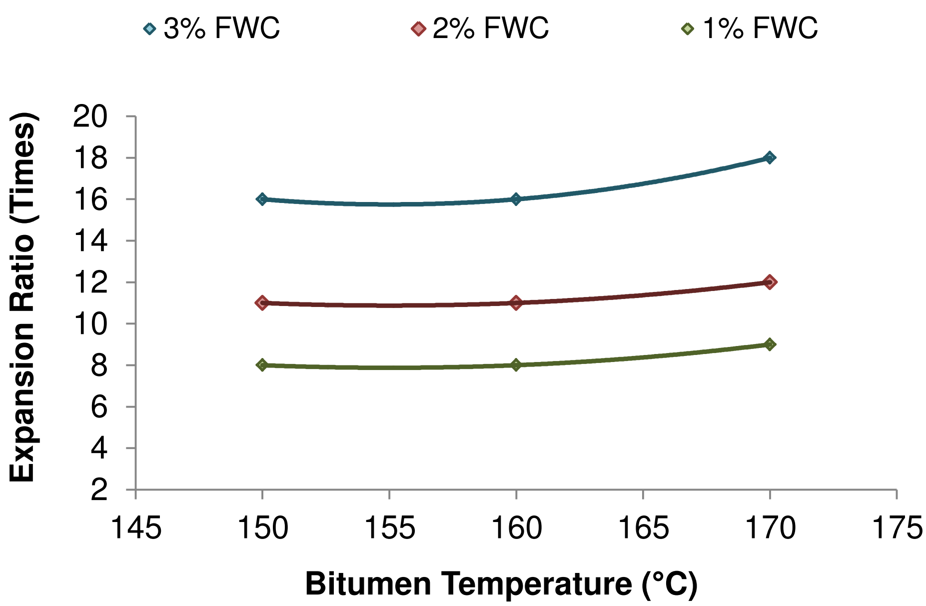


Figure 7.7 Expansion Ratio characteristics of Foamed Bitumen

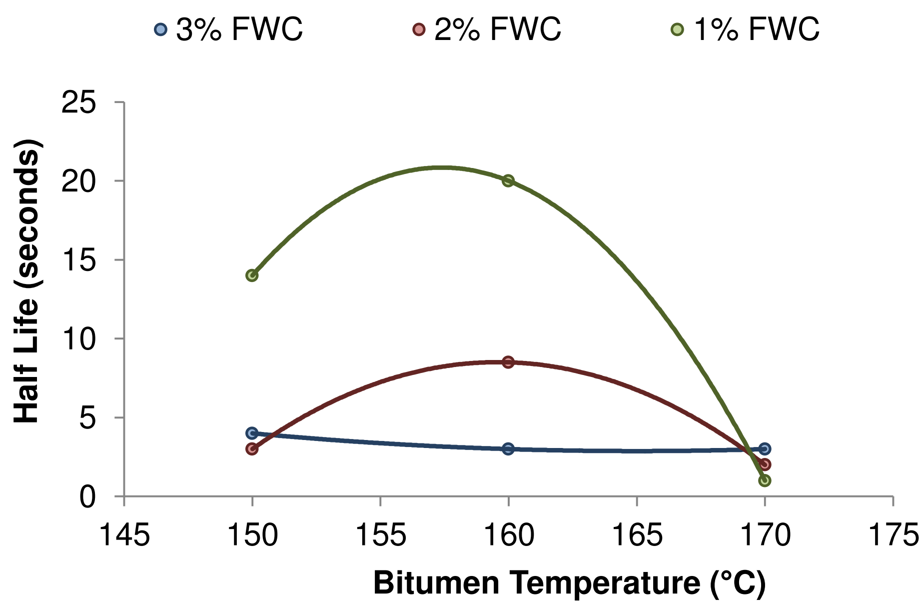


Figure 7.8 Half-Life characteristics of Foamed Bitumen

As can be seen in Figure 7.7, the expansion ratio of the foamed bitumen is less affected by the changes in bitumen temperature; however, a significant effect can be seen when the foamed bitumen is produced using different rates of foaming water contents. The increase in the foaming water content injected for the production of foamed bitumen generated more steam forming more bubbles of expanded bitumen hence, the expansion ratio increased. Therefore, for this reason, foaming water contents more than 3% were not included. This decision was also due to the height of the cylinder used for collection of the foamed bitumen being less than the height of the standard cylinder.

On the other hand, the Figure 7.8 indicates that the half-life of the foamed bitumen is highly affected by both the changes in the bitumen temperature and foaming water content. It shows a maximum peak when the foamed bitumen is produced at a bitumen temperature of 160°C hence, this temperature was selected as the optimum.

The foaming characteristics of foamed bitumen produced at this temperature, expansion ratio and half-life, were plotted in a graph as shown in Figure 7.9. The optimum foaming water content was found to be about 2%. Hence, the expansion ratio and half life of the foamed bitumen were 11 and 9s. Both these values were higher than the minimum values recommended (Wirtgen 2012 since the aggregates were conditioned to 30°C, simulating the average temperature of Brunei Darussalam. These values have been used to set the foamed bitumen plant as summarised in Table 7.3.

It was observed that the accuracy of the visual observation in the foaming experiments is usually reliant on the operator's experience and judgement. It is rather subjective, especially when measuring the half-life. The collapse time might be more rapid and the judgement of the half volume level might be different with different operators. However, three repeat tests were conducted for a set of temperature and foaming water contents to minimise the errors.

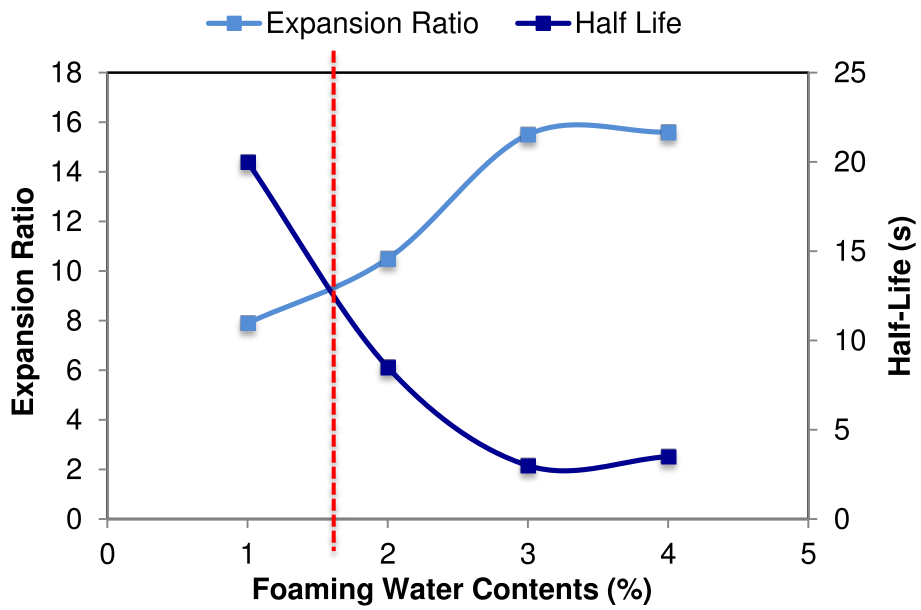


Figure 7.9 Optimum characteristics of foamed bitumen produced at bitumen temperature of 160°C

Table 7.3 Foamed bitumen plant's settings

Setting Parameters	Values
Expansion Ratio (Times)	10
Half Life (s)	9
Bitumen Temperature (°C)	160
Water Flow Rate (%)	2
Discharge Time (g/s)	113.5

As mentioned earlier, there are three important control settings of the foamed bitumen plant, WLB10 to be adjusted before mixing is carried out. They are bitumen temperature, water flow rate and discharge time and they must be selected to produce the preferred set of foamed bitumen characteristics. For the WLB10s, the discharge time was not necessary because it has an adjustable pump speed as mentioned in the earlier section to control the quantity of the discharged foamed bitumen. In this study, the settings were adjusted in accordance with the results previously described and are summarised in Table 7.3.

7.5 FOAMED BITUMEN MIXTURE SAMPLE PREPARATION

7.5.1 Conditioning

The selected graded aggregate was conditioned in an oven at 30°C for at least three hours prior to production of foamed mix. A temperature of 30°C was selected as being a representative of ambient temperature (instead of 20°C typically used in the U.K) based on the average temperature recorded for Brunei Darussalam. All steel moulds for compaction were also conditioned at this temperature.

7.5.2 Mixing Techniques

The aggregate was freshly taken from the oven and placed in the bowl of the mixer. It was pre-wetted with the selected mixing moisture content (percentage by OMC of raw aggregates). When using the Hobart mixer, the pre-wetted aggregate mixtures were mixed manually by hand for a minute or until a uniform distribution of moisture could be visually observed. However when using the twin shaft mixer, all aggregates were mixed with water in the drum mechanically due to the large mass used. The mixer was then fixed to the main foamed bitumen plant and the mixer was allowed to mix for a minute. Immediately after that, without stopping the mixer, the foamed bitumen was discharged and another one minute was allowed to mix the pre-wetted aggregates with the discharged foamed bitumen.

7.5.3 Riffling

Following the mixing, the wet loose foam mix was again manually mixed and poured onto a tray, for a small mass, and into a container for a large mass. The mix was then riffled to distribute samples into moulds. The riffling stages required for different mixers are illustrated in Figures 7.10 and 7.11. The mould, which was taken from the oven at 30°C, was then immediately filled with the foam mix and was ready for compaction. The period between mixing and compaction was made as short as possible for all samples. During this period, it is important to cover the remaining loose foamed mix to avoid losing excessive moisture as the mix begins to dry out.

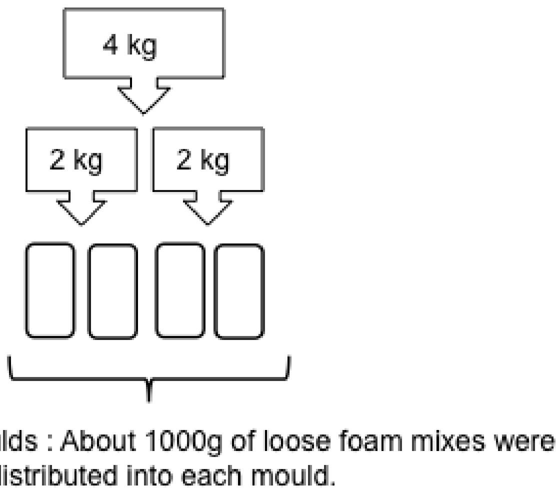


Figure 7.10 Riffling stage when using the Hobart mixer

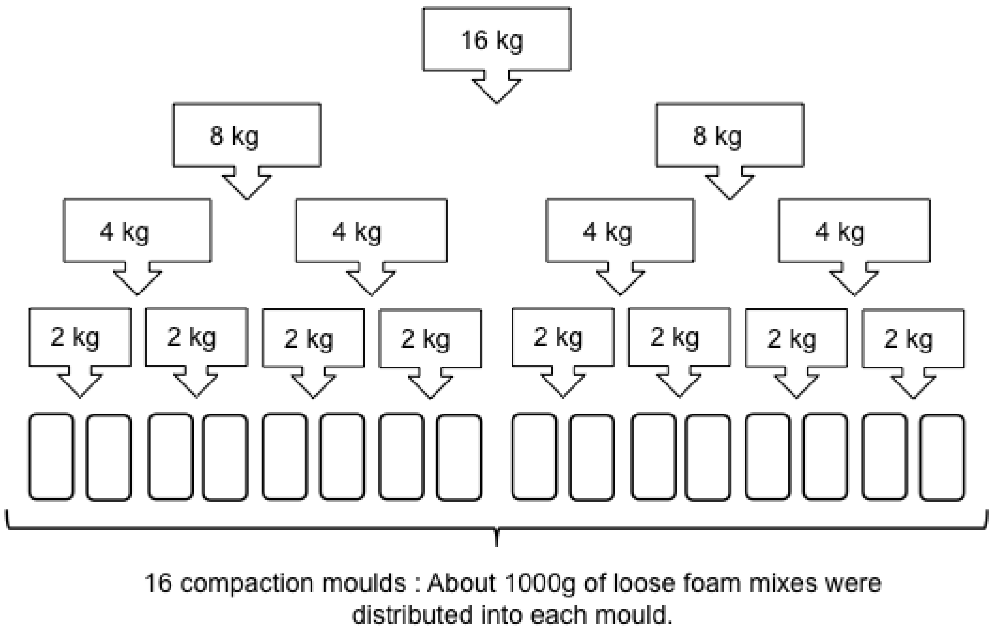


Figure 7.11 Riffling stages when using the Twin Shaft Mixer

7.5.4 Specimen Fabrication

All specimens were fabricated and compacted using a gyratory compactor. All cylindrical test specimens with a diameter of 100mm were produced at the same energy level of a target number of gyrations, 100. The gyratory compactor was set using a pressure of 600kPa and an angle of gyration of 1.25 degrees (BS EN 12697-31:2007).

All the compacted specimens were left in their moulds for 24 hours to gain cohesion at a temperature of 30°C prior to extracting the specimens. It is important to grease the moulds before filling them to ensure that specimens could be easily extruded and the mould cleaned again. Great care must be taken in de-moulding the wet or uncured specimens in order to avoid breakage. After extraction, all the test specimens underwent curing according to the adopted curing conditions as explained in Chapter 6.

7.6 DETERMINATION OF OPTIMUM BINDER CONTENT

In order to design a foamed bitumen mixture, optimum binder content must be defined. The designed graded aggregates were mixed with four levels of foamed bitumen contents: 2%, 3%, 4% and 4.5% by mass of dried aggregates, under dry and wet curing conditions. The 4.5% was selected because the next interval, 5%, was not economically feasible.

In this study, the stiffness modulus was used as the property of the foam mix to identify the optimum binder content, being the one that would produce the maximum stiffness modulus value at a particular curing condition. Four replicates were prepared for each mix and all test specimens were initially dry cured. They were then firstly tested under dry ITSM and the results were plotted as shown in Figure 7.12 to determine the dry-cured optimum binder content. Following that, these same specimens were wet cured by full immersion in a water bath tank for 24 hours at 30°C. The wet-cured specimens were all tested under ITSM and the results were plotted in Figure 7.13 to determine the wet-cured optimum binder content. It should be noted that prior to the wet-cured ITSM test, the surface of the wet-cured specimens was cleaned using a towel producing a saturated surface dry condition.

Comparing Figures 7.12 and 7.13 the dry-cured optimum binder content, 3%, was 1% lower than the wet-cured optimum binder content, 4%. The mix with dry-cured optimum binder content exhibits about 3000MPa while unfortunately, under wet curing, the stiffness of the same mix reduced to a low stiffness, approximately 300MPa, lower than the ones with wet-cured optimum binder content, 350MPa. In general, the dry-cured specimens pose higher stiffness values than those of the wet specimens. Overall the wet-cured specimens have stiffness values lower than 400MPa with, at 2% binder content, zero stiffness. It can be observed that the designed foamed bitumen mixtures in this study had high moisture sensitivity.

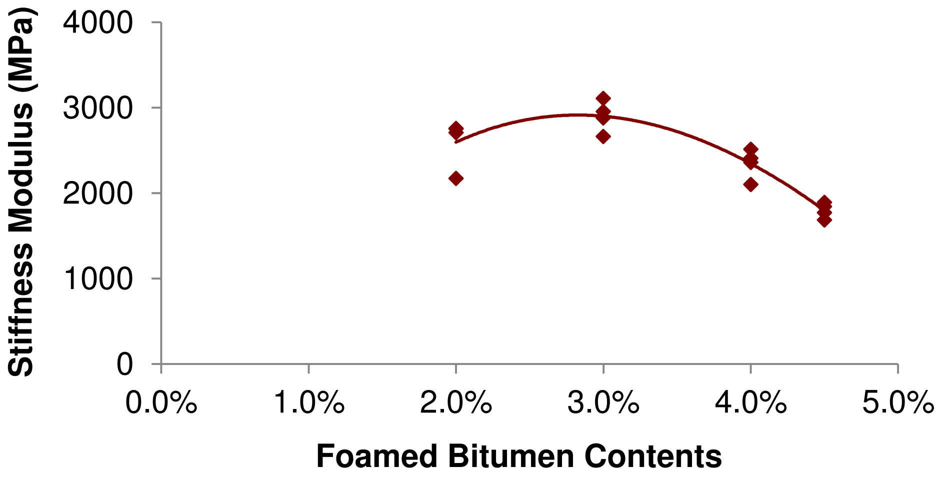


Figure 7.12 Dry-cured optimum foamed bitumen binder content

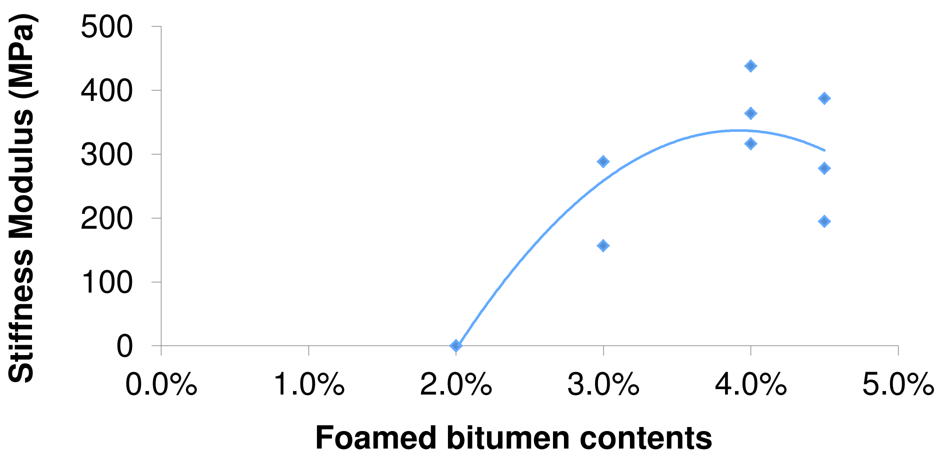


Figure 7.13 Wet-cured optimum foamed bitumen binder content

Furthermore, it was observed that some test specimens had gone beyond the target horizontal deformation of 3μm, particularly the wet specimens, so tests were repeated across different diameters of that specimen and the stiffness modulus results were taken as an average value. A few test specimens were damaged during the test and the stiffness modulus was taken as zero.

7.7 EXPERIMENT DESIGN

On the basis of the optimum binder content results, it appears that the foamed bitumen mixtures had high moisture sensitivity. This could be due to the utilisation of low quality aggregates that impaired performance in a wet-cured condition as compared to other aggregates such as limestone (Sunarjono, 2008) and, being a partially unbound material, this perhaps caused the sensitivity to increase.

Therefore, an experiment was designed to investigate ways to improve the foamed bitumen mixtures using two main additives, which have different roles, as explained in Chapter 5. The experiment design is presented in a flow chart as shown in Figure 7.14. Both foam mix containing the dry-cured and wet-cured optimum binder contents, 3% and 4% by mass of dried aggregates, were experimented for their sensitivity at three selected MMC levels, 60%, 70% and 80% (of the OMC of untreated aggregates, 6%) as well as to determine the optimum MMC of foam mix when the stiffness modulus was at its maximum. The certainty in choosing the right binder content is important in order to obtain the best mix that provides consistent performance and, preferably, will maintain the bituminous characteristics of foam mix being flexible and ductile rather than having a rigid, cementitious characteristic.

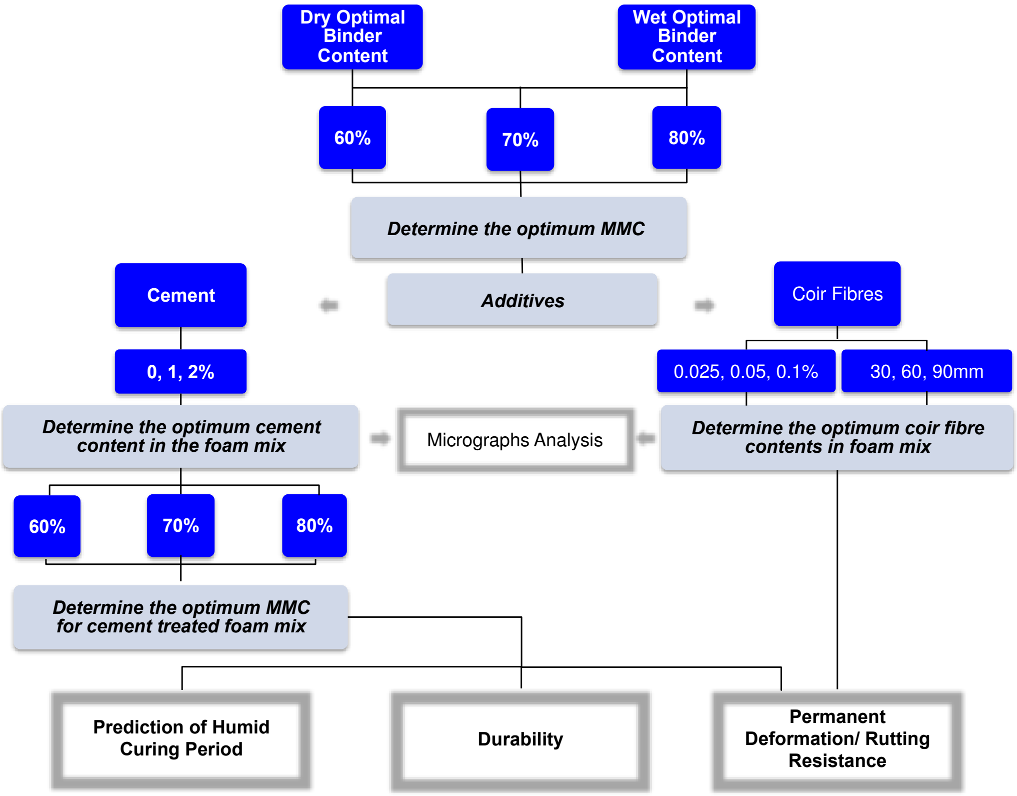


Figure 7.14 Experiment design

Due to this reason, when cement was used as an additive co-treatment agent, a careful evaluation of the stiffness response of the foam mix was required. Therefore, the investigation included the effect of cement on the foam mix containing low and high bitumen contents produced at a MMC equal to 70% of OMC as this had been indicated to be the optimum MMC.

On this basis the foam mix, 4FB (see Table 7.4), was confirmed to be a consistent mix and dominated by its bituminous characteristics. A more extensive experiment was carried out on the effect of cement content at three levels, 0%, 1% and 2% (by mass of dried aggregates) on the mix produced at three selected MMC levels. The objective was to investigate the sensitivity of the cement-treated foam mix to the MMC levels. Once the optimum MMC was identified, further laboratory experimentation and evaluation was followed such as the development of stiffness modulus under humid curing, and the durability of foam mix due to the effect of alternate wet and dry cycles on the stiffness modulus property.

Separately, a preliminary investigation on the effect of coir fibres was performed only on the foam mix, 4FB (see Table 7.4), in terms of its stiffness modulus and tensile strength. The effect of the coir fibre contents and lengths would be included in the investigation to determine the optimum mix. The effect of coir fibre lengths was further investigated by using the RLAT to find out if the coir fibres-reinforced foam mix would resist permanent deformation.

Table 7.4 presents the coding scheme of the foam mixes to be used for simplification in presentation and discussion of the results. The first digit in the mixture codes represents the percentage of the foamed bitumen binder content. This is followed by the letter symbols such as FB i.e, the 4FB code is bituminous foam mix containing 4% foamed bitumen binder content. If these foam mixes are treated with other additives, the succeeding codes consist of a first digit standing for the percentage of the additives and initials indicating the additive type. For example, the 1C code means 1% cement as a mass proportion of the dry aggregates. Any other additives are coded in a similar way with their initials such as Coir Fibres (CF), Hydrated Lime (HL) and Wet Fix (WF).

Table 7.4 Coding scheme of the foam mixes (for cylindrical test specimens)

Mixture Codes	**Descriptions**	**Thesis's Chapter**
3FB	Foam mix containing the dry optimal binder content of 3% (by mass of dry aggregates)	8*
3FB1C	As 3FB but with addition of cement, 1% by mass of dry aggregates)	8*
3FB2C	As 3FB but with addition of cement, 2% by mass of dry aggregates)	8*
4FB	Foam mix containing the wet optimal binder content of 4% (by mass of dry aggregates)	8*, 10, 11, 12
4FB1C	As 4FB but with addition of cement, 1% by mass of dry aggregates)	8*, 10, 11, 12
4FB2C	As 4FB but with addition of cement, 2% by mass of dry aggregates)	8* 10, 11, 12
4FB1.5HL	As 4FB but with addition of hydrated lime (1.5% of loose foam mix)	10, 11
WF4FB	4FB made using pre-blended base bitumen with Wet Fix	10, 11
WF4FB1.5HL	As WF4FB but with the addition of hydrated lime (1.5% of loose foam mix)	10, 11
WF4FB1C	As WF4FB but with the addition of cement (1% of loose foam mix)	10, 11

*Note: * The foam mixes in this chapter were produced using the old foamed bitumen plant, WLB 10 and mixed using the Hobart mixer*

The 4FB and 4FB1C specimens mentioned in Chapter 8 were manufactured using the WLB10. Unfortunately a fault developed in the WLB10 that was not repairable, therefore the 4FB and 4FB1C studied in Chapters 10 and 11 had to be manufactured using the new plant, WLB10s. Other foam mixes, coir fibre-reinforced foam mixes and test slab specimens are explained separately in Chapters 9 and 12 respectively.

7.8 SUMMARY

The production of foamed bitumen used two types of plant that have different operational systems and mixers. Due to a fault in the WLB10, it was eventually replaced by a new improved plant, the WLB10s. Therefore, the laboratory test program was adjusted as results became available.

The optimum foamed bitumen characteristics for bitumen 90pen were produced using the foamed bitumen plant, WLB10, when the bitumen temperature was set to 160°C and the foaming water content was at 2%. The setting parameters would be used throughout the study; however slight changes were anticipated when using a different foamed bitumen plant.

The optimum foamed bitumen binder contents under dry and wet curing were not the same, 3% and 4% respectively. Therefore, both of these foam mixes were considered for further evaluation to determine the optimum mix design, particularly as regards the effect of MMC and cement on them to find the best consistent mix.

CHAPTER 8 STIFFNESS BEHAVIOUR OF BITUMINOUS FOAM MIXES

8.1 INTRODUCTION

Findings from Chapter 7 have indicated that the optimum foamed bitumen binder contents are 3% and 4% under dry-cured and wet-cured conditions respectively. In light of the weakness of these foam mixes to perform in a wet environment as discussed in Chapter 7, a co-treatment agent, cement, was introduced into the mixes. Therefore, this chapter presents laboratory experimentations on two main categories of foam mixes, non-cemented foam mixes and cement treated foam mixes. In this case, moisture sensitivity of these foam mixes would be the main attention in this experimentation. and it focuses mainly on the stiffness response of these foam mixes at three MMC levels, 60% , 70% and 80% (of OMC) as illustrated in the experiment design flowchart in Figure 7.14. The range of MMC levels was intended to be increased but the fault of the WLB10 did not allow more foam mixes to be manufactured at that time. Moreover, when the WLB10 was replaced with the new plant, the WLB10s, the foam mixes manufactured using the new plant would not be directly comparable with the foam mixes studied in this Chapter. Therefore, the discussions in this Chapter are limited to the results obtained on the foam mixes produced by the WLB10 with the three MMC levels.

8.2 TEST MATRIX

The main aim of this laboratory experimentation was to choose the right binder content to obtain the best foam mix that provides consistent performance and, preferably will maintain the bituminous characteristics of flexibility and ductility. Therefore, the objectives of the experimentation were as follows:

- To choose the optimum MMC, at the maximum stiffness, for bituminous foam mixes, 3FB and 4FB, by investigating the stiffness response of these foam mixes produced with a MMC equal to 60%, 70% and 80% of OMC.

- Investigation on the effect of low and high binder contents (3FB and 4FB mixes) in cement-treated bituminous foam mixes at the indicated optimum MMC level, 70% of OMC.

- Further evaluation of the selected foam mix, 4FB, being the best mix, together with the equivalent cement-treated foam mixes, 4FB1C and 4FB2C, for their sensitivity to the MMC levels.

Thus, there were a total of fourteen mixtures and four replicates were fabricated for each mixture as shown in Table 8.1. ITSM tests were carried out on all foam mixes after they were cured under dry and wet conditions. For road base application in Brunei Darussalam, wet conditions can be expected due to the frequent rainfall. Hence, the presence of moisture in foam mixes is expected to be critical to its performance. In this case, the study also considered the optimisation of MMC for the bituminous foam mixes.

Table 8.1 Test matrix for the foam mixes (see Table 7.4 for coding scheme)

Foam Mixes	MMC 60%	MMC 70%	MMC 80%
3FB	X	X	X
3FB1C		X	
3FB2C		X	
4FB	X	X	X
4FB1C	X	X	X
4FB2C	X	X	X

8.3 BITUMINOUS FOAM MIX RESPONSE

The results obtained from the analysis of the foam mixes, 3FB and 4FB, at three MMC levels can be compared in Figure 8.1. It can be seen that the 3FB is much more sensitive to the changes in MMC than the 4FB. In contrast, the stiffness curve for 4FB shows a fairly low effect of MMC when assessed under both dry and wet condition. This result shows that the 4FB foam mixes are less sensitive to the action of water than the 3FB mixes, which are more porous due to the low binder content (Brennen, 1981).

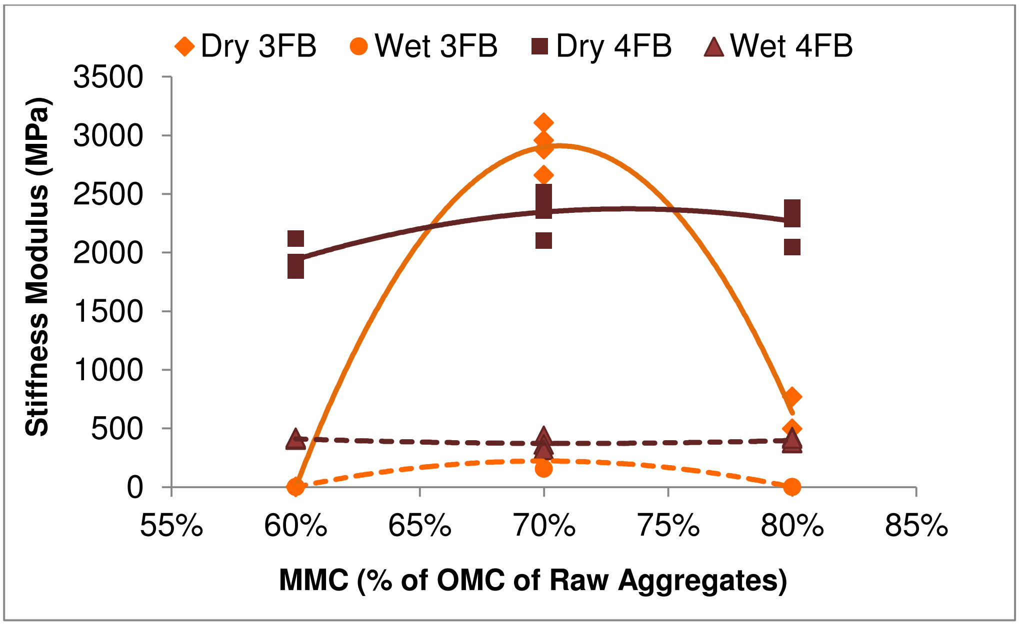


Figure 8.1 Stiffness modulus at different MMC levels for foam mixes, 3FB and 4FB

At 3% foamed bitumen content (3FB) and at a MMC equal to 60% of OMC, the dry cured mix was observed to be non-uniformly moist, ‘spotty’, and that it immediately showed signs of fast drying. After being compacted and cured, the specimen started to show signs of crumbling, particularly during handling. It further failed almost immediately under dry ITSM testing; demonstrating that this mixture has almost no measurable stiffness.

On the other hand, the foam mix, 3FB, at a MMC that was 80% of OMC, had a low stiffness value under dry conditions, but was completely decomposed while it was soaked in the water bath at 30°C for 24 hours. In contrast, the 4FB performed better, resulting in a fairly constant stiffness at all MMC values.

When the foam mixes were produced using a MMC equal to 70% of OMC, the dry cured foam mixes, 3FB, reached their peak stiffness values of about 500MPa higher than those of the 4FB. The lower amount of foamed bitumen content in the 3FB mix created a less deformable but brittle mixture. The high stiffness under dry conditions could be attributed to the low VMA (Voids in mixed aggregates) in the 3FB mix. However, the mix tends to damage easily by the action of water in wet conditions. This might be due to the low foamed bitumen coating of the aggregates. In dry conditions, the sensitivity to the MMC level made the mixtures prone to failure except for MMC equal to 70% of the OMC (of raw aggregates) in which the 3FB mix performed well indicating that the MMC was appropriate to produce a good mix.

In contrast, the 4FB had a fairly constant performance at all selected MMC levels. The 4FB foam mixes contained sufficient foamed bitumen coating the aggregates so that it could generate a more flexible and ductile mixture, hence resulting in a lower but more reliable stiffness. Therefore, the results of this study indicate that the foamed bitumen content, 4% (by mass of dried aggregates) is appropriate to select as an optimum, as determined earlier in Chapter 7 (see Figure 7.13), and that MMC 70% was also appropriate.

8.4 CEMENT TREATED FOAM MIX RESPONSE

8.4.1 Effect of Binder Contents on the Cement Treated Bituminous Foam Mixes

From section 8.3, it can be observed that the overall performance of the foam mixes in wet cured condition was poor. Thus a co-treatment agent, cement, was introduced into the foam mixes. Due to the hydration reaction of cement, it was important to evaluate the stiffness response of both foam mixes, 3FB and 4FB, which were produced at MMC 70% where both mixes achieved their maximum stiffness.

The stiffness modulus results of the two foam mixes are reported in Table 8.2, under dry and wet cured conditions. These data are plotted against cement content in Figure 8.2, demonstrating the powerful effect of cement.

Table 8.2 Stiffness modulus of foam mixes with and without cement

	Cement Content (%)					
	0		**1**		**2**	
Foamed Bitumen Content (%)	*Dry ITSM (MPa)*	*Wet ITSM (MPa)*	*Dry ITSM (MPa)*	*Wet ITSM (MPa)*	*Dry ITSM (MPa)*	*Wet ITSM (MPa)*
3	3108	-	4669.5	3453.5	7854.5	*
	2956	-	3517.5	2668.5	5574	4642.5
	2661	288	4665.5	2628.5	8012.5	5396.5
	2881	157	4035	2756.5	8018.5	5426.5
4	2514	438	5414	3424	4513	*
	2359	364	5381	3492	4950	3897
	2409	316	4884	3257	5301	4206
	2102	-	5255	3108	5487	4165

Note: * Specimens were used for SEM analysis

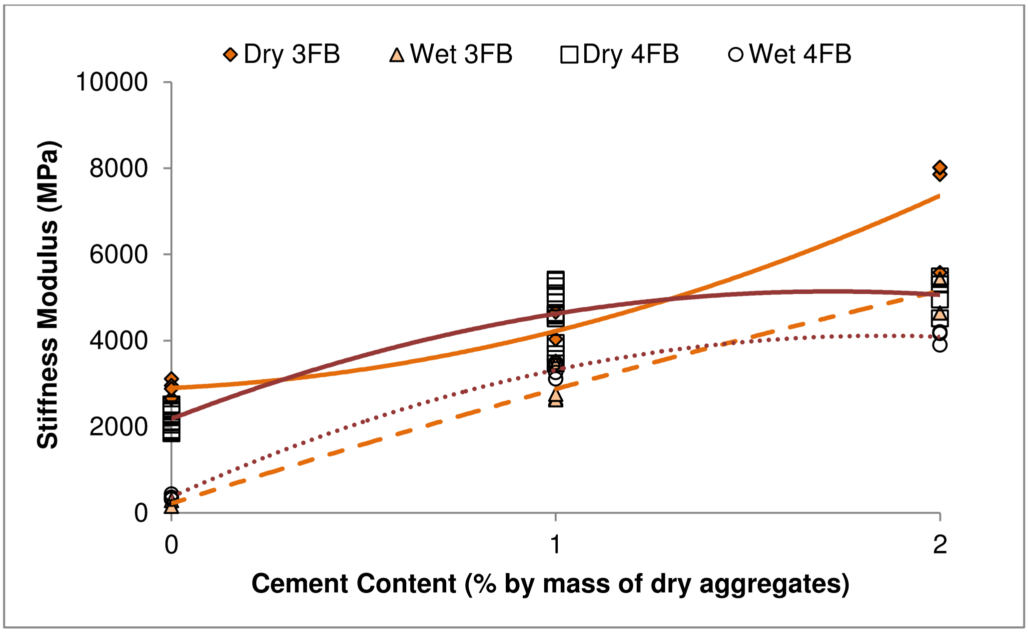


Figure 8.2 The effect of cement on bituminous foam mixes, 4FB and 3FB, produced at MMC of 70%.

For foam mix containing 3% foamed bitumen content, 3FB, the curves show an almost linear relationship between the stiffness modulus and cement content. Even though the dry cured foam mix without cement reached a stiffness of 3000MPa, it was severely damaged after undergoing water conditioning and eventually lost all its stiffness.

In contrast, the stiffness modulus curve for foam mix, 4FB, shows a gradual increase from 0% to 1% cement content and it then stabilises as more cement is added.

When 1% cement content was added, the dry-cured stiffness modulus increased to about twice the initial values, for both foam mixes. When the mixes underwent wet curing, their stiffness only decreased to about 3000MPa, which is still higher than the dry cured 4FB, without cement, value. However, the cement treated foam mix, 4FB1C, showed slightly higher stiffness than that with 3% foamed bitumen content, 3FB1C. An average stiffness modulus of 5000MPa was recorded and was maintained at about 3500MPa after undergoing wet curing.

At 2% cement content, the stiffness modulus of the 3FB mix increased dramatically to average more than 6000MPa and was maintained at about 5000MPa under wet cured conditions. In contrast, the stiffness modulus of the 4FB mix experienced only a slight increase in stiffness modulus at the same cement content.

The results have shown that as the cement content increases so does the stiffness modulus value of the foam mixes, but this increase is associated with certain features. In particular, the behaviour of the mixes containing 3% foamed bitumen content starts to act more like a weak concrete. The mixes containing 4% foamed bitumen content, however gain stiffness less. When the poor wet cured behaviour and the response to cementation of the 3FB mixes are compared to those of the 4FB mixes it appears that the 3FB mix relies, significantly, for its strength on aggregate interaction which can be readily modified by cement. On the other hand, the 4FB mix appears to rely much more on foam mastic bonding so is more resistant to water when untreated with cement and less responsive to cement treatment.

The addition of cement was observed to cause a significant change in the behaviour of all the foam mixes, particularly after they had undergone water conditioning, as compared to the change caused to the dry-cured mixes. It is surmised that cement can give benefit in a wet environment (as this will activate its chemical reaction and the formation of C-S-H gels) with the cementation replacing any bituminous bonds that might have been damaged due to water infiltration.

8.4.2 Relationship of Stiffness Modulus and Dry Density of Foam Mixes with Increasing Cement Contents at Different MMC

Further evaluation of the bituminous foam mix, 4FB, together with its cement-treated mixes, 4FB1C and 4FB2C were conducted and this section analyses the results obtained. The results of stiffness modulus and dry density of foam mixes were plotted against MMC as shown in Figure 8.3, 8.4 and 8.5.

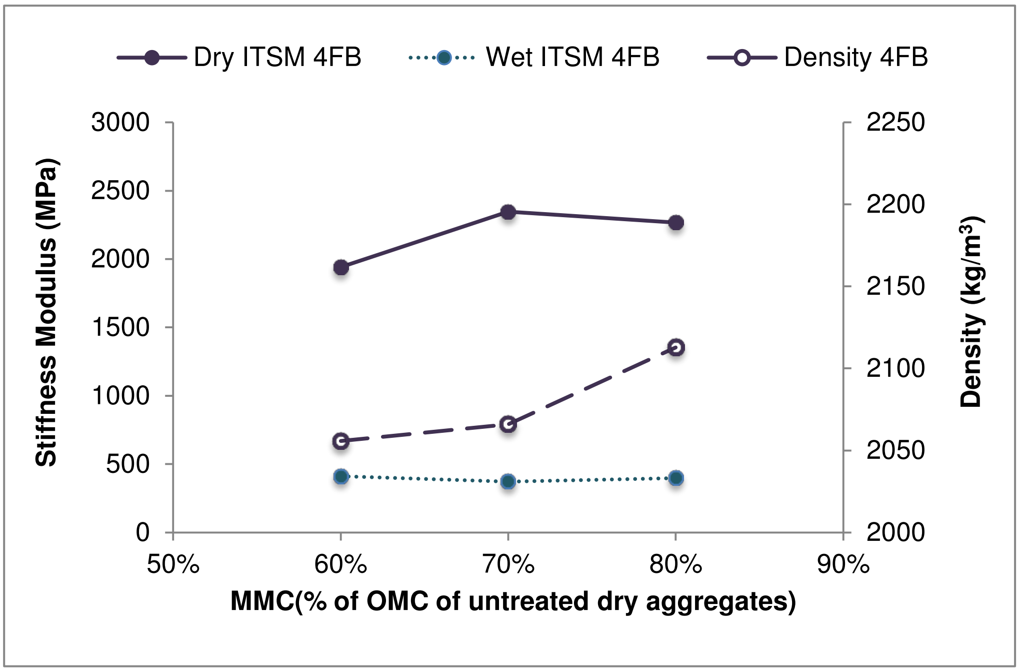


Figure 8.3 ITSM results and dry density of foam mixes, 4FB (No cement)

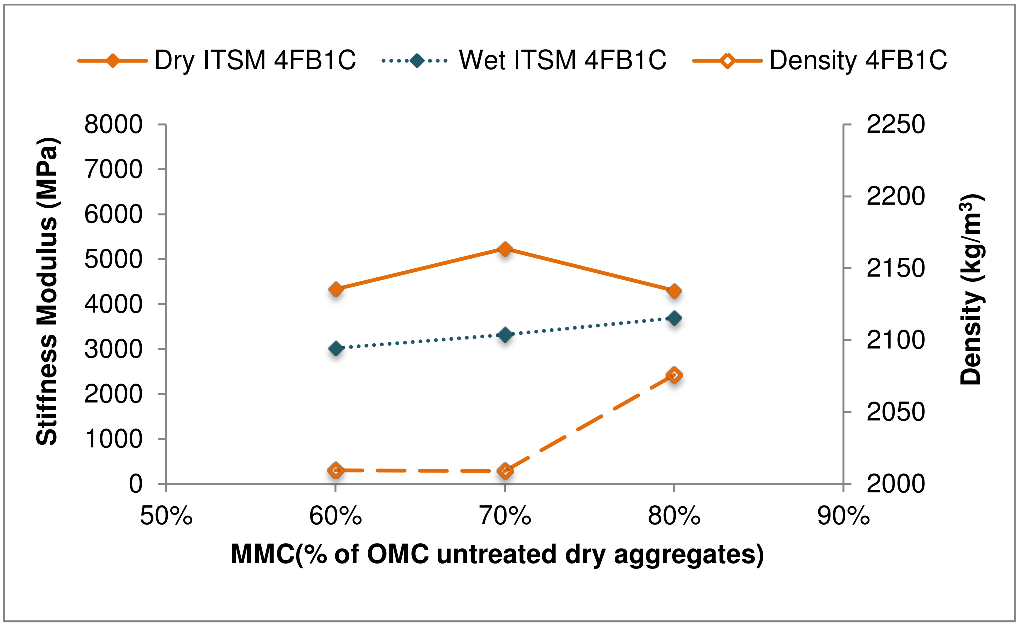


Figure 8.4 ITSM results and dry density of foam mixes, 4FB1C (with 1% cement content)

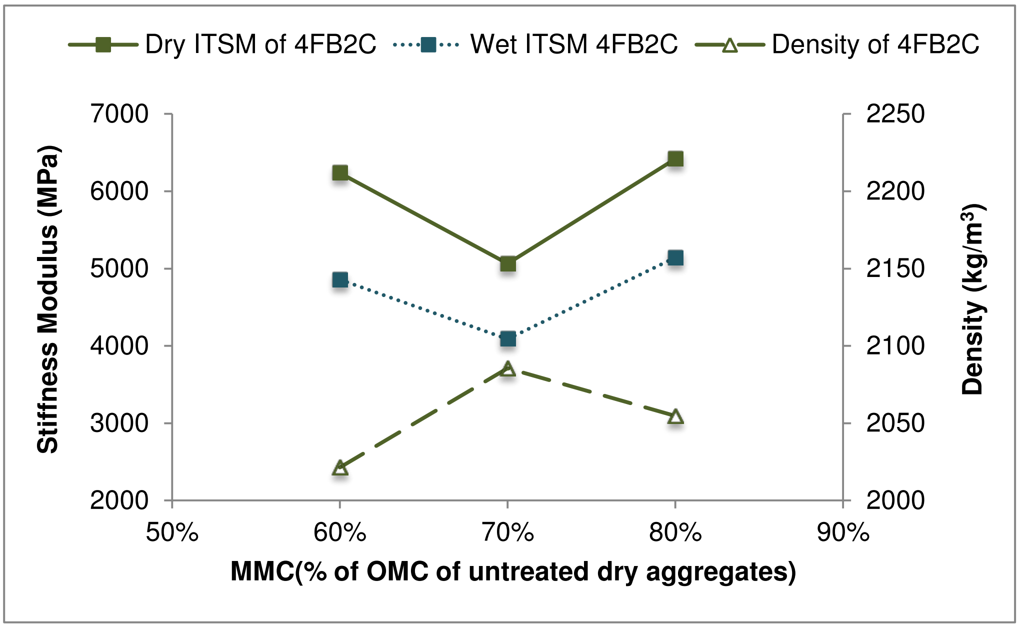


Figure 8.5 ITSM results and dry density of foam mixes, 4FB2C (2% cement content)

It can be seen in Figure 8.3 that the stiffness modulus of 4FB appears to reach a peak value at a MMC equal to 70% of OMC. However according to the dry density results, the MMC equal to 80%% of OMC seems to yield the preferred 4FB mix. The low stiffness for 4FB mix at MMC 60% could be due to the ravelling of aggregates that reduced the aggregate skeleton integrity.

As to wet ITSM results, the 4FB behaviour did not change significantly as the MMC increased from 60% to 80%. As more moisture was introduced to the foam mixes during curing, the samples became vulnerable to disintegration.

Figure 8.4 presents the test results for 4FB1C. When cement is introduced at 1% by mass of dry aggregates in the 4% foam mixes, 4FB, the results show improved stiffness modulus, particularly at 70% of OMC. An optimum combination of dry and wet ITSM results was also exhibited at 70% of OMC. However, when the MMC increased to 80% of OMC, the 4FB1C gained in dry density.

As shown in Figure 8.5, the stiffness modulus of the 4FB2C mix was lowest when produced with MMC equal to 70%, both under dry and wet conditions. The dry density of the 4FB2C mix appeared to reach a peak value with MMC equal to 70% of OMC but reduced when lower and higher MMC was added, 60% and 80% respectively. Perhaps the addition of cement, which represents additional fines in the mix caused the effect of MMC on the 4FB2C mix to be different from the other foam mixes, 4FB and 4FB1C.

The foam mixes made using a MMC of 60% give the least reliable results due to their more scattered data particularly when more cement is added. Although the foam mixes produced using a MMC 80% are apparently quite reliable, it perhaps would be more suitable for foam mixes when more cement is added. Thus, it would form stiffer mixes due to the formation of cementitious bonds which are not the interest of this study.

The consistency of the stiffness response of all the foam mixes produced at MMC 70% can be observed in this study. It suggests that the MMC is adequate to produce the most reliable mix particularly with the addition of 1% cement which was the best mix based on this study.

8.4.3 Relationship of Dry ITSM and Dry Density of Foam Mixes with Increasing Cement Contents at each MMC level

Figure 8.6 presents the stiffness modulus and dry densities of foam mixes produced with MMC equal to 60% of OMC with increasing cement contents. Amongst all foam mixes, the 4FB mix exhibits the lowest stiffness value ranging from 1500MPa to 2100MPa. The dry density results were rather low and varied.

With the introduction of cement (1% of dry mass of aggregates), the foam mixes, 4FB1C, became stiffer with stiffness values between 3000MPa and 5500MPa. Slight variation in the dry density results can be seen, around 2000 kg/m^3.

With even more cement added, the stiffness increased to about 7000MPa although one specimen is clearly atypical, perhaps indicating a production problem. This could be dependent on the aggregate interlock with the increase in fines (cement) as well as a greater bonding caused by the cement gels.

The variability of the stiffness modulus for the low MMC mixes may be due to drying effects as a small amount of evaporative loss from a low amount of moisture might be expected to have a large effect during the manufacturing of specimens. It was also observed that the resulting specimens did not remain intact particularly the 4FB mix after they had been extracted from their moulds. Ravelling of aggregates seemed to be normal and handling the specimens often caused more damage. The variations in the stiffness modulus results between the replicates for the cement-treated foam mixes, 4FB1C and 4FB2C, may not only due to the drying effects but also the rapid cement hydration processes, resulting in the effective compaction moisture being sometimes insufficient when the MMC is only 60%.

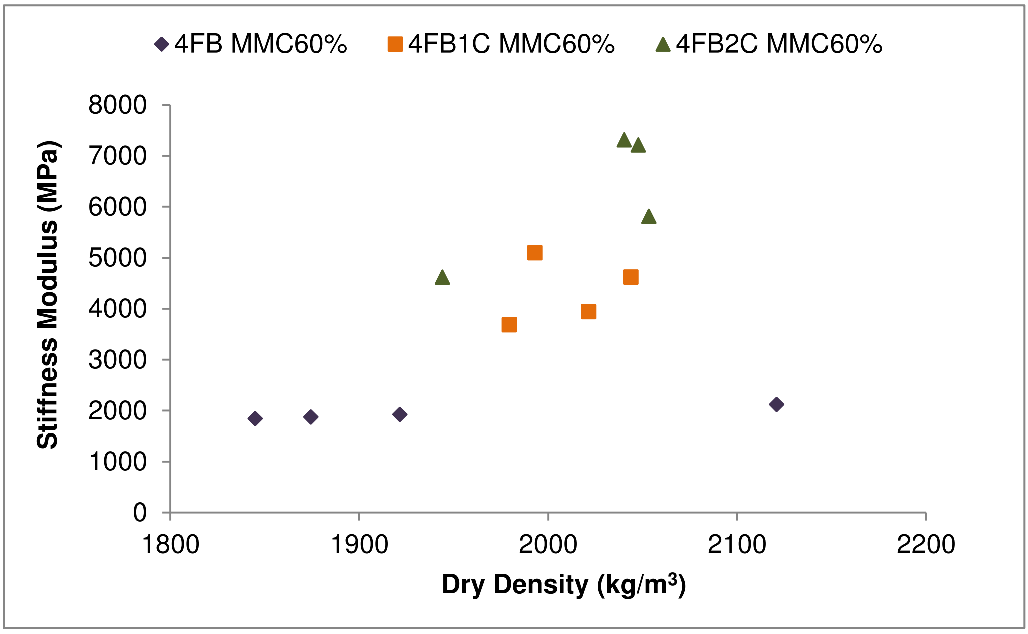


Figure 8.6 Stiffness modulus against dry density of foam mixes with increasing cement contents at MMC 60%

Figure 8.7 presents the stiffness modulus and dry density results for foam mixes produced using MMC of 70% with increasing cement content. It was observed that the samples were more workable with adequate lubricating effect for better binder dispersion. Comparing to the previous results, the stiffness values of the 4FB apparently improved slightly reaching between 2000MPa and 2700MPa.

When cement is added, the stiffness values of the foam mixes increased. It can be seen that the 4FB mix exhibits a stiffness modulus of 5000MPa. However, the stiffness did not increase much when more cement, to a total volume of 2%, was added suggesting that the stiffness was still dominated by its bituminous mastic points.

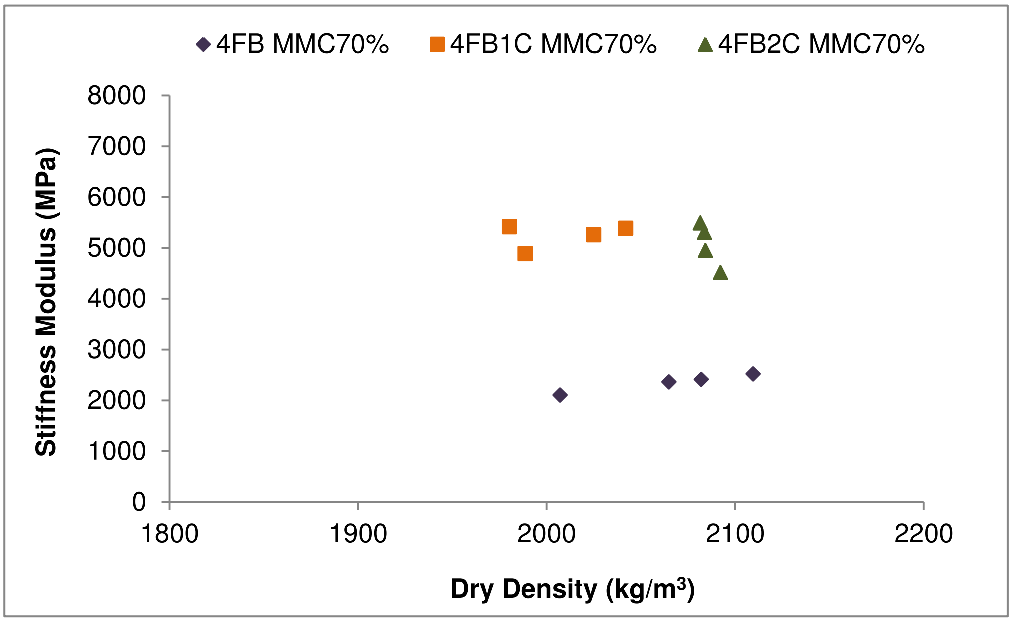


Figure 8.7 Stiffness modulus against dry density of foam mixes with increasing cement contents at MMC 70%

As more moisture was added during mixing (MMC 80%) a clear range of stiffness values could be distinguished between the foam mixes as shown in Figure 8.8. A greater lubricating effect was expected to produce foam mixes that had stiffness values as high as the foam mixes produced with MMC 70%. This could be observed for the 4FB mix which remains in the same range (about 2000MPa) regardless of the MMC levels (see Figure 8.6 and 8.7).

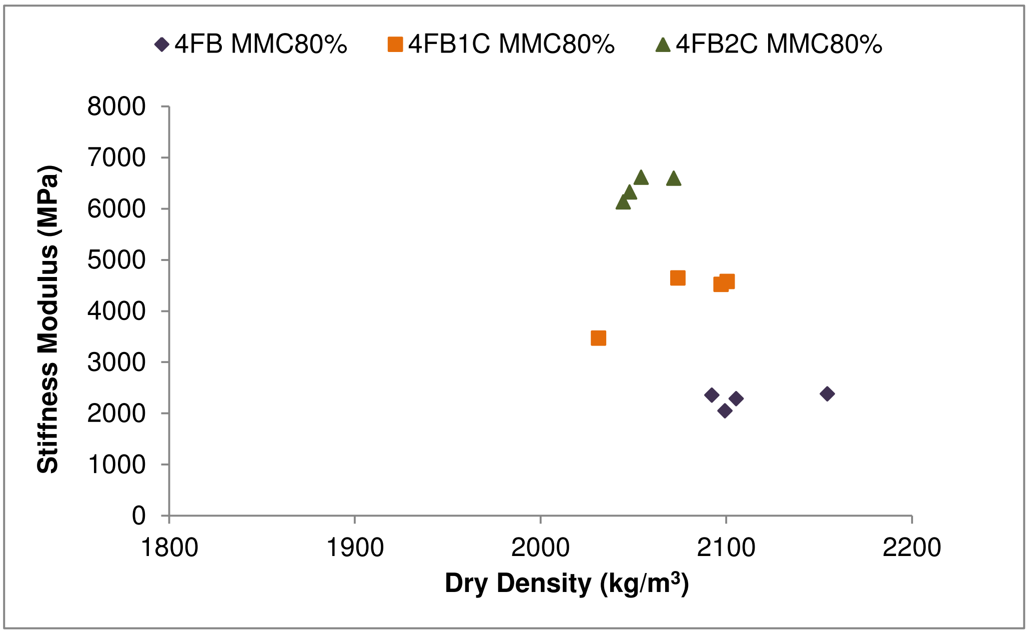


Figure 8.8 Stiffness modulus against dry density of foam mixes with increasing cement contents at MMC 80%

As cement was added, the stiffness modulus generally increased. The 4FB1C reached about 4500MPa and the 4FB2C mix reached its highest stiffness modulus values (about 6500MPa). The 4FB2C became stiffer than the 4FB1C could be, probably due to the presence of extra water that generated further hydration process in the presence of extra cement, adding more cemetitious bonds to the mix.

8.5 SUMMARY

The laboratory results confirmed that moisture content is a vital factor affecting the production of consistent foam mix. Less moisture might have resulted in a lower lubricating effect, which could affect the dispersion of binder during mixing. Furthermore, the fast drying effect could generate inconsistency in the moisture amongst the foam mixes' replicates particularly in the presence of cement that, at the same time, would be removing water by chemical reaction. This could lead to variations in the stiffness modulus results.

Insufficient or too much effective compaction moisture would prevent the foam mix from reaching maximum densification hence, low stiffness modulus value would be expected. However, in the case of the dry cured foam mixes exhibiting high stiffness modulus value, this would be due to the presence of the matrix suction and aggregate interlock. However, this is influenced by the percentage of cement content in the foam mixes.

As moisture was introduced to the foam mixes, the foam mixes became vulnerable to disintegration, particularly the non-cement treated foam mix, 4FB, at any MMC levels but the presence of cement in other foam mixes caused the stiffness to only experience a slight reduction from their dry curing condition with both MMCs of 60% and 80%. The addition of cement in the foam mixes assessed in this study significantly improved the mix's stiffness behaviour as regards both curing and moisture condition. The cement treated foam mixes show the resistance to the wet environment.

MMC need to be clearly designed to optimise the stiffness property of the foam mix. The study suggests that a MMC, equal to 70% of OMC, is adequate to produce the most reliable foam mix particularly with the addition of a small amount of cement, 1%. Therefore, MMC 70% has been used to produce foam mixes for other laboratory investigations.

CHAPTER 9 PRELIMINARY INVESTIGATION OF COIR FIBRE REINFORCED FOAM MIX

9.1 INTRODUCTION

As it has been mentioned in Chapter 4, bituminous foam mixes are partially bound materials exposing some unbound aggregate particles. Because of this reason, coir fibres, as reinforcement agents, on the basis of the literature surveys, might provide a means of overcoming the weakness of bituminous foam mixes. Therefore, it was decided to investigate the use of coir fibres as reinforcement agents in the foam mix and furthermore, they have never previously been used in cold bituminous mixtures.

This chapter presents the results from laboratory investigations on the effect of coir fibres on the fundamental properties of foam mixes such as stiffness modulus, tensile strength and permanent deformation.

9.2 EXPERIMENT DESIGN

Parameters considered in the investigation were percentage and lengths of coir fibres. The fibres were manually trimmed into three different length groups, with an average length of 30mm, 60mm and 90mm respectively. The experimental design is shown in Table 9.1.

It should be noted that this investigation was conducted alongside the trial foam mixes manufactured using the new foamed bitumen plant, WLB 10s. The reason was to avoid wastage of materials during familiarisation with the operational system of the new plant. In this case, the foam mix design in this investigation was prone to mistakes as it was a trial. The main goal was to produce a foam mix (with no coir fibres) that had the closest stiffness property to the old foam mix (manufactured from the WLB10 plant) which was about 2400MPa.

The first part of the laboratory experiment consisted of two selected length groups, 30mm and 60mm, to evaluate the stiffness modulus and tensile strength properties by conducting ITSM and ITS tests. The effect of fibre content was evaluated by testing specimens in which the dosage rate of the fibres was varied, but all other significant variables were controlled. Each group was prepared by adding coir fibres 0g, 0.25g, 0.5g and 1g per 1000g of loose foam mixes. The 30mm length was selected in comparison to the potential 60mm that could, hypothetically, entangle or reinforce at least two stones of maximum size 20mm that were utilised in the foam mixes. The 90mm length was not included in these tests as it was hard to control when mixing and distributing them in foam mixes.

Due to limited resources, the same test specimens as used in the ITSM tests were used for the ITS test. The specimens were left conditioned at 30°C after having been tested under dry and wet ITSM condition. The limited availability of the ITS equipment resulted in the specimens being tested after an unspecific curing period. However, each group of specimens manufactured from the same mixing batch were maintained at the same conditions before testing. Each test specimen was weighed to determine the bulk density prior to testing.

The first two groups, 30mm and 60mm lengths, were mixed into separate batches at different foaming water content (FWC) and the bitumen temperature was set in a range of 160°C – 165°C. The reason for the different foaming water content was due to on error in setting the FWC to a very low value, < 1% (2 l/hr). The FWC for the second mixing batch was set to 2.5% (9 l/hr). Each batch would produce four types of foam mixes where each mixture type had four replicates compacted using the gyratory equipment.

The coir fibre reinforced foam mixes were further evaluated to investigate the resistance to the permanent deformation. The permanent deformation was conducted using RLAT on the three lengths group, 30mm, 60mm and 90mm, of foam mixes at one selected content (1g per 1000g of loose foam mixes) including the control mixes (trial foam mix) without fibres. They were mixed and produced at FWC of 2.5% where the bitumen temperature was set at 168°C. All test specimens were cured under dry conditions. However, they were tested in their moist condition to simulate the worst field condition. Therefore, they were soaked in water for 24 hours at a temperature of 30°C prior to RLAT testing after they were dry cured. During testing, they were wrapped in a plastic membrane to maintain the moisture in the mixes.

Since the effect of coir fibres on foam only mix was found to be poor in this preliminary investigation therefore, it was necessary to carry out a small investigation on the selected cement treated foam mixes. This was the last part of the investigation of coir fibre as a reinforcement agent. The test specimens were manufactured with foamed bitumen produced at FWC 2.5% at 168°C. This foam mix design was also found to obtain the closest stiffness modulus property to the old foam mix design. A summary of the coir fibre reinforced foam mixes is presented in Table 9.1 with the respective mixture codes.

Table 9.1 Mixture codes for the trial coir fibres reinforced foam mixes

Mixture Codes	Fibre Length (mm)	Fibre Dosage (g per 1000g of foam mixes)	Bitumen Temperature Setting	Foaming Water Contents (FWC, %)
4FB0CF30	30	0	165°C	<1
4FB0.25CF30	30	0.25	165°C	<1
4FB0.5CF30	30	0.5	165°C	<1
4FBCF30	30	1	165°C	<1
4FB0CF60	60	0	160°C	2.5
4FB0.25CF60	60	0.25	160°C	2.5
4FB0.5CF60	60	0.5	160°C	2.5
4FBCF60	60	1	160°C	2.5
4FBCF90	90	1	160°C	2.5

9.3 SAMPLE PREPARATION

A large mass of each loose foam mix was riffled to distribute approximately 1000g for each compactor's mould. Each distributed mass of loose foam mixes was weighed prior to compaction. For the foam-fibres mixes, the distributed foam mix was first placed on a tray and the required mass of coir fibres was calculated accordingly based on the recorded mass of loose foam mix to go into each mould. The mass of coir fibres were weighed according to the required dosage rate. The fibres had to be continuously separated by hand whilst spreading and distributing them across the mixes. Care had to be taken to ensure the fibres did not clump forming a ball of fibres that would float on the mixes. It was found that adding fibres with a large amount of materials was difficult to control both during mixing and also riffling. Therefore, they were mixed with a small mass of loose foam mixes to gain control of their distributions. Figure 9.1 shows a comparison of the loose foam mixes with coir fibres at different percentage contents.

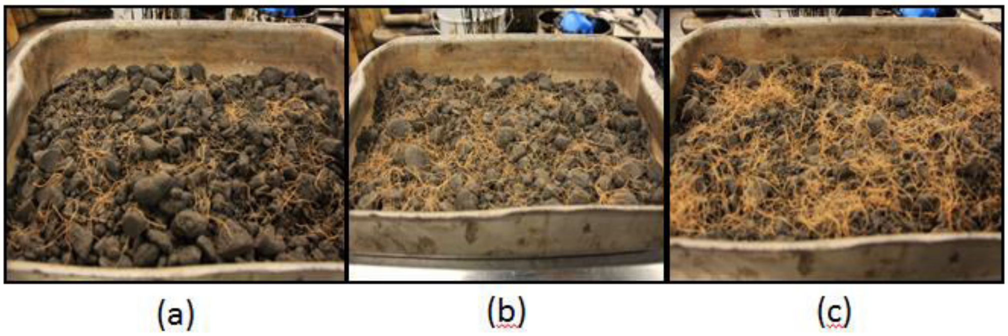


Figure 9.1 (a) 0.25g/1000g foam mixes (b) 0.5g/1000g foam mixes (c) 1g/1000g foam mixes

These foam-fibre mixes were transferred to a compactor's mould in small increments to minimise the clumping or balling of fibres and then compacted using the gyratory compactor. It was noticed that the compacted layers of the specimen did not bond easily with each other as compared to the foam mix without fibres, experiencing low compaction as shown in Figure 9.2. The more fibres were added into the mix, the more noticeable was the separation of the layers (marked by white arrows).

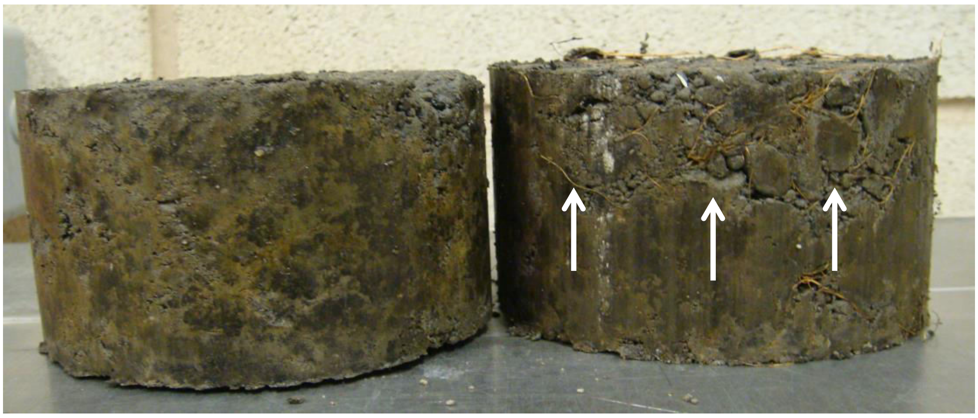


Figure 9.2 Compacted foam mix with and without coir fibres

9.4 STIFFNESS MODULUS OF COIR FIRBES REINFORCED FOAM MIXES

9.4.1 Moisture Movement

The mass of foam-fibre mix specimens was weighed and recorded to monitor the moisture movement as soon as they were extracted from the moulds and at certain curing times. Figure 9.3 represents the mass of moisture lost in the specimen after dry curing. It shows that as the amount of coir fibre increases, the loss of moisture from the foam mixes gradually increases. The trends are the same when short fibres are added. It suggests that the moisture lost is effectively influenced by the quantities of the fibres in the mixes, perhaps because of increasing air porosity.

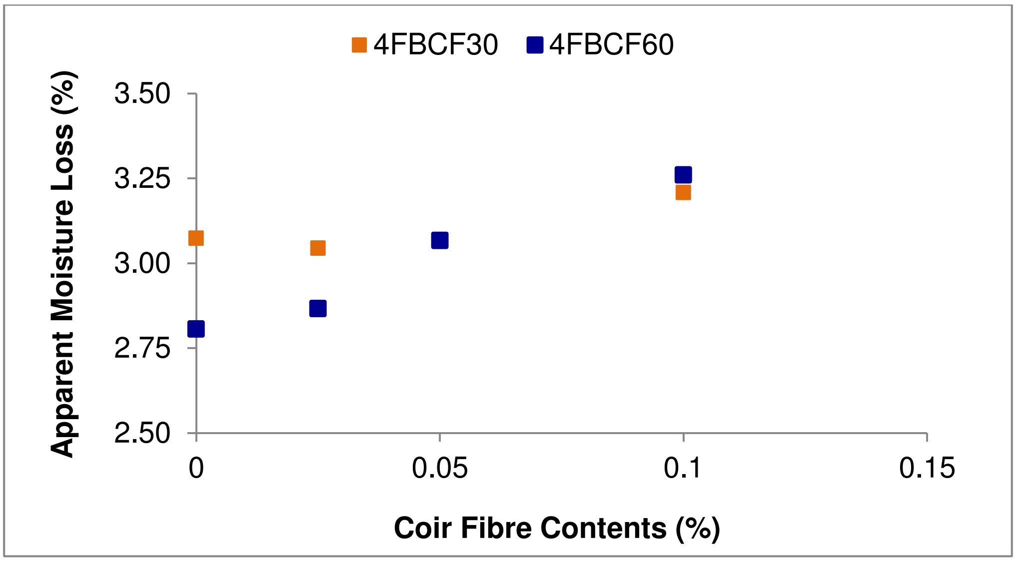


Figure 9.3 Apparent moisture loss in dry cured foam mixes with increase in Coir Fibre contents

9.4.2 Effect of Coir Fibre Contents

All test specimens were tested under dry conditions to determine the effect on stiffness modulus properties. These results are plotted in a graph shown in Figure 9.4. Although the curve trends give some indication of an increase in stiffness modulus as the foam mixes have increased coir fibres content the improvement is neither large nor certain for either type of foam mixes (i.e whether with 30mm or 60mm coir fibres lengths). The results of the wet stiffness modulus of all test specimens are poor as plotted in the same graph. The drop in stiffness with 60mm long fibres particularly evident.

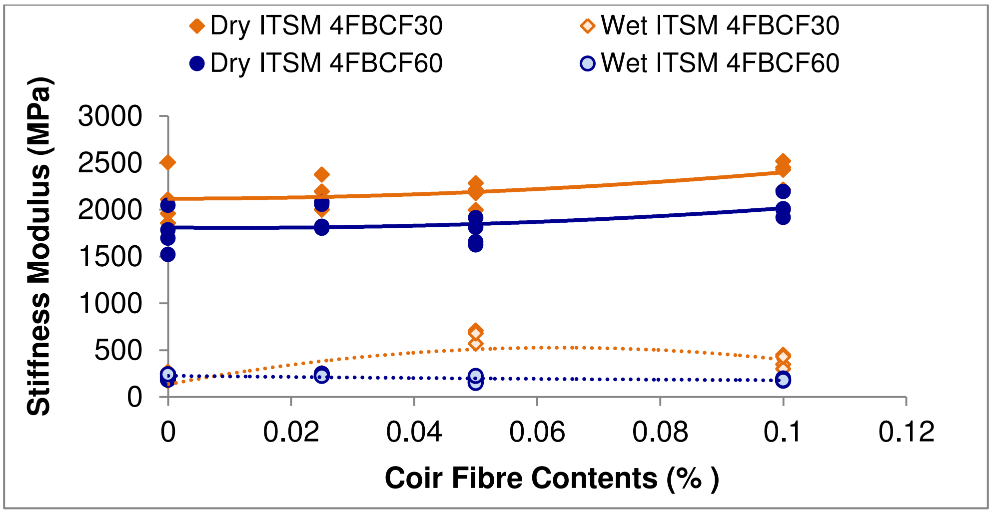


Figure 9.4 Stiffness modulus of foam Mixes with increasing coir fibre contents

Figure 9.5 presents the increases in the water infiltrated into the foam mixes when the coir fibres were added into the foam mixes. It suggests that the coir fibres absorb water resulting in more damage to the specimen and hence, lower stiffness modulus. It might be also due to the high voids generated in the mixes when more coir fibres are added (see Figure 9.2).

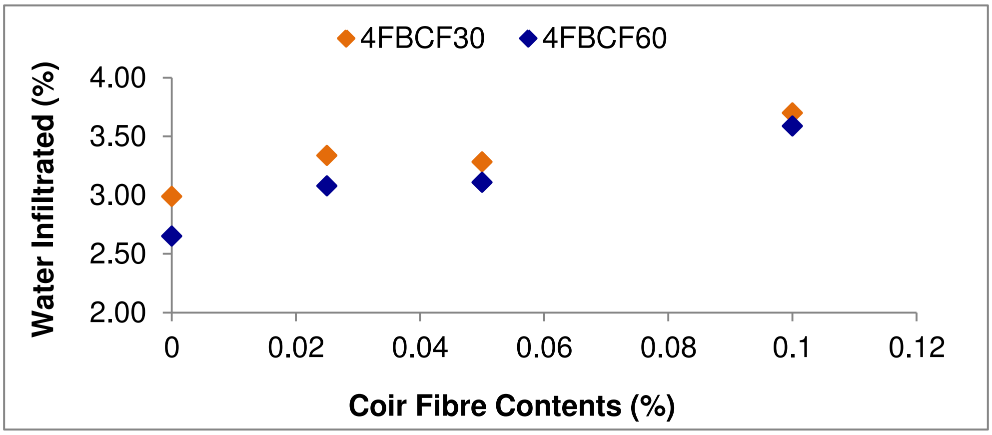


Figure 9.5 Mass of water gained in wet cured foam mixes with increasing coir fibre contents

9.5 INDIRECT TENSILE STRENGTH OF COIR FIBRE REINFORCED FOAM MIXES

The strength of foam mixes with increase in coir fibre content can be shown in Figure 9.6. The addition of 60mm coir fibres to foam mixes did not give tangible benefits compared with the addition of 30mm fibres and neither addition produced greater strength when compared to the foam mixes without fibres, 4FB.

On the other hand, the load to displacement curve for the foam mixes with 30mm coir fibre specimens (Figure 9.7) does vary significantly from the foam mixes with 60mm coir fibres (Figure 9.8). It can be seen that the foam mixes with the highest fibre contents took most displacement to reach peak load. Furthermore, these specimens did not split as the coir fibres held them together whilst the rest of the mixes failed by splitting. In contrast when adding short coir fibres, they all failed at almost same displacement, split into half, and there was no strain benefit of additional fibre content.

The ITS test results have shown that when the coir fibres are added to the foam mixes, regardless of lengths, the strength improvement was zero. The fibres occupied spaces between the aggregates causing air gaps and less aggregate-aggregate interactions. Nonetheless, the coir fibres may help the foam mixes to delay and/or stop the development of large cracks, but this was only evident when 60mm fibres were used in the foam mixes and not when the short fibres were used. The long fibre lengths provide connections amongst the coarser aggregates whereas, the short fibres, presumably, function as filler materials in the foam mixes rather than reinforcement. When the specimens reached failure, adding long fibres and increasing the proportion to 0.1% of the foam mix prevented the specimen from breaking apart.

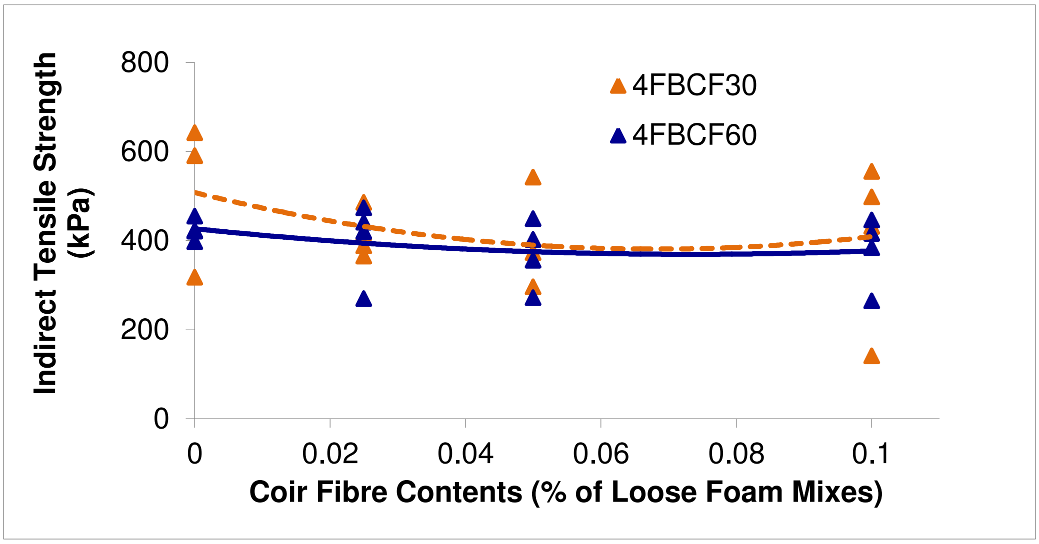


Figure 9.6 Indirect Tensile Strength of foam mixes with increase in coir fibre contents

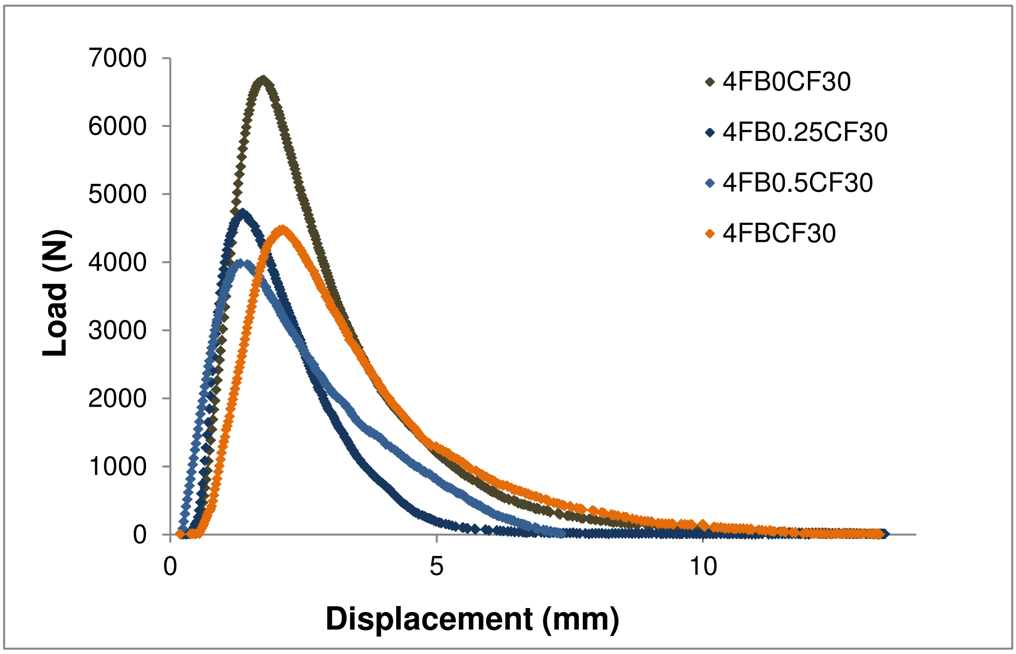


Figure 9.7 Load against displacement for the 30mm length coir fibre reinforced foam mixes with increasing coir fibre contents

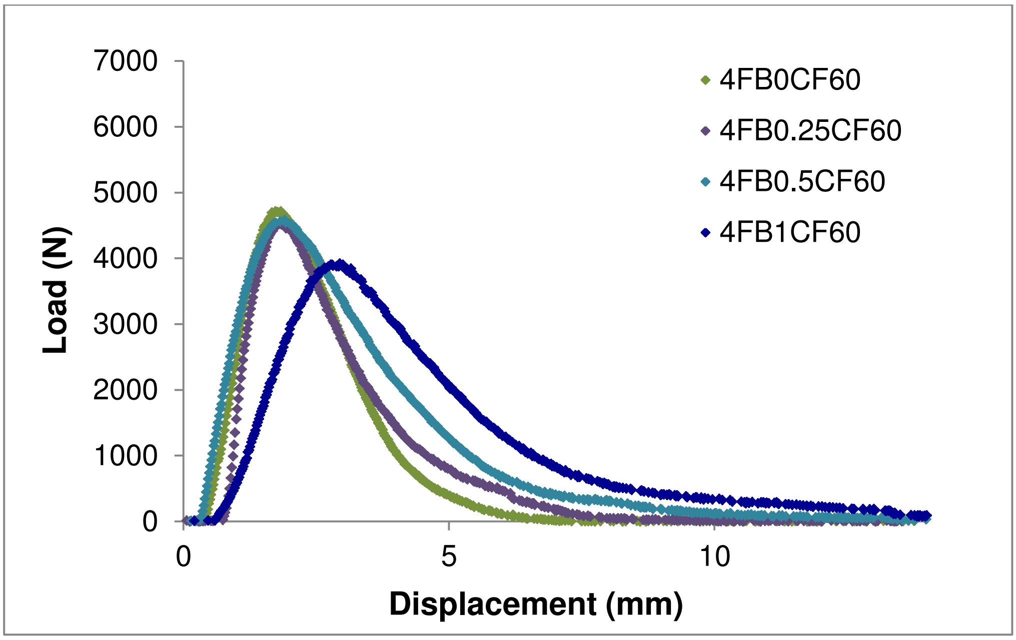


Figure 9.8 Load against displacement for the 60mm length coir fibre reinforced foam mixes with increasing coir fibre contents

9.6 PERMANENT DEFORMATION RESULTS

The results of RLAT tests are plotted in the graph shown in Figure 9.9 to determine the resistance to the permanent deformation. This shows that the strain increases as the foam mixes used longer coir fibre lengths. It indicates that the longer the coir fibres added in the foam mix, the poorer the resistance to permanent deformation.

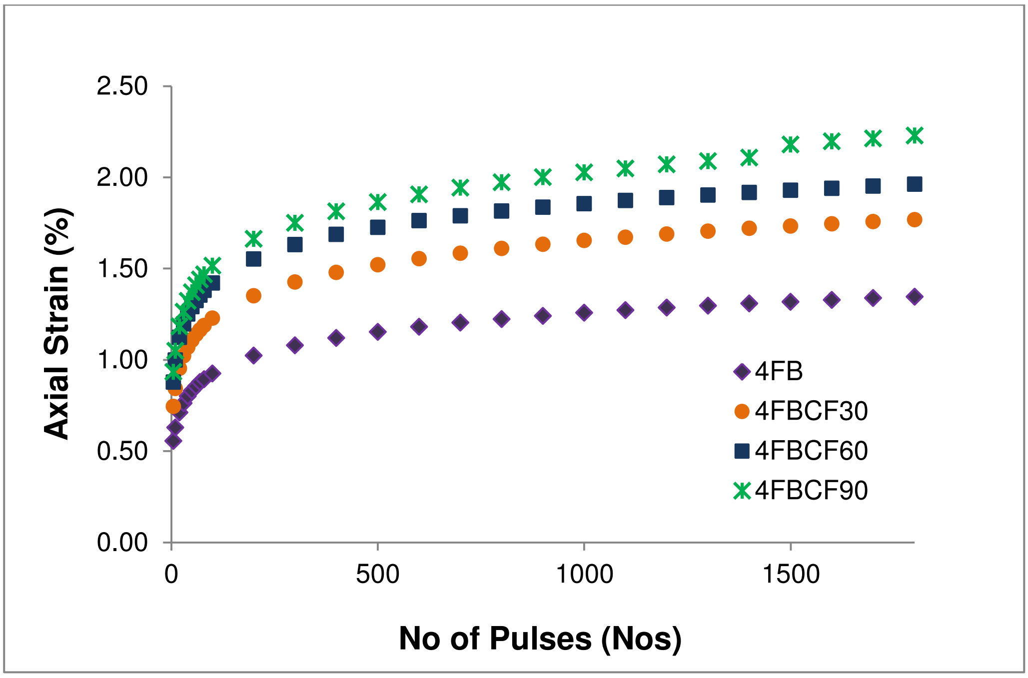


Figure 9.9 Permanent deformation for foam mixes with increasing coir fibre lengths

9.7 EFFECT OF COIR FIBRE ON CEMENT-TREATED FOAM MIXES

As can be seen in Figure 9.10, the inclusion of coir fibres in the cement-treated foam mixes did not improve the stiffness of the mixes particularly in wet conditions. Therefore in this study, the results indicated that the foam mixes should only be improved with the inclusion of cement, not coir fibres. It is possible that strength or resistance to permanent deformation may have been improved by the addition of coir fibres to the cement-treated foam mixes, but those aspects were not investigated.

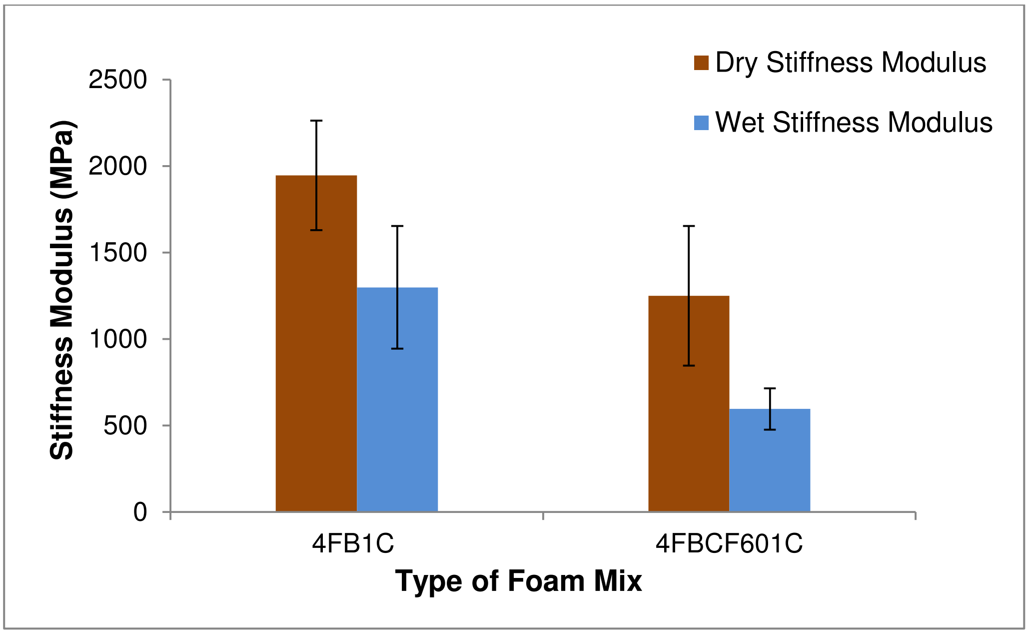


Figure 9.10 Stiffness modulus of cement-treated foam mix with and without coir fibres.

9.8 SUMMARY

The foam only mix experienced no tangible improvement in stiffness behaviour nor did the cement-treated foam mix, when the coir fibres were added.

Although greater loss of moisture could be observed in foam mixes when coir fibre content increased to 0.1% (of the foam mix), a consequential increased in void content occured that did not help the material to resist damage in a wet environment. The more coir fibres were added into the foam mixes, the more water would be infiltrated and absorbed during a soaking condition.

The addition of fibre does not significantly affect the corresponding peak load at which the foam mixes reach their failure state. However, it delayed failure significantly as the displacement to peak load was greater than the unreinforced foam mixes. This is most evident for 60mm coir fibres at their highest contents, 0.1%.

Only the reinforced foam mixes with high, 60mm, coir fibre contents did not rupture completely as the fibres held the specimen, preventing it from splitting in half. All other specimens of foam mixes with short fibres were broken in half. Therefore, the foam mix reinforced with longer coir fibres could potentially minimise the load induced cracks from propagating further, post failure.

However, poor resistance to permanent deformation in the wet condition was indicated as longer coir fibres were added in the foam mix.

CHAPTER 10 LABORATORY HUMID CURING STUDY

10.1 INTRODUCTION

This part of the laboratory investigation was set out predominantly to assess the stiffness development in a particularly humid condition, simulating field-curing. The aim was to estimate the short-term stiffness and establish equilibrium conditions of stiffness and moisture, where both reached a state of little or no change after a period of curing.

By estimating the time needed to develop the short-term stiffness, the period after which the road could be opened to traffic can be studied. The establishment of an equilibrium region for both stiffness and moisture could assist in understanding the impact of humid curing and its practical implications.

For this reason, specimens were cured in a laboratory designed humid tank in an unsealed state for the duration of the analysis period. The specimens were not allowed to be in direct contact with moisture or water so as to investigate only the effect of humid curing on the foam mixes. Furthermore, the effect of water in foam mixes has already been covered separately in other chapters such as Chapter 6.

10.2 EXPERIMENT DESIGN

In this study, only two foam mixes were tested, 4FB and 4FB1C. Both mixes were manufactured using the foamed bitumen rig, WLB 10, and the Hobart mixer. However they were actually produced on different occasions due to the tight laboratory schedules and some technical problems with the plant encountered that necessitated a break in mixing works. The test schedule for both mixes was initially programmed so that the two foam mixtures would be prepared under exactly the same conditions.

All test specimens were mixed and compacted at MMC equal to 70% of OMC and this value was taken as the initial moisture for all specimens. Due to the breakage of the WLB10, no other foam mixes were included in this study. If other mixes had been produced using the new foamed bitumen rig, WLB 10s, they would not have been directly comparable. Furthermore, from the ITSM results (i.e under Dry, Wet and Dry/Wet cycles), 4FB1C has shown its potential as an optimum performance material. It maintained the benefits of bituminous characteristics as compared to the more rigid mix, 4FB2C, and was better performing than in combination with other additives such as coir fibres in Chapter 9 and hydrated lime and Wet Fix in the following Chapter 11, particularly when in a wet condition.

10.3 DEVELOPMENT OF STIFFNESS MODULUS OVER HUMID CURING PERIOD

This section presents the development of stiffness modulus during a humid curing period. The results are presented in two sections: a short and long term humid curing condition. The short term would predict the effects of humid curing on the development of stiffness modulus in the early days whereas the long term period would assist the study of the development of stiffness modulus and moisture in foam mixes until they reach an equilibrium state.

10.2.1 Short Term Humid Curing Period

Figure 10.1 shows the development of stiffness modulus of foam mixes over a period of six weeks. The 4FB1C mix gained strength to about 3000MPa on the 4th day, which is about three times the stiffness of the 4FB mix, 1000MPa, which was only gained after a week. An attempt was made to test 4FB on the 4th day, with only one specimen being tested and this gave a result of about 400MPa. Because of the weak result, the ITSM test on other replicates was discontinued. It was observed that the specimens were still in a visibly moist condition; hence, it was decided to keep the test specimens for more days so that there was less risk of being severely damage by handling.

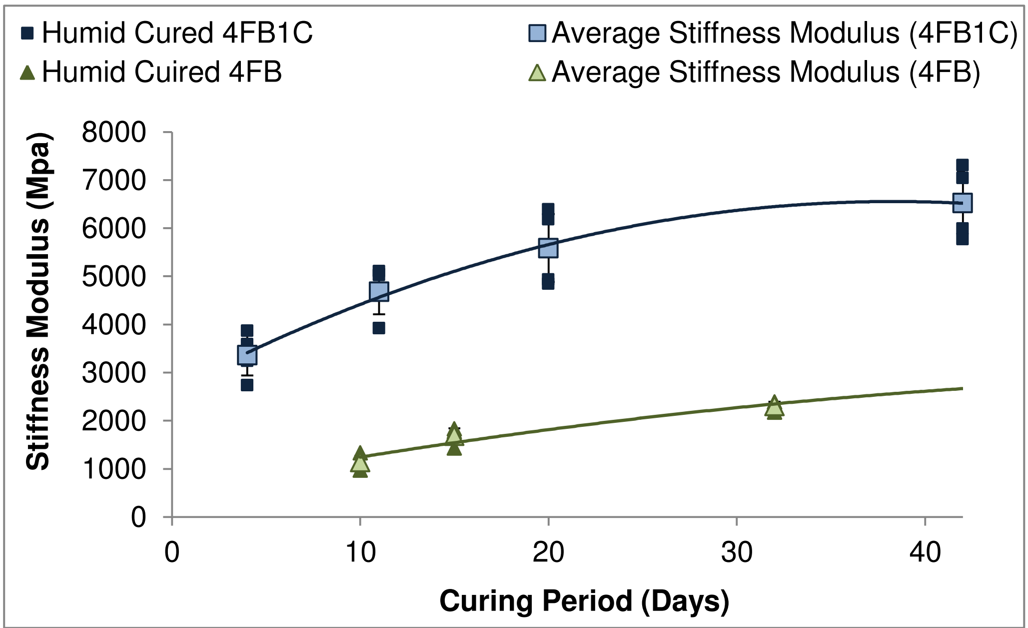


Figure 10.1 Stiffness Modulus of Foam Mixes over a month of Curing Period

After six weeks, the stiffness modulus of the 4FB mix only reached about 2500MPa. In contrast, the 4FB1C mix achieved an average stiffness of about 6500MPa, about twice its first ITSM test value on the

4th day of humid curing. The high stiffness modulus value of the 4FB1C mix demonstrates its potential application in a tropical humid climate.

10.2.2 Long Term Humid Curing

Figure 10.2 presents a graph showing the development of stiffness modulus of foam mixes, 4FB and 4FB1C, over the long term humid curing period. The early stiffness value of the 4FB mix resulted in about 40 to 50% of the stiffness value of the 4FB1C mix at the same age. The stiffness of 4FB increased over time until, after six months, the curve was almost flat with a maximum average stiffness modulus of 3500MPa. In contrast, the stiffness modulus value of the 4FB1C mix could reach a maximum average of almost 8000MPa. However, the stiffness modulus eventually decreased, perhaps caused by the presence of micro-cracks or by cement paste carbonation.

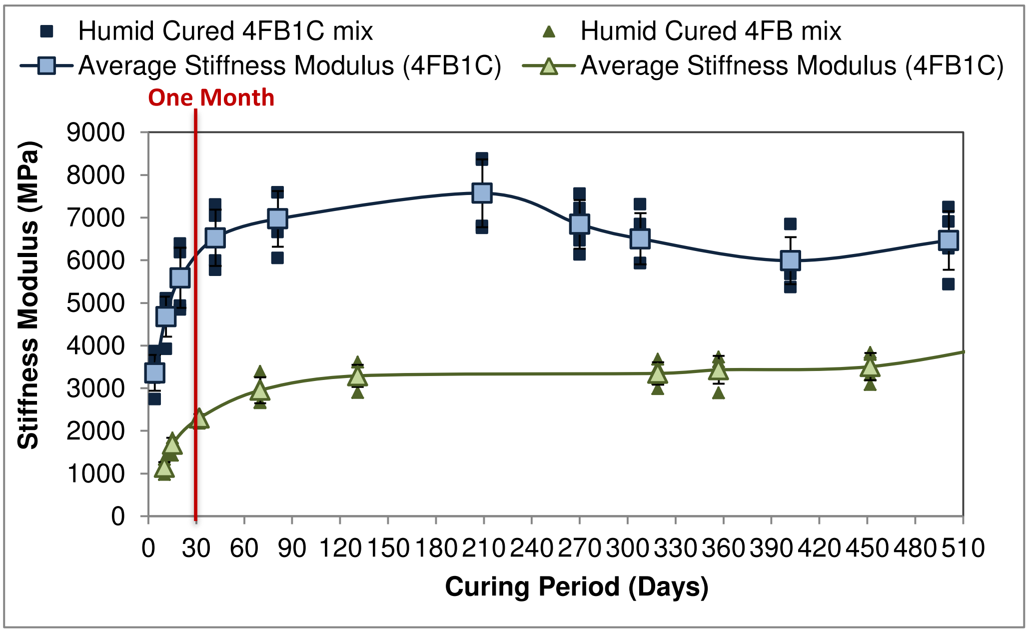


Figure 10.2 Development of Stiffness Modulus over a Long Humid Curing Period

Due to the limited material resources, the same test specimens were used for the ITSM tests throughout the investigation. Repeated ITSM tests might have changed the aggregate orientations in the mixtures relative to the load platens, but this would lead to apparently random variation from test to test that is not evident in Figure 10.2. Cumulative damage to brittle cement bonds seems a more likely mechanism for mix, 4FB1C, especially as no similar reduction is seen in the 4FB mix.

10.3 DEVELOPMENT OF STIFFNESS MODULUS WITH REDUCTION OF OMC

Figure 10.3 is plotted to assess the development of stiffness modulus with a reduction of moisture content. The moisture content was the difference between the initial moisture in the foam mix and the moisture content during testing. The initial moisture content was the MMC (equal to 70% of OMC). The stiffness moduli of the dry cured foam mixes were also plotted against their reduced OMC for comparison with the humid cured foam mixes. The dry cured foam mixes were initially sealed (kept in the mould) and placed in an oven at 30°C for 1 day to gain cohesion followed by unsealed dry curing at 40°C for 3 days. These dry cured foam mixes experienced a reduction in moisture content, to about 25% of OMC after 4 days in the case of 4FB1C.

The dry cured 4FB mix gained a value of stiffness modulus between 2000MPa and 3000MPa, higher than the humid cured 4FB, which had a value of approximately 1100MPa but only after 10 days. This is due to the rapid reduction of moisture content in the dry cured 4FB mix, to 5% of OMC, compared to the humid cured 4FB mix with a reduction of moisture to about 12% of OMC. In order to reach the same stiffness modulus as the dry cured 4FB mix, the humid cured 4FB mix had to continue reducing its moisture to 0%. This shows that the humid environment slows the curing process of the foam mixes. It can be observed that the dry cured 4FB lost moisture more rapidly than the dry cured 4FB1C, to 5% of OMC.

In contrast, the dry cured 4FB1C mix reached a stiffness modulus of about 5000MPa as compared to the humid cured 4FB1C that had then only reached between 3000MPa and 4000MPa, both foam mixes being tested after their 4th day of curing. The moisture in the dry cured 4FB1C mix reduced to about 25% of OMC within 4 days of curing, 7% less than the humid cured 4FB1C mix, about 32% of OMC. The dry cured, 4-day value of 5000MPa was achieved by the humid cured 4FB1C when its moisture had reduced to about 13% of OMC.

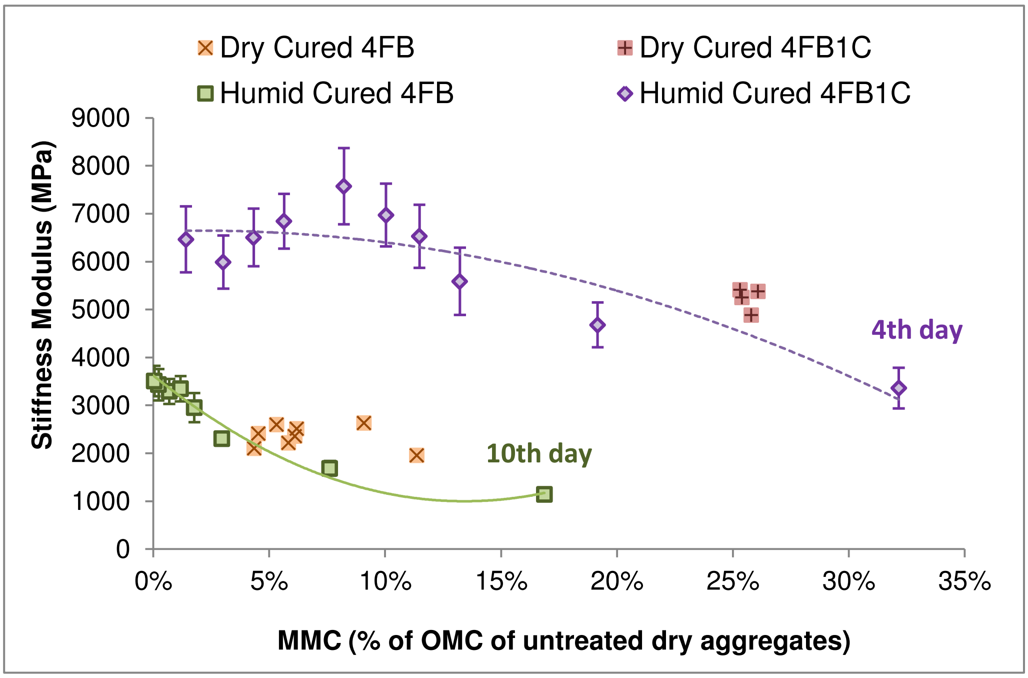


Figure 10.3 Stiffness modulus of foam mixes with reduction in moisture content

Generally, the development of the stiffness modulus for both humid cured foam mixes can be seen as the moisture content reduces.

However when the moisture contents reduced to 0% of OMC, the development of the humid cured 4FB mix slowed down and it began experiencing disintegration of fines. The disintegration was partly responsible for the almost zero moisture values calculated, because the loss in mass of the test specimens probably included a small amount of fine particles. The ultimate stiffness modulus reached by the humid cured 4FB mix was between 3000MPa and 4000MPa. This same value was reached by the humid cured 4FB1C after its 4^{th} day of curing. In contrast, the stiffness modulus of humid cured 4FB1C mix reached its peak, about 7500MPa, and then began to decline.

It can be deduced that the unsealed 3 days dry oven curing at 40°C after 1 day kept in the mould at 30°C did not model a humid, long term curing environment.

10.4 MOISTURE MOVEMENTS IN HUMID CURED FOAM MIXES

Figure 10.4 shows a plot of the moisture in the foam mixes (assuming loss of mass indicates a loss of moisture) in terms of the percentage of OMC against curing period (in days). The tests show that the initial moisture content of 70% of OMC, which was also the MMC of the foam mixes during mixing and compaction, reduced to approximately 13% of of OMC after a week of humid curing at 30°C.

At early age, the observed trends marked by a red dotted circle show similarity for both 4FB and 4FB1C. The inclusion of cement did not seem to significantly affect the loss of mass.

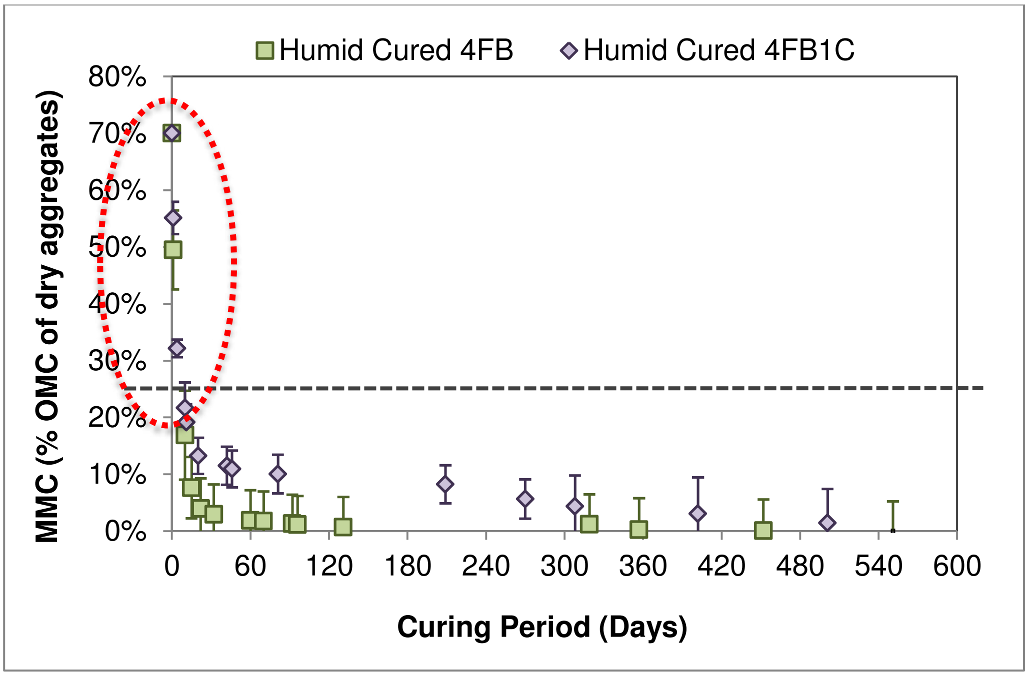


Figure 10.4 Reduction of OMC over the Curing Period

Therefore, the loss of water in the 4FB mix appears to be matched by an equal loss of moisture to the atmosphere plus an unquantified amount that has been reacted with the cement in the hydration process.

After one week, the 4FB mix lost moisture more than the 4FB1C mix which reduced to 22% of OMC. After a month, the humid cured 4FB mix continued to lose moisture until it reached an almost horizontal line at zero moisture content. In contrast, for the humid cured 4FB1C mix, the moisture content stabilised at just above 0%.

When the moisture contents reduced to about 20% of OMC (presented as dotted line), the 4FB mix continued to lose moisture at a relatively rapid rate and but clearly more than the 4FB1C mix until the moisture content reduced to about 0% of OMC, whereas the cemented mix lost less moisture. This is probably because the 4FB1C mix contains less water that can be lost given the mass removed into hydration products.

10.5 RELATIONSHIP BETWEEN STIFFNESS MODULUS AND MOISTURE CONTENTS

The graph in Figure 10.5 presents the experimental results showing the relationship of the stiffness modulus of 4FB and 4FB1C with the increasing loss in moisture under accelerated humid curing for about one year. The stiffness modulus generally increased with increasing moisture loss, as indicated by the previous graphs.

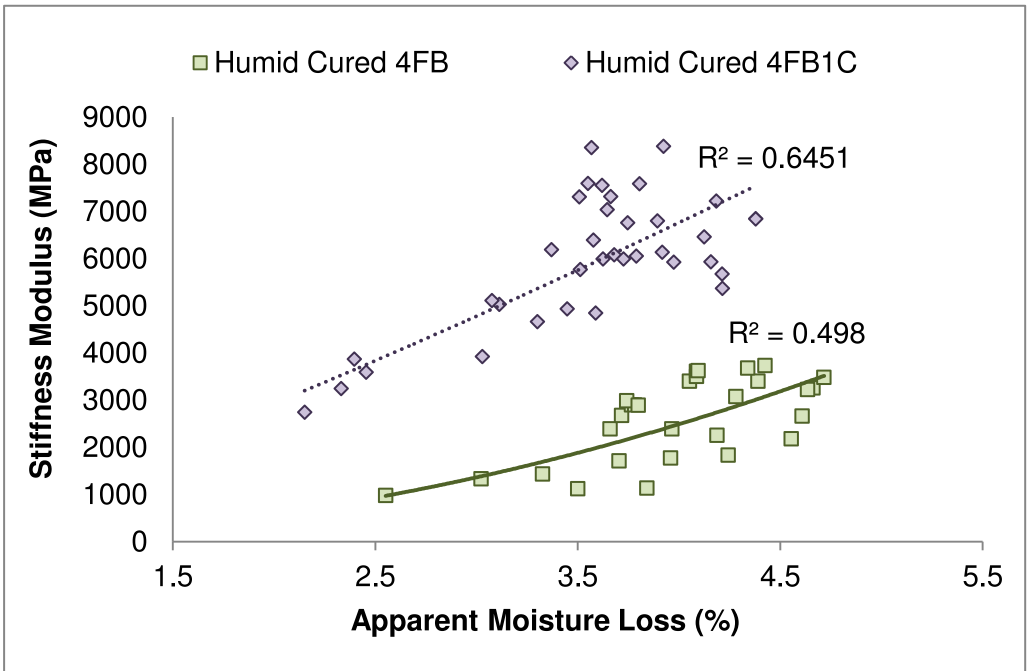


Figure 10.5 Relationship between the Stiffness Modulus of Foam Mixes and the Moisture Lost in Humid Curing Condition

As the moisture lost increased, the results became more scattered. For 4FB mix, as the moisture loss progressed to about 3.5%, the results became scattered reaching to a maximum of about 4.5% resulting in a stiffness modulus of approximately 4000MPa. With the inclusion of cement, the 4FB1C mix gained a higher stiffness than the 4FB mix at all comparable moisture loss points. As the moisture loss of the 4FB1C mix progressed to about 3%, the results started to show wider scatter reaching a maximum stiffness modulus of about 7000MPa at a moisture loss of about 4%.

10.6 SUMMARY

It can be concluded that the humid environment has greatly influenced the curing process of foam mixes and hence the development of stiffness modulus of the foam mixes. However, the findings have shown that a little amount of cement is required to assist the foam mixes in gaining early strength in order to reduce the risk of premature damage due to the presence of moisture. The damage would be accelerated more rapidly with the increase in traffic loadings once the road is in service.

The 4FB mix requires a longer curing period than the 4FB1C mix and the stiffness modulus remains significantly less at all ages. It only rose to about 1000MPa after a week. The low stiffness modulus of the 4FB mix indicates that it has a high risk of moisture damage particularly in the event of rainfall during the curing period.

The stiffness modulus of 4FB1C reached about 3000MPa by the 4th day of the humid curing period. This finding indicates that a short curing period of such a mix could allow for early trafficking.

Although the cement-treated foam mix reached a maximum stiffness modulus of 8000MPa, it eventually experienced reduction in the stiffness modulus after 8 months. But it was capable to maintain a stiffness modulus of approximately 6000MPa.

Generally, the stiffness modulus of bituminous foam mix increased as the moisture lost increased. However, the results became more scattered after the moisture lost exceeded 3%.

CHAPTER 11 DURABILITY OF BITUMINOUS FOAM MIXES

11.1 INTRODUCTION

Durability of pavement materials is an important factor to be considered in the evaluation of mechanical performance and is greatly influenced by the environmental condition, particularly the presence of moisture. The local climatic pattern in Brunei Darussalam can comprise of alternate cycles of relatively hot sunny and wet rainy days or intermittent rains in a hot sunny day. For this reason, it is important to study the durability of the foam mix by investigating the effect of alternating dry and wet cycles on its stiffness properties.

Hence, this chapter describes the experiment design in which, apart from cement and coir fibres, two other additives were included, namely Wet Fix (WF) and Hydrated Lime (HL). These two additives were actually used for a Master's degree project to evaluate the moisture susceptibility of the foam mixes. For comparison purposes, these bituminous foam mixes are also included in the analysis and discussions.

11.2 SAMPLE PREPARATION

Four replicates were prepared for each mixture in which the total mass of four replicated mixtures would be about 4kg. All solid additives, Cement, Coir Fibres, HL, were added separately at their selected amount into the required mass of loose foam mix; but not liquid additive, WF. The reason for not adding the solid additives before mixing with foamed bitumen was the limitation of the twin shaft mixer which could only work with a minimum mass of 12kg. A wastage of materials would be expected if 12kg mass of mixture was used to produce four replicate mixtures which only required a mass of about 1000g for each replicate. Time for preparing and riffling aggregates and the work needed to clean the mixer between the mix types also meant that one full batch was the sensible choice.

As mentioned in Chapter 5, a liquid anti-stripping agent, WF, was used as additive and is normally mixed with heated bitumen prior to adding aggregate. The mixing of the WF and bitumen was carried out prior to its use because it generates a generous amount of fumes. Therefore, it is not safe to add WF directly into the hot bitumen in the foam mix plant's tank.

All test specimens were initially cured under the normal dry condition in the oven at a temperature of 40°C for three days to ensure the specimens gained stiffness. They were subsequently tested for ITSM to determine the dry stiffness modulus. Then the wet conditioning followed for 24 hours in a water bath at a temperature of 30°C. The ITSM test was then repeated on these wet specimens. They were then dried in the oven for 24 hours at a temperature of 30°C. The dried specimens were tested and the wet-dry cycle was then repeated. All mixing works were timed such that all initial tests occurred on Monday morning and began the alternate wet-dry cycles until the last cycle and tests were completed on Friday in the same week.

The moisture movement for each of the foam mixes was monitored by recording and weighing the mass of each test specimen after it was cured under each curing cycle. The dimensions of all test specimens were also recorded for bulk density measurements.

11.3 EFFECT OF DRY WET CYCLES ON STIFFNESS MODULUS OF FOAM MIXES

Figure 11.1 illustrates the variation of stiffness modulus of bituminous foam mixes with different additive combinations, their changes from their initial dry curing state and after being subjected to wet and dry cycles. In general, all the foam mixes recovered from their wet condition, after dry curing, but became less stiff after multiple wet-dry cycles. Moreover, foam mixes without cement or hydrated lime were significantly damaged during the wet curing .

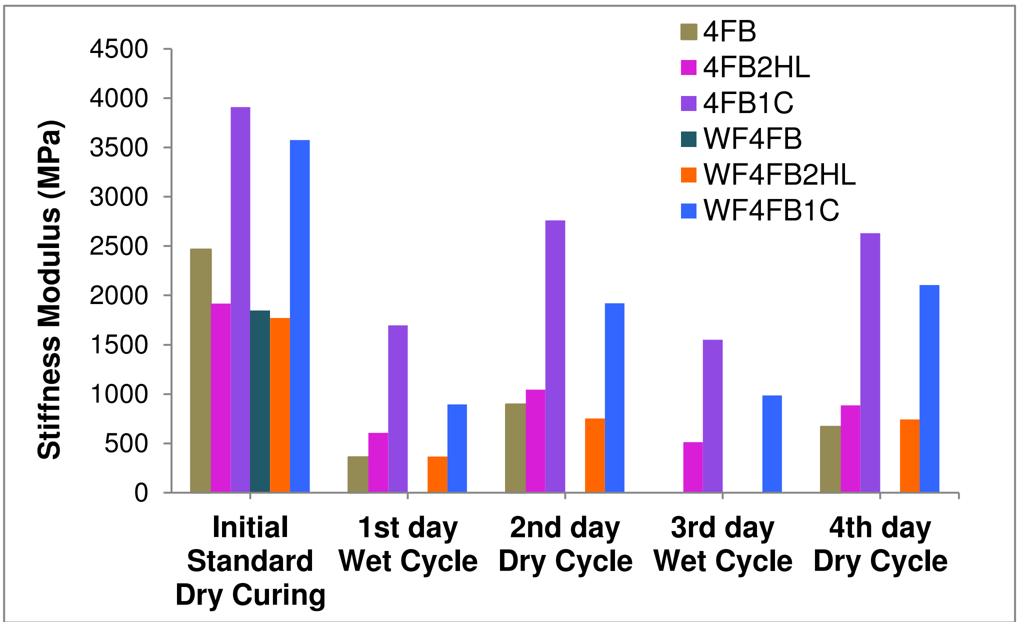


Figure 11.1 Wet-Dry cylces of initially dry cured foam mixes

The cement treated foam mixes, 4FB1C and WF4FB1C, clearly stand out amongst all foam mixes at their initial dried curing state and after the wet-dry cycles. However, the cement treated foam mixes produced using standard bitumen, 4FB1C mix, showed the highest initial dried stiffness modulus of 4000MPa and are able to retain about 1500MPa under wet curing as well as regaining stiffness modulus to about

2500MPa when they were dried at 30°C for 24hours. The WF4FB1C mix exhibits only a slightly lower stiffness modulus under its dried condition but unfortunately it did not perform as well as the 4FB1C mix under wet curing.

For the 4FB mix, the initial dry stiffness modulus was about 2500MPa, higher than those produced with the WF blended bitumen, WF4FB mix. It shows that the addition of WF into the base bitumen did not assist in improving the foam mixes without cement as was expected, it being an anti-stripping agent. The initial stiffness of 4FB mix reduced to 500MPa following wet curing. When allowed to dry, the stiffness regained to about 1000MPa. In contrast, the WF4FB mix seemed to lose almost all its stiffness as soon as water was introduced to the mixture. It did not recover its stiffness even after it was dried for 24hours. Therefore, both mixes were found to be susceptible to water but WF4FB mix was the worst. Physically, the WF4FB mix looked more brittle than the 4FB mix, resulting in more ravelling. It may be that the operation of WF with bitumen hinders foam adhesion to the wet aggregates during the initial mixing.

The HL treated foam mixes, 4FB2HL, did not seem to gain a stiffness modulus higher than 1000MPa. When the HL was mixed with aggregates, the pozzolanic reaction started to form cementitious compounds. However, the different mixing technique, adding the solid additives into loose foam mix, adopted in this study might be less effective as it left some HL unreacted. It was observed that, when the test specimens were made and compacted, white hydrated lime powder could still be seen within the mix (see Figure 11.2). This might reduce the direct surface interaction between the HL and aggregate particles. Therefore, less cement compounds will have been produced than in the cement treated foam mix, 4FB1C. This helps to explain the lower stiffness of the hydrated lime treated foam mix. The hydrated lime contributed less stiffness improvement as opposed to cement treated mix, which agreed with a previous study (Jitsangiam et al, 2012).

Figure 11.2 Example of hydrated lime treated foam mix

11.4 MOISTURE SENSITIVITY

The moisture sensitivity was evaluated by monitoring and measuring the mass of the test specimens after each curing cycle. The largest loss in the mass was due to the fast drying under the initial standard dry curing where they were placed in an oven for three days at a temperature of 40°C, higher than the later dry cycles which were at 30°C.

The graphs shown in Figure 11.3 to 11.8 illustrate the influence of moisture content in the different foam mixes on their stiffness modulus. In general, the findings demonstrate that an increase in moisture content resulted in a reduction of stiffness modulus values. The cement-treated foam mixes, particularly 4FB1C, had the highest moisture resistance and produced the most durable foam mixes. The WF4FB mix degraded the most amongst all the foam mixes.

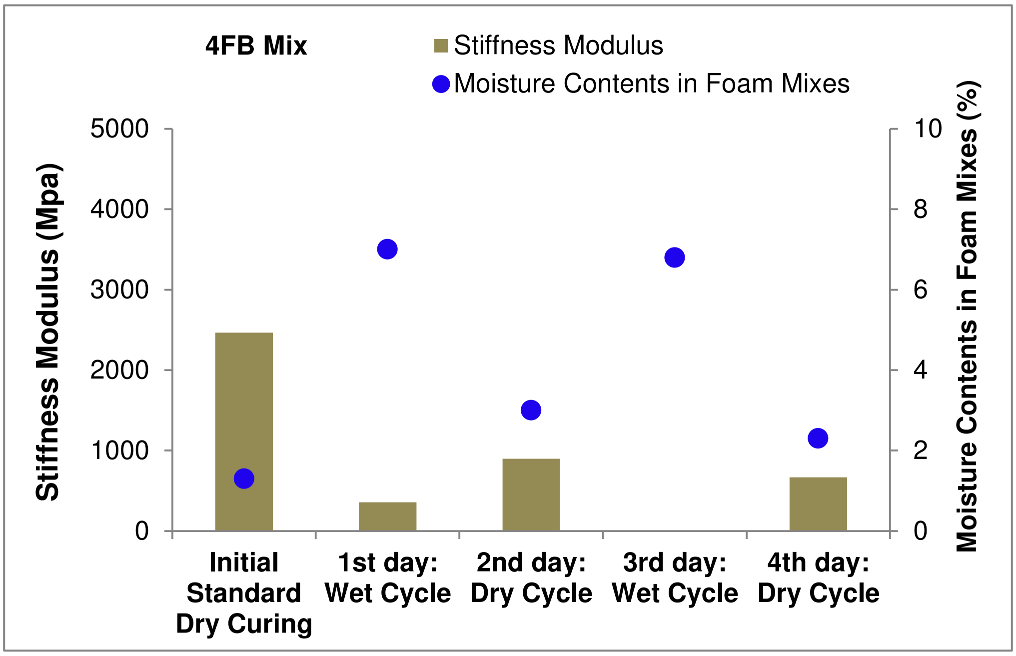


Figure 11.3 Effect of moisture contents in the 4FB mix on its stiffness modulus

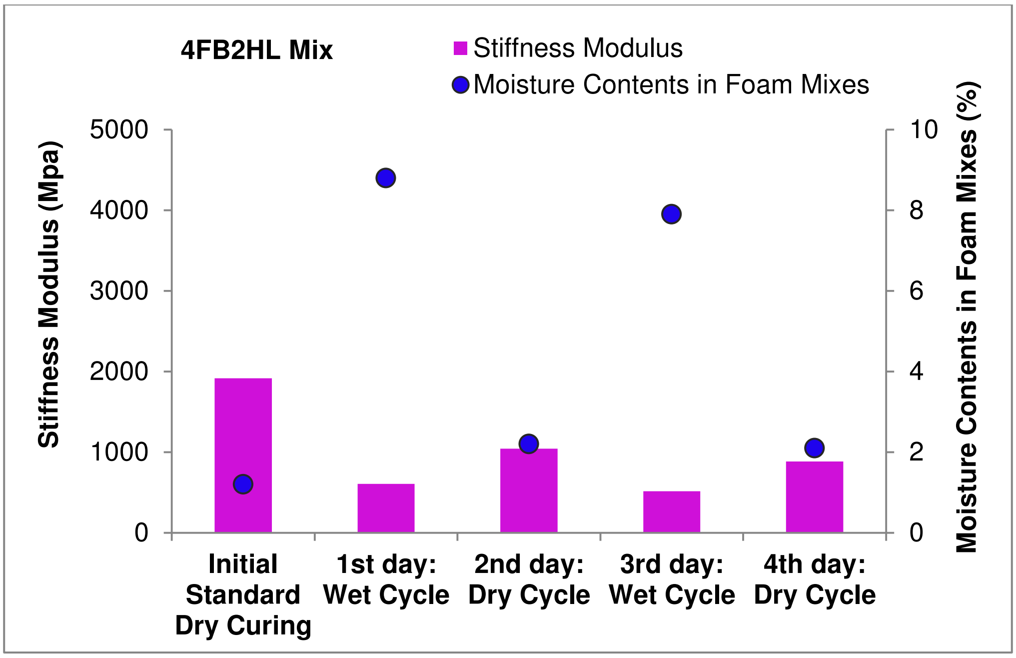


Figure 11.4 Effect of moisture contents in the 4FB2HL mix on its stiffness modulus

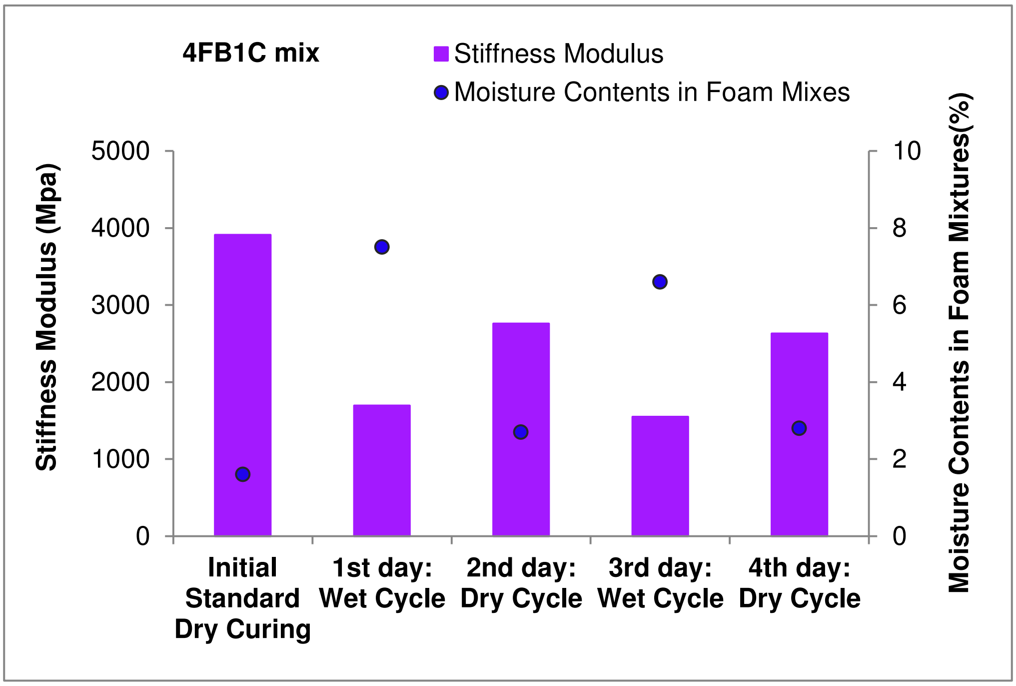


Figure 11.5 Effect of moisture contents in the 4FB1C mix on its stiffness modulus

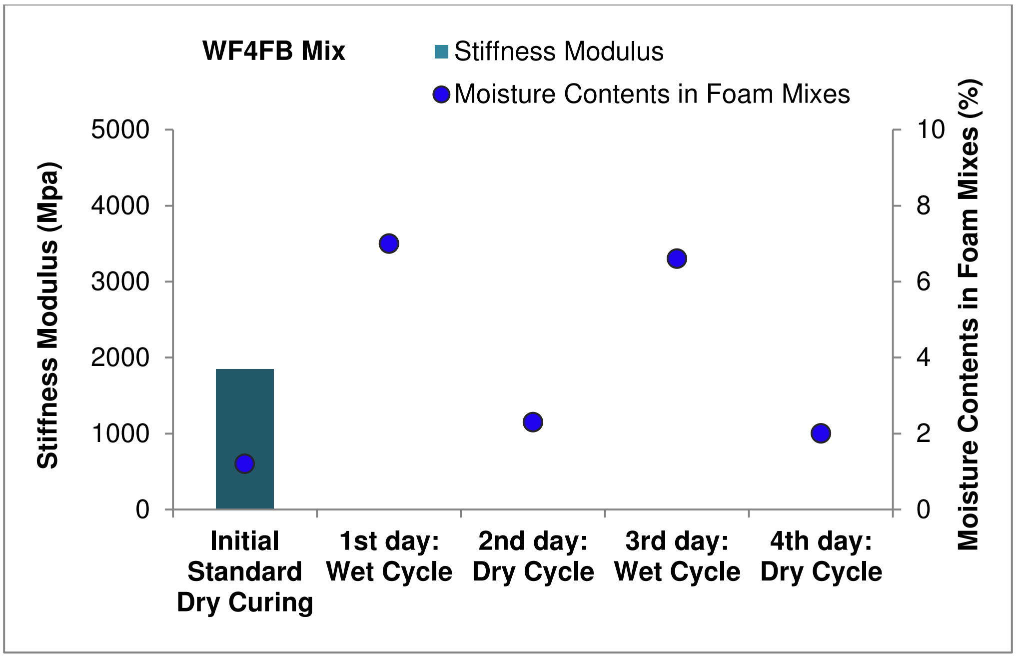


Figure 11.6 Effect of moisture contents in the WF4FB mix on its stiffness modulus

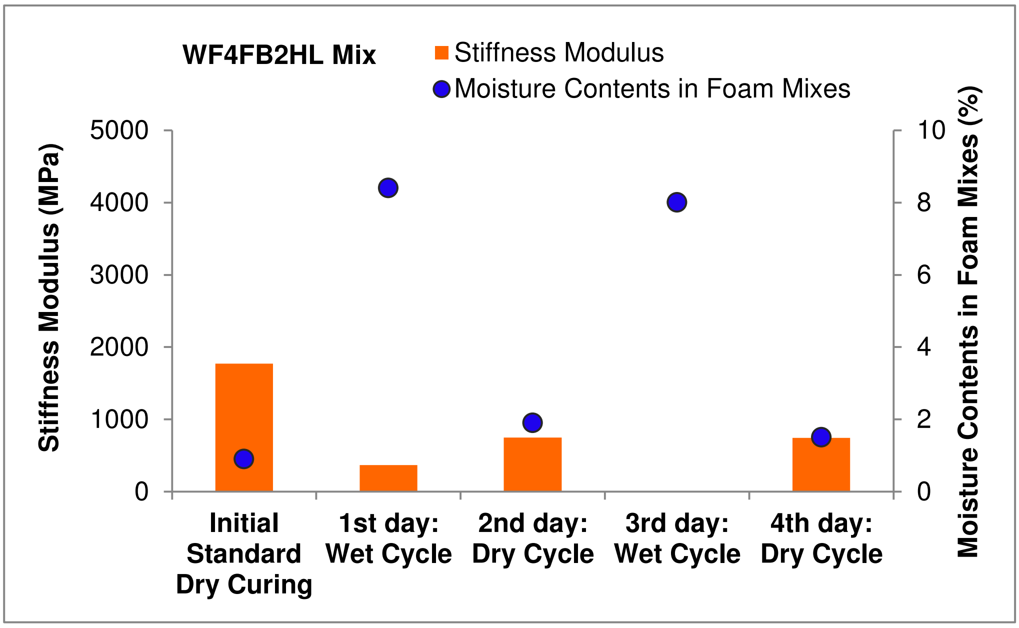


Figure 11.7 Effect of moisture contents in the WF4FB2HL mix on its stiffness modulus

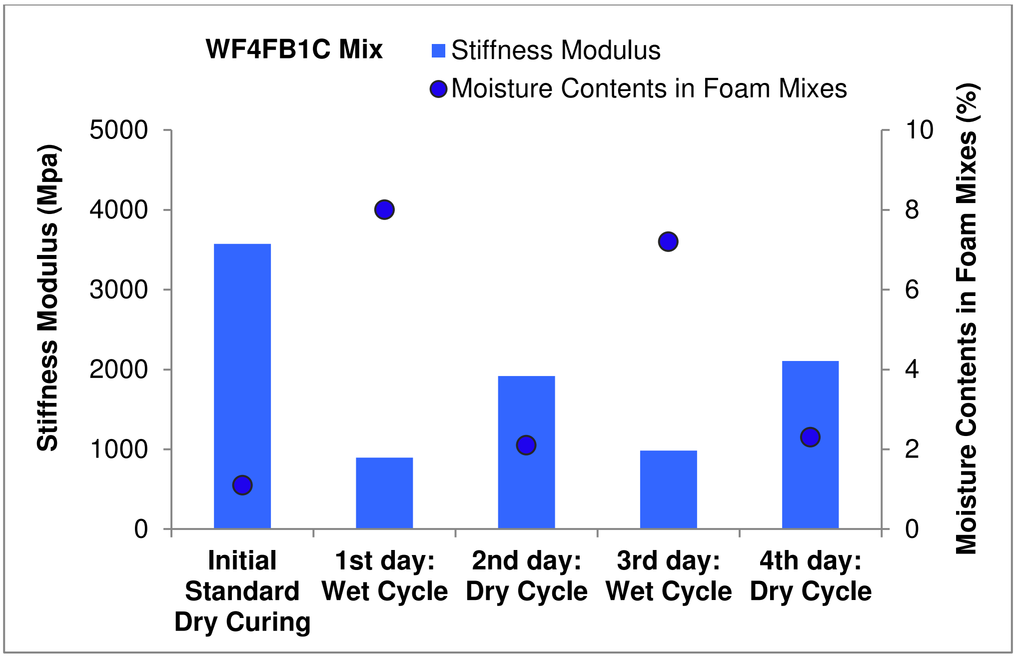


Figure 11.8 Effect of moisture contents in the WF4FB1C mix on its stiffness modulus

11.5 SUMMARY

The durability of the foam mixes is weakened by repetitive dry-wet cycles. When a high amount of moisture infiltrated into the foam mixes, the stiffness modulus reduced. Some foam mixes could recover and regain stiffness after being dried but some foam mixes were severely disintegrated under repeated introduction of water. This shows that some bonds can be damaged permanently by the action of water. Therefore, not all foam mixes are able to stay intact after reintroduction of water even if they are cured as stated by Fu (2009) but this could be due to the poor quality of sandstones used in this study which made the foam mix more sensitive to a wet environment. Perhaps some fines might have migrated from the foam mixes generating more voids. Hence, some foam mixes recovered to a slightly reduced stiffness modulus and others did not recover at all (e.g. 4FB and WF4FB).

Hydrated lime appears to be a less advantageous additive in comparison to cement, but does contribute to stiffness improvement and recovery, although to a lower degree than cement.

The WF4FB mix degraded the most amongst all foam mixes. It was hypothesized that the presence of WF in the binder would help in limiting the stripping of aggregates from the binder under a wet environment. However when Wet Fix blended bitumen was used to produce foamed bitumen which was then sprayed onto cold damp aggregates then the water repelling characteristics of WF appeared to prevent the foamed bitumen binder from adhering as thoroughly to the aggregates leading to a poor adhesion resulting in poor moisture-resistant foam mix. Further investigation would be required to ascertain any chemical interactions between the surface of the different components in the foam mix and also the characteristics of the foamed bitumen itself due the modifications of the neat bitumen used for its production. The changes in the foamed bitumen characteristics could result a poor foam mix.

Based on the findings of this study, the foam mix with 1% cement, 4FB1C, yielded the best climatic durability relative to all other additives evaluated. The cement has improved the stiffness and climatic behaviour from that of the foam mix without cement, 4FB.

CHAPTER 12 RUTTING PERFORMANCE OF FOAM MIX

12.1 INTRODUCTION

Rutting appears as longitudinal depressions along the wheel paths of the road surface and is caused by the accumulation of deformations of pavement materials due to repetitive traffic loads. It has been identified as one of the pavement distress types experienced by bituminous foam mix materials (Vorobieff, 2005). Therefore, for this reason, the assessment of rutting resistance is important in this study to ascertain the choice of foam mixture. The common test is by using a wheel tracking device. This generates a rut or surface deformation by the repeated passes of a loaded wheel at constant temperature. This chapter presents the results and findings from the Wheel Tracking test (WTT) (see section 6.2.4) such as the vertical surface deformation, wheel tracking slope and dynamic stability, to assess the rutting resistance of foam mixes.

12.2 EXPERIMENT DESIGN

The experiment was designed to evaluate the rutting resistance of those bituminous foam mixes listed in Table 12.1. The study included foam mixes manufactured using both mixing plants. The reason was that the earlier WTT tests were conducted on slabs manufactured using the old foamed bitumen plant, the WLB10, and the Hobart mixer. Due to the replacement by the new plant, the WLB10s and twin shaft mixer, later mixes used that plant while the early foam mixes test slabs had to be re-made to compare with later specimens. The foam mixes without cement were manufactured with different percentages of voids to see their effects on rutting performance. The foam mixes with 10% voids were then treated with cement additive at a percentage of 1% and 2% (by mass of dried aggregates). The findings would determine the preferred foam mix. A test temperature of 60°C was selected (see chapter 5) in order to asses rutting under the worst conditions in the field. Following the standard procedure A (BS EN 12697-22), the test duration is six hours, or until the rutting reaches 15mm.

Table 12.1 Mixture codes for each foam mix

Mixture Codes	**Descriptions**
$4FB_{old}$ (15% Voids)	Standalone bituminous foam mixes containing 4% Foamed Bitumen Contents (by mass of dry aggregates) and designed with 15% voids
$4FB_{new}$ (15% Voids)	As $4FB_{old}$ but manufactured using New WLB10s and Twin Shaft Mixer
$4FB_{new}$ (10% Voids)	As $4FB_{new}$ but with 10% voids
4FB1C	As $4FB_{new}$ (10% voids) but with the addition of 1% cement (by mass of dried aggregates)
4FB2C	As $4FB_{new}$ (10% voids) but with the addition of 2% cement (by mass of dried aggregates)
$4FB_{new}$ (5% Voids)	As $4FB_{new}$ but with 5% voids

12.3 SAMPLE PREPARATION

To manufacture slab specimens using the new plant, a large mass of materials of about 16kg was used. Since the twin shaft mixer could hold a minimum of 12kg and maximum of 30kg materials, each mixture type could be mixed and prepared in one batch.

For the cement-treated foam mixes, the cement was premixed with dried aggregates in the drum mixer. The MMC used in these foam mixes was equal to 70% of OMC in the form of water added to the aggregates. For cement-treated foam mixes, the cement was added and mixed with the dried aggregates for about 30 seconds prior to adding the MMC. Then after the MMC was added, the wet aggregates and cement mixtures were continuously mixed for one minute. Whilst mixing, the foamed bitumen was squirted onto the mixtures and they were continually mixed for another one minute.

The loose foam mixes were then riffled to fill each of the slab moulds with the required mass to produce two replicates for each mixture type. The required mass of foam mixes to fill the mould was calculated by multiplying the designed density by the volume of slab. All slab specimens were compacted by using a roller compactor to produce a dimension of 305 x 305 x 50mm thickness for WTT.

All test specimens were placed in the oven at a temperature of 30°C for the first 24 hours to gain cohesion before they were de-moulded from the compactor's mould to the WTT's steel moulds. Following that, the test specimens were dry cured at a temperature of 40°C for three days and then these were weighed to calculate their density before commencing the WTT test. The test slab was then mounted on the WTT's table. Care was taken to ensure that the specimens fitted exactly without any allowance for movement in the mould. All the specimens were conditioned to the testing temperature of 60°C in the WTT's cabinet for at least six hours prior to testing. A thermocouple was attached to the specimen by drilling a hole approximately 20mm under the surface to monitor the temperature. The surface was lightly dusted with talc on the wheel path to minimise adhesion between the tyre and the asphalt.

12.4 WHEEL TRACKING TEST RESULTS AND ANALYSIS

Some specimens may rut excessively at the early stage of the rutting test compared to others yielding high rut depth values. This might be caused by the densification and localisation of binders along the wheel path. In this case, to identify the effect of early excessive rutting, two parameters are commonly considered in a wheel-tracking test to help evaluate the rutting performance of bituminous materials.

The first is the dynamic stability of the foam mixes as seen in Equation 12.1 (China Standard T0719, 2000). The standard time for measuring rut depth is at 45 minutes (t_1) and at 60 minutes (t_2).

$$DS = \frac{t_2 - t_1}{d_2 - d_1} \times N \qquad \text{Equation 12.1}$$

where

DS : Dynamic Stability (Load cycles per mm)

N : Number of load cycles per minute

t_1, t_2 : Time at 45 minutes and 60 minutes respectively (mins)

d_1, d_2 : rut depth reached in 45 minutes and 60 minutes respectively (mm)

The second measure is the wheel tracking slope. It is regarded as the primary measure of the resistance to rutting (BS EN 12697-22, 2003) and it is calculated using Equation 12.2.

$$WTS_{Air} = \frac{d_{10000} - d_{5000}}{5}$$ Equation 12. 2

where

WTS_{Air} : Wheel tracking slope in air (mm per 1000 load cycles)

d_{5000}, d_{10000} : Vertical Deformation after 5000 and 10000 load cycles (mm)

In this WTT, the LVDT measures between the resultant pavement surface level and the start reference level (newly compacted surface) and thus, defines a vertical surface deformation rather than a rut depth as illustrated in Figure 12.1. Therefore, the following graphs, Figure 12.2 and Figure 12.3, use the term vertical surface deformation.

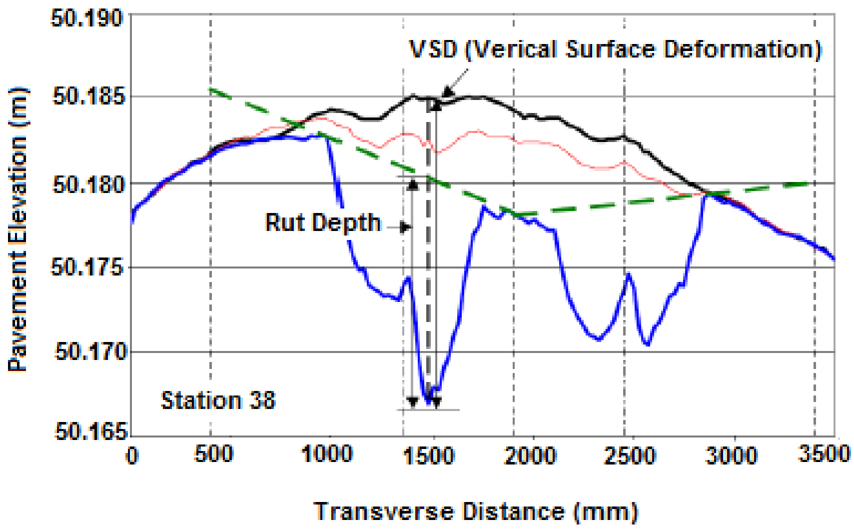


Figure 12.1 Comparison of rut depth and vertical surface deformation for a pavement (Arnold, 2004). Black line is original surface; Blue line is after rutting.

Figure 12.2 shows plots of vertical surface deformation experienced by the foam mixes with and without cement. The foam mixes without cement were produced with different void contents and the others were produced with cement contents of 1% and 2%. Clearly, cement treated foam mixes have a good resistance to rutting with the 4FB1C mix being the best mix. For the foam mixes without cement, the vertical deformation curves can be seen to continually increase at a steady rate resulting in higher vertical surface deformations than the cement treated foam mixes. This indicates that the foam mixes without cement can be expected to reach rutting failure more rapidly than the cement-treated foam mixes.

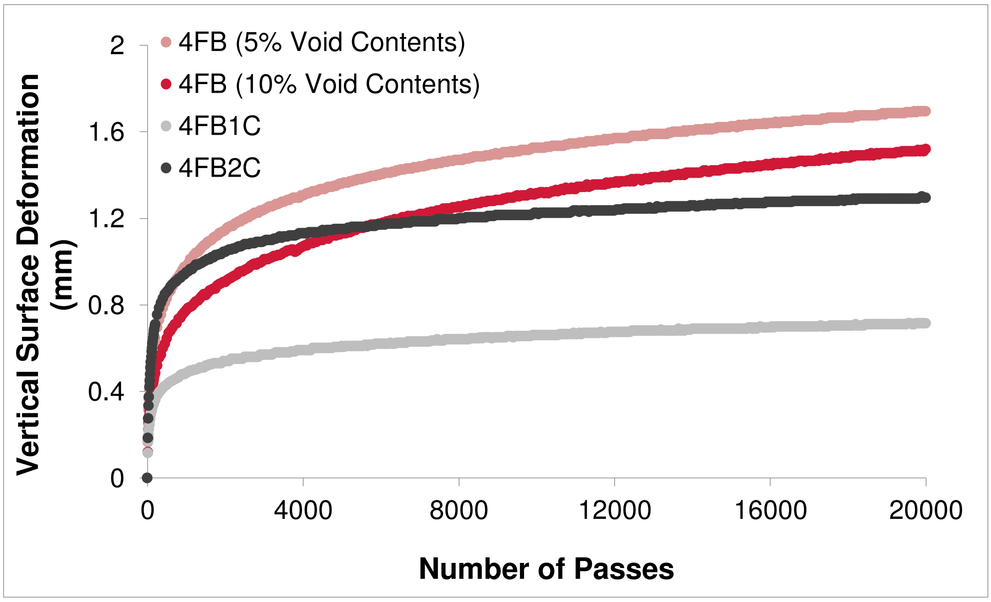


Figure 12.2 Average vertical surface deformation for each foam mix type

Figure 12.3 compares the foam mixes with different void contents and mixing processes on plots of vertical surface deformation. It can be seen that the foam mixes with low voids (5% or 10% voids) could resist rutting failure better than those with an open void structure (15%

voids). Another finding was that the rutting resistance can be influenced by the mixing techniques. The largest deformations, $4FB_{old}$, were observed for the mix which was manufactured using a Hobart Mixer that has a small capacity; therefore several batches were carried out to produce the required mass of one slab specimen. The consistency of each mixture produced from one batch might, therefore, vary from one to another. In contrast having a large capacity mixer like the Twin Shaft Mixer, a large mass could be mixed in one batch and then be riffled into two slab specimens. With the latter, the production could be consistently controlled to produce a homogenous mixture that is more highly rutting resistant than the ones manufactured by the small mixer.

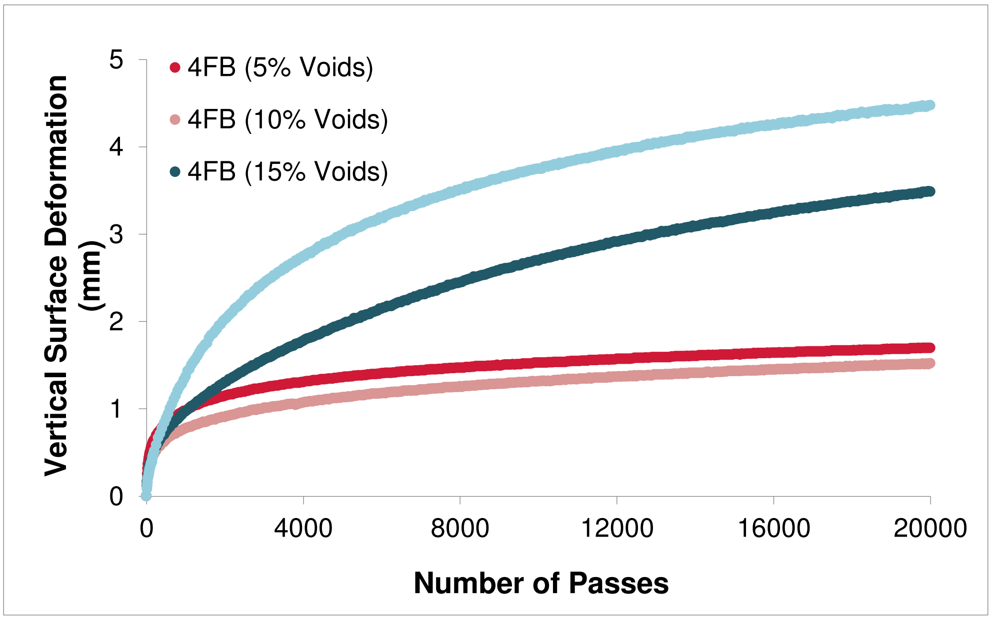


Figure 12.3 Average vertical surface deformation for standalone bituminous foam mixes

Overall this WTT has shown that none of the foam mixes reached the standard rutting limit of 15mm when the test terminated at 10000 load cycles. As stated by many studies (Dawson and Kolisoja, 2004, Sunarjono, 2008, Jitareekul, 2009) there was an initial densification of the mixtures, which was continued by the wheel tracking until the particles reach their interlocked state. The following graphs, Figure 12.4 and Figure 12.5, show the analysis of rutting profiles obtained by all the foam mixes in term of dynamic stability and wheel tracking slope.

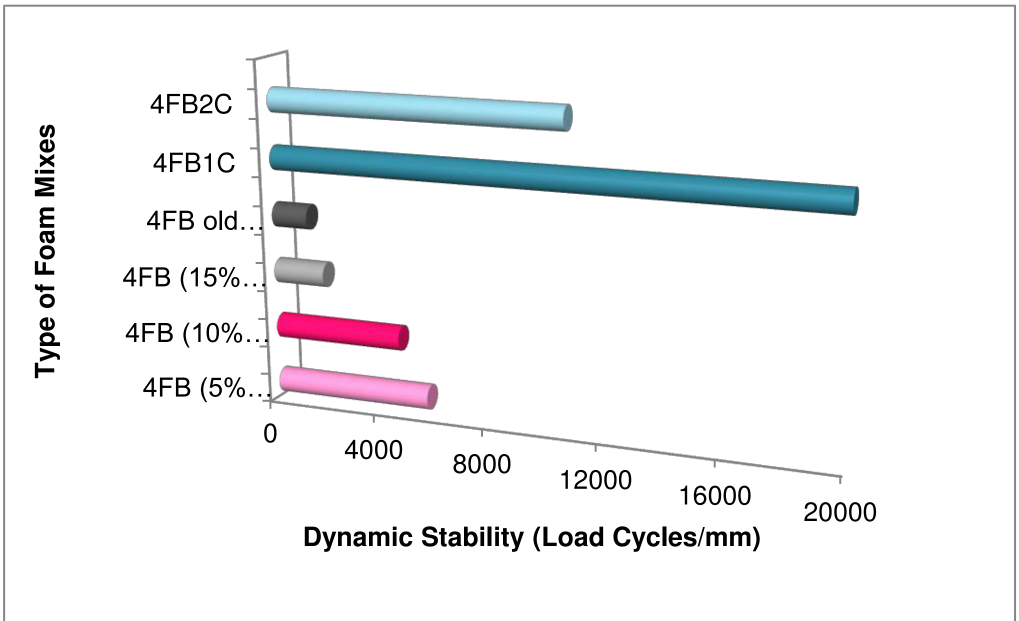


Figure 12.4 Dynamic stability of the foam mixes

With added cement, the dynamic stability (DS) of the bituminous foam mixes is larger than that of the foam only mixes regardless of void contents (Figure 12.4). The addition of 1% cement makes a positive contribution to the rutting resistance by a factor of about four. For 2% cement content, the contribution is somewhat less and, moreover, the 4FB2C mix seems to be a heterogeneous mixture perhaps due to the high cement contents causing lower workability and poorer distribution of binder. These appear to cause the mixture to become more brittle and prone to rutting. Nonetheless, the 4FB2C has higher rutting resistance than the solely foam mix, regardless of void content. The lowest DS is found for the $4FB_{old}$ manufactured by the Hobart mixer.

The graph in Figure 12.5 shows the results of the wheel tracking slope for each foam mix, which was calculated by Equation 12.2 to determine the average vertical surface deformation of the bituminous mixtures per 1000 load cycles during the latter part of the test. It can be observed that 4FB1C mix has the least surface deformation amongst all and the foam mixes with 15% voids deformed the most, regardless of the mixing technique. It can also be seen, by comparing Figure 12.4 and Figure 12.5, that the Dynamic Stability and Wheel Tracking Slope are, approximately, inversely correlated.

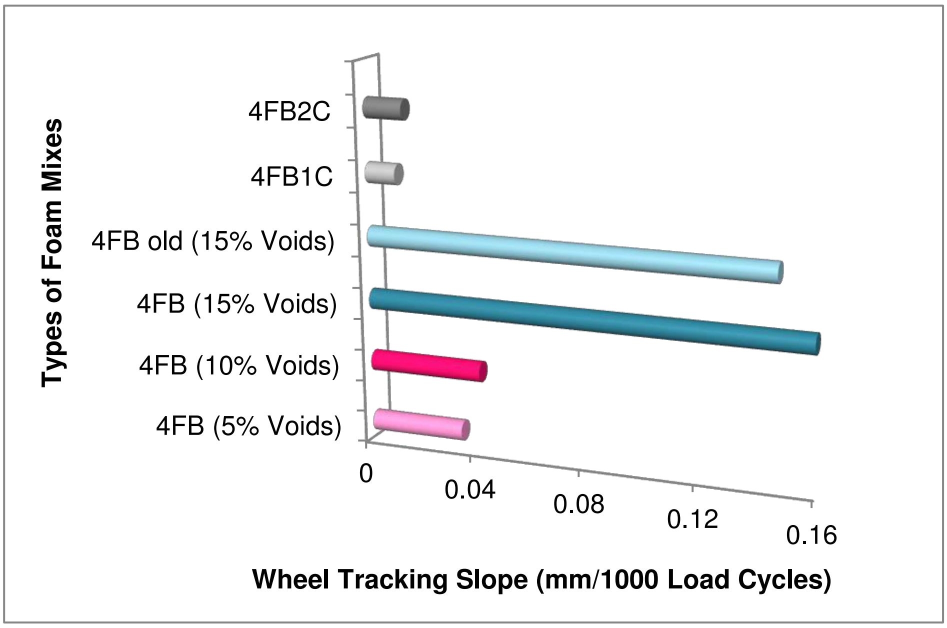


Figure 12.5 Wheel tracking slope for the foam mixes

12.5 SUMMARY

None of the foam mixes reached the standard rutting failure of 15mm when the test terminated at 10000 load cycles.

All foam mixes experienced a rapid increase in the vertical deformation in the early stage of wheel tracking tests. In the case of foam mixes, the presence of a localised soft spot of foamed mastic along the wheel path could be one of the causes apart from reduced inter-granular locking of aggregate particles. This could be manifested by the formation of cracks on the test slab's surface along the side of the wheel track.

Densification seems to continue under the tracking of the wheel load for most of the solely foam mixes, whereas, the cement-treated foam mixes seem to exhibit non-increasing constant deformation values following early-life rutting and had the least deformation value per 1000 wheel load cycles.

Based on the findings, the 4FB1C mix is indicated to have the best rutting resistance of all the foam mixes. Its high dynamic stability indicated that the addition of cement enhanced the high temperature stability (60°C) of the foam mixes.

CHAPTER 13 MICROSTRUCTURE OF THE FOAM MIXES

13.1 INTRODUCTION

This chapter aims to study the microstructure of foam mix with a view to obtain understanding of the interactions between the elements in the mix and correlations between the micro and macro behaviours in the laboratory will be discussed. The observation of the microstructure is done by imaging techniques, namely quantification of selected features in the foam mix using image analysis software and direct analysis of internal structures using a microscope.

One of the techniques used in this study was by acquiring camera captured-images on a fracture face obtained from the splitting test, ITS, in which the images were processed and analysed for easy quantification. It aims to quantify and compare the distribution of foamed mastic in two bituminous foam mixes, 3FB and 4FB. Another technique is a direct observation using a Scanning Electron Microscope (SEM). SEM allowed the author to diagnose the microstructure, revealing important features that could not be seen by the naked eye such as the interface between the elements, particle bonding, crystals (in case of cement-treated mixes) and voids. The processes of both imaging techniques are described and discussed in the following sections.

13.2 RATIONALE FOR THE MICROSCOPIC ANALYSIS OF FOAM MIX

Foam mix is generally regarded a multi-phase composite material, as shown schematically in Figure 13.1, made up of (Fu et al, 2010):

- The coarse aggregates, which form the skeleton

- Foamed mastic, made of bitumen bonded with fines, the only bound part in the mix, and concentrated in spots holding the aggregate skeleton

- Fines that remain uncoated by foamed bitumen during mixing; they fill voids within the aggregate skeleton

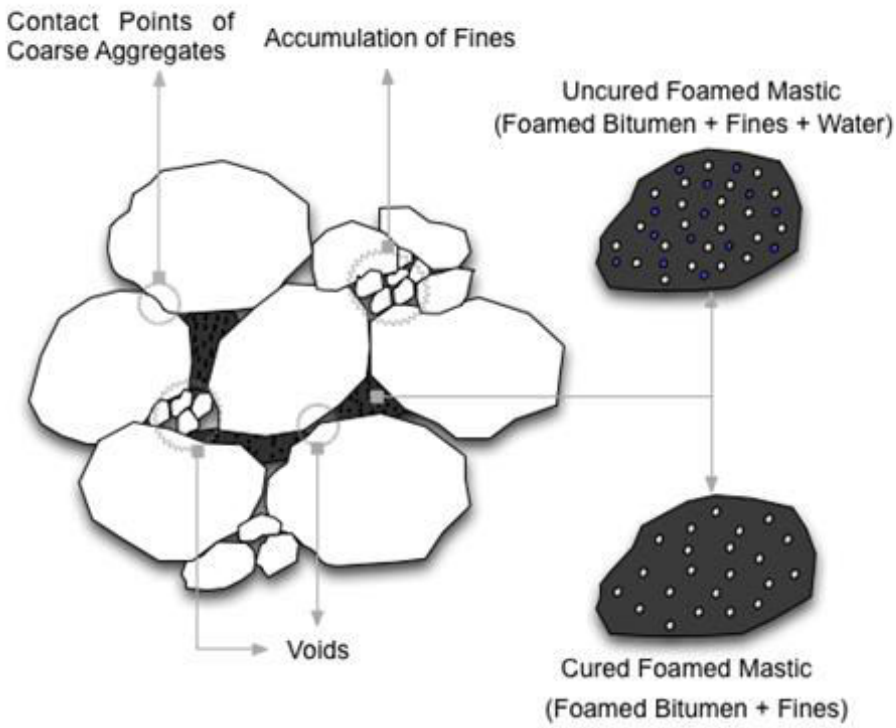


Figure 13.1 Schematic of the Phase Distribution in Foam Mix (inspired by Fu, 2010)

In this case, the microstructure of foam mix is important to gain an understanding of the interactions between the different phases. Most research studies on the evaluation of the macro properties of foam mix have been justified due to the influence of many physical factors such as the types of materials, production process and conditions. However, the microstructure of foam mix has not been extensively reported. Therefore, this study aims to correlate the macro properties of foam mix obtained in the laboratory to its microstructure. A direct representation of a foam mix's microstructure can reveal more features that could not be seen by the naked eye. These features may provide useful information and permit a better understanding of the reasons for its macroscopic behaviour.

13.3 IMAGING TOOLS

Available imaging tools include increasingly advanced analytical techniques with sophisticated equipment, both hardware and software. They offer unique opportunities that become more useful in the quantification of important features to characterise civil engineering materials through their acquired images.

There are two imaging tools used in this study, namely the digital image analysis software and scanning electron microscope (SEM) as described in the following sections.

13.3.1 Digital Image Analysis

An image is a numerical representation of a 'picture' or a set of numbers interpreted by a computer creating a visual representation that is understood by humans. It is normally digitised into picture elements or known as pixels. Each pixel contains a sample of original image containing intensity value.

Pictorial information in the digital images either in colour or grey scale format contains important features that can be manipulated through image processing and can be analysed for quantification of properties such as aggregate shape properties (Pan and Tutumluer, 2007). The image analysis software used was the 'Image Pro-Plus 4.5.1' with live view camera 'Evolution MP'. Both image processing and analysis can be carried out using the software program.

13.3.1.1 Image Acquisition

The samples were obtained from a cylindrical test specimen previously subjected to Indirect Tensile Strength (ITS) testing that initiated splitting into two fracture face pieces. The image of the fracture face was then acquired on a contrasting background.

A gray scale image is normally favoured as it represents pixel values of only different levels of greyness as opposed to Red, Blue and Green (RGB) format which is made up of three separate samples representing level of brightness of its respective colours. 8-bit grey scale images are the most common, allowing for effective processing of the image with its 1 byte per pixel size and providing 256 distinct levels of grey from black (intensity of 0) and white (intensity of 255) (Pan and Tutumluer, 2007). The software allows for a live coloured image or grey scale image that is previewed on the computer screen to monitor the positioning of the object and also to permit adjustment of the camera into full focus. When the image is visually satisfactory then it is acquired and saved.

It should be noted that calibration is essential when analysing images for geometric measurement purposes. The setting of the unit measurements is done in a spatial calibration command in the software. Once the calibration is set, care should be taken when handling the camera. Slight variation in the height of camera from the object can give imprecision in the measurement results.

13.3.1.2 Image Processing

An image of an object does not usually retain the natural features of the object as viewed by eye. The blurring effect, variation in intensity or presence of shadows would cause the loss of the essential nature of the object in the image acquisition. Therefore, image processing is a necessary step to correct the deficiencies in the image to prepare for analysis. However, it preferable not to overdo this processing as it could distort the original data in the image.

An image-processing tool called segmentation is used to extract the object of interest and distinguish it from other elements in the background image. It is applied by selecting the threshold value by adjusting the histogram-based graph at a level that visually distinguishes between the elements or points of interest present in the images, resulting in a binary image as shown in Figure 13.2. A binary image contains only two colours, black and white. In this way, the segmentation extracts the object of interest from the background (e.g foamed mastics or blobs). It contrasts and defines the contour of foamed mastic blobs from the coarse aggregates. It is a critical step for the success of quantification results as it defines the blobs which are evaluated in the later steps of the image analysis.

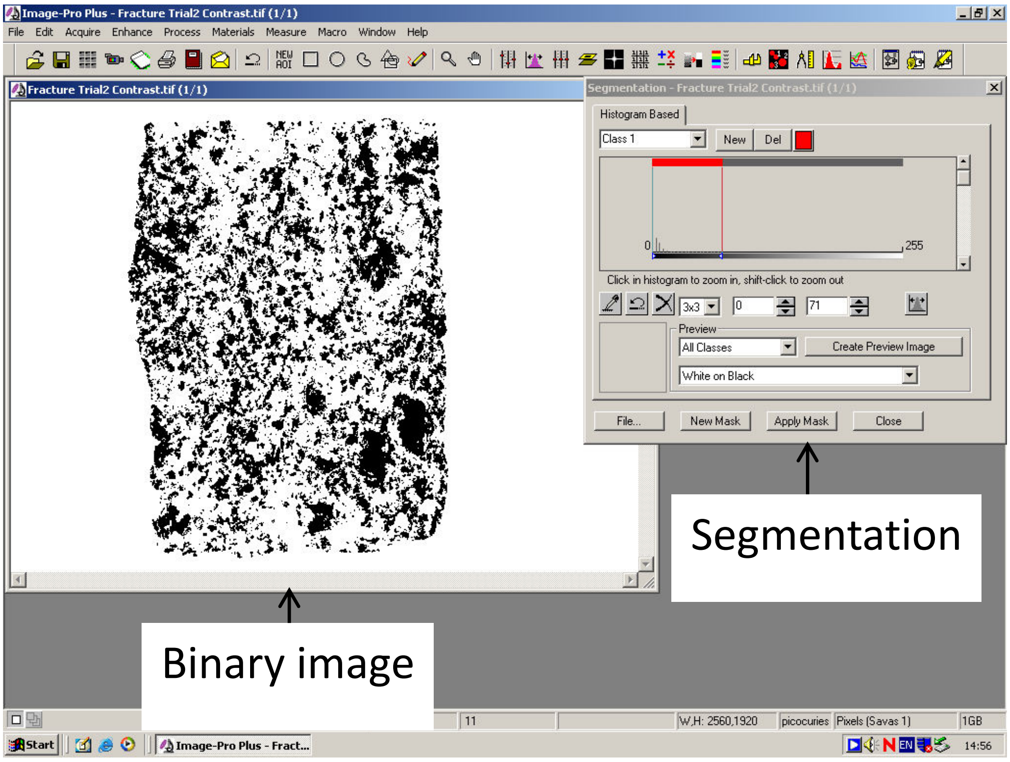


Figure 13.2 Segmentation based on histogram equalisation graph to produce binary image

The software enables the counting of the blobs as the object of interest by running an automatic counting command of bright and dark objects. With the automatic counting of dark object command, it automatically recognises and counts all the dark objects, each as one, in this case referring to the black colours. However, the function works well only if the area of interest has clear particle contours that can be distinguished from other elements in the image. Figure 13.3 shows the outlines of all dark objects found in the fracture face image by using the sorting objects command. Any geometrical parameters of the blobs can be quantified using this software such as areas, fractal dimension, perimeter, diameter and so on.

Figure 13.3 Dark objects as foamed mastic blobs

In image processing, it is important to use another adjustment process called filtering that is undertaken to reduce or increase the rate of change that occurs in the intensity transitions within an image. In general it is applied to eliminate or minimise noise points. The noise points naturally occur in most acquired images and can be found both inside and outside the object domain in the image. The sources of noise are mainly from the fluctuations of illumination or brightness level of the laboratory or fine motion of the camera due to draughts or faults in its mechanical parts during image acquisition. The noise points contaminate the image by providing undesired information and in this case, they would strongly affect the final profile of the object of interest and thus, later would influence the measurement results. Median filtering is commonly used as an effective function at removing random or high impulse noise from an image while preserving its sharp edges.

It is used to smooth an image by modifying pixels that vary significantly from their surroundings.

13.3.3 Scanning Electron Microscope (SEM)

SEM is a highly sophisticated technique that has the capability to generate high-resolution images to directly examine the details of the key characteristics of the microstructure that are invisible to the naked eye or in conventional digital images. It has been used to characterise asphalt film in bituminous mixtures (Mostafa et al, 2008). Images are normally captured at a low magnification and then repetitively enlarged at higher magnifications to reveal some distinct features that could be found within the matrix of the microstructure. SEM is normally operated in a high vacuum mode; it collects two types of electrons producing secondary electron and back scattered electron images. A secondary electron image shows a surface topographical structure of the sample and in contrast, a backscattered electron image reveals the compositional differences due to the different atomic numbers of the material compositions. Further elemental composition can be analysed and detected using the Energy Dispersion X-Ray (EDX) that is coupled to the SEM.

The SEM analysis was performed by a Philips Oxford INCAX-Sight FEI XL30 in both secondary electron and backscattered electron mode that was coupled with energy-dispersive X-ray analysis (using Inca energy program 350 for the treatment of data). Energy dispersive X-ray spectroscopy (EDX) is a method used to determine the energy spectrum of X-ray radiation. Especially the backscattered electrons may be used to detect the contrast between areas with different chemical compositions. This can be observed when the average atomic number of the regions is different.

It was operated at a voltage of 20kV, at a working distance of 10mm. The magnification level was varied depending on the required level of detail. The magnification level and particle sizes can be determined on the scale bar shown in each image.

13.4 ANALYSIS OF MICROSTRUCTURE

13.4.1 Digital Image Analysis of Mastic Blobs

This part of the study was aimed to quantify the foamed mastic blobs (dark objects) within the foam mixes, 3FB and 4FB, in terms of area. The frequency of each area was recorded and plotted. However, due to software limitations, full confidence in the values of the area cannot be assured. Therefore, the results should be treated as indicative. It should also be noted that the voids within the mix might appear as dark objects which could be calculated as the total area of the mastic blobs by the software.

The areas of the blobs would indicate the differences between the two foam mixes in relation to their behaviours particularly when cement was added that had turned the 3FB mix into a rigid cementitious material while allowing the bituminous characteristic of the 4FB mix to be retained.

Figure 13.4 plots the frequencies of each counted area of the foamed mastic blobs for the 3FB mix. In contrast, the high bitumen content, 4FB mix, is clearly covered with many mastic blobs as shown in Figure 13.5.

For instance, the frequencies for the area of blobs less than 11mm^2 are less in the 3FB mix than the 4FB mix. The reason is due to the thin bitumen film in the 3FB mix, which ties in with the 3FB mix being physically brittle such that handling could damage the test specimen. In the 4FB mix, there are many blobs of these sizes (less than 11mm^2); hence, the 4FB mix was quite intact and much easier to handle than the 3FB mix.

The very large areas of mastic shown on the right of Figure 13.4 and 13.5 perhaps result from improper distribution of foamed bitumen. Another possible reason might be the presence of large voids which have been mistaken for mastic.

It is clear that the 4FB has more bituminous characteristics than the 3FB mix. Having a poorer distribution of bitumen to form mastic in the 3FB mix, it would give opportunity, for an additive such as cement, to occupy more spaces where the binder can act to generate rigid cementitious characteristics.

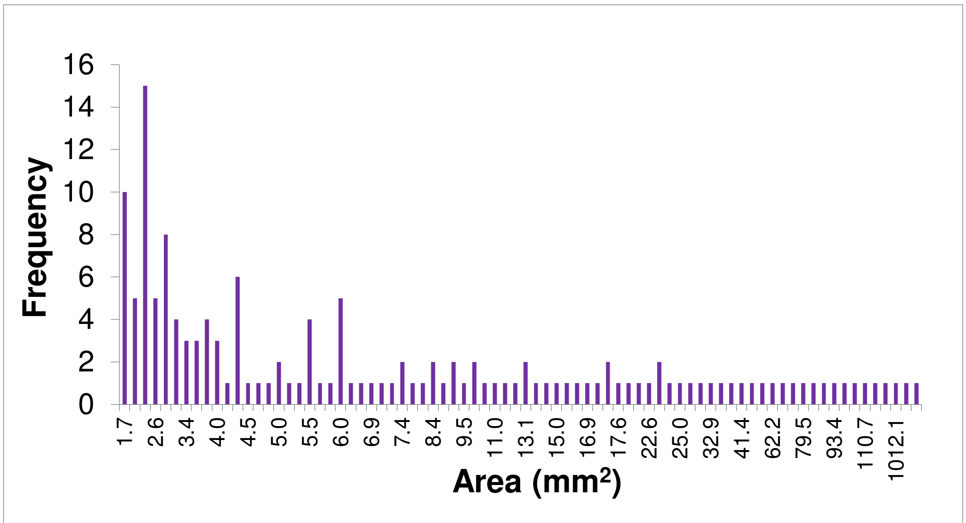


Figure 13.4 Frequencies of each mastic blob's area for the 3FB mix

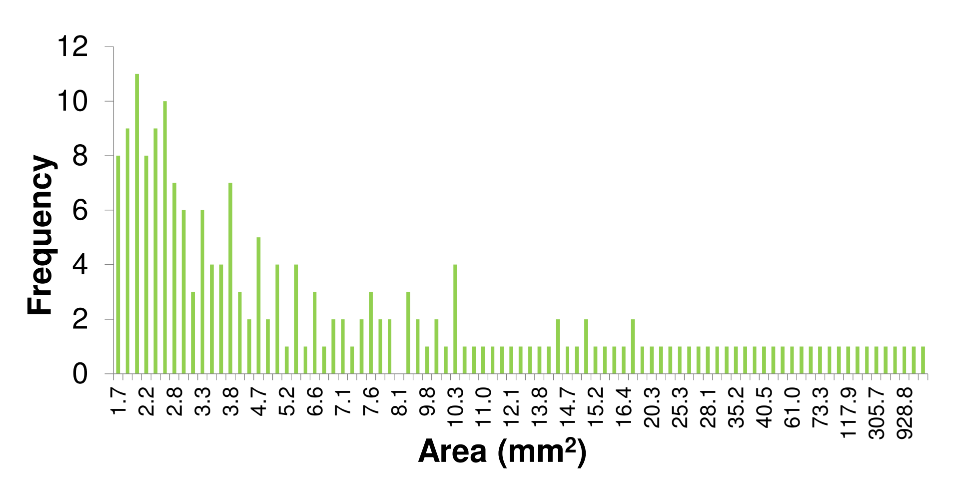


Figure 13.5 Frequencies of each area of the foamed mastic blobs for the 4FB mix

13.4.2 SEM Micrograph Analysis

To aid in understanding of the 'source of weakness' in the foam mixtures, they were further examined at micro-scale using SEM. Due to the multi-phase features of cement-treated foam mix, these were selected to be examined in this study as well as the coir fibre reinforced foam mixes.

The foam mix samples were taken from broken tested cylindrical specimens and selected based on their best flat surface for SEM purposes. In SEM, the sample is required to be electrically conductive by coating it with elements such as carbon, aluminium or platinum, whichever coats the samples best. In this case, the coating used is platinum as it is the most conductive. Prior to coating, the sample was cemented using carbon cement mortar, onto an aluminium stub and allowed to set overnight. It was then placed in a vacuum to pump out the air from the pores prior to evaporating the coating onto it. Once completed, the sample was then ready to be placed in the SEM chamber for further examination.

In this process, the micrograph images are captured first at a low magnification and then repetitively enlarged at higher magnifications to reveal different features on the surface.

13.4.2.1 Microstructure of Cement Treated Foam Mix

Cement, when dispersed in water, undergoes a hydration reaction. The main phases present in hydrated cement products are Calcium Hydroxide ($Ca(OH)_2$), Calcium Silicate Hydrates (CSH) and ettringite (Hydrated calcium aluminium sulphate). The morphology of $Ca(OH)_2$ is a hexagonal plate-like structure, CSH forms foil-like structures and the ettringite is a needle-like structure (Han et al, 2012). As hydration proceeds, the hydration products progressively fill the void spaces in the foam mix. As the void spaces become less, the foam mix decreases permeability leading to an increase in durability of the foam mix.

Figure 13.6 and 13.7 compare the microstructure of foam mixes containing the same binder content, 3%, but different cement content, 1% and 2% respectively. A mass of flocculent structure of CSH gels almost occupies the foam mix with high cement content, 2%, whereas the foam mix with low cement content shows lesser bundles of CSH gels. This can be correlated to the high stiffness modulus value obtained by the 3FB2C that developed concrete-like properties such as rigidity, thus losing the flexibility typical of bituminous materials.

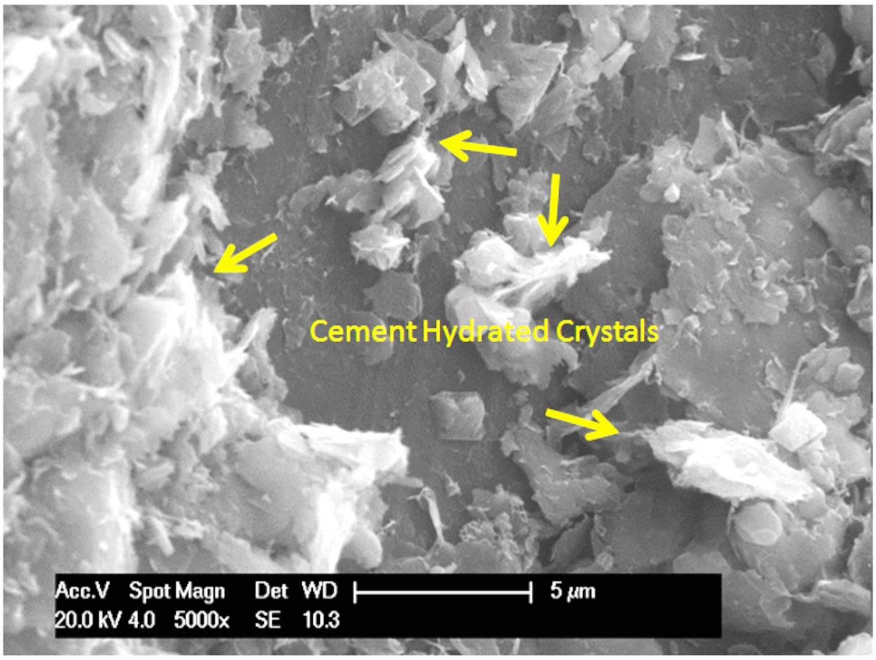


Figure 13.6 Microstructure of foam mix, 3FB1C, with 1% cement contents.

Figure 13.7 Microstructure of foam mix, 3FB2C, with 2% cement

In contrast when the 3FB2C is compared to 4FB2C as shown in Figure 13.8 and Figure 13.9, it can be observed that more bundles of CSH gels and distinctive needle-like structures of ettringite occupy the foam mix with low foamed bitumen content, 3%, whereas, with the high foamed bitumen content, the dark soft bitumen spot still can be observed with lesser bundles of CSH gels. The more numerous bundles of CSH gels supports our understanding of the previous stiffness modulus results where the 2% cement contents exhibits higher stiffness in foam mix with 3% foamed bitumen than with 4% foamed bitumen.

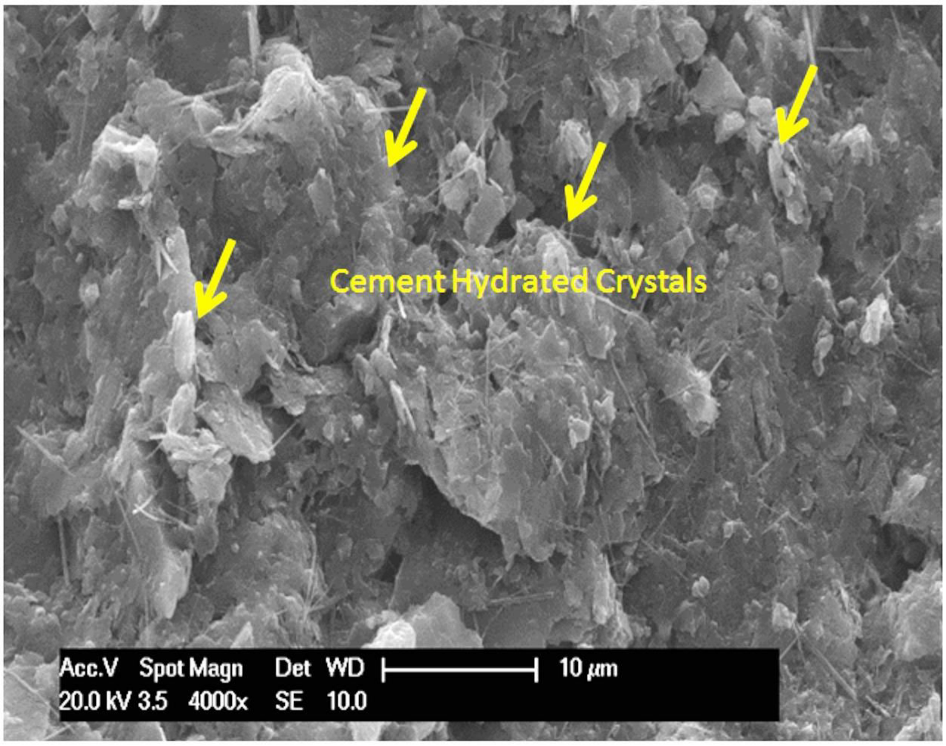


Figure 13.8 Microstructure of foam mix with addition of 2% cement containing low binder content, 3FB2C.

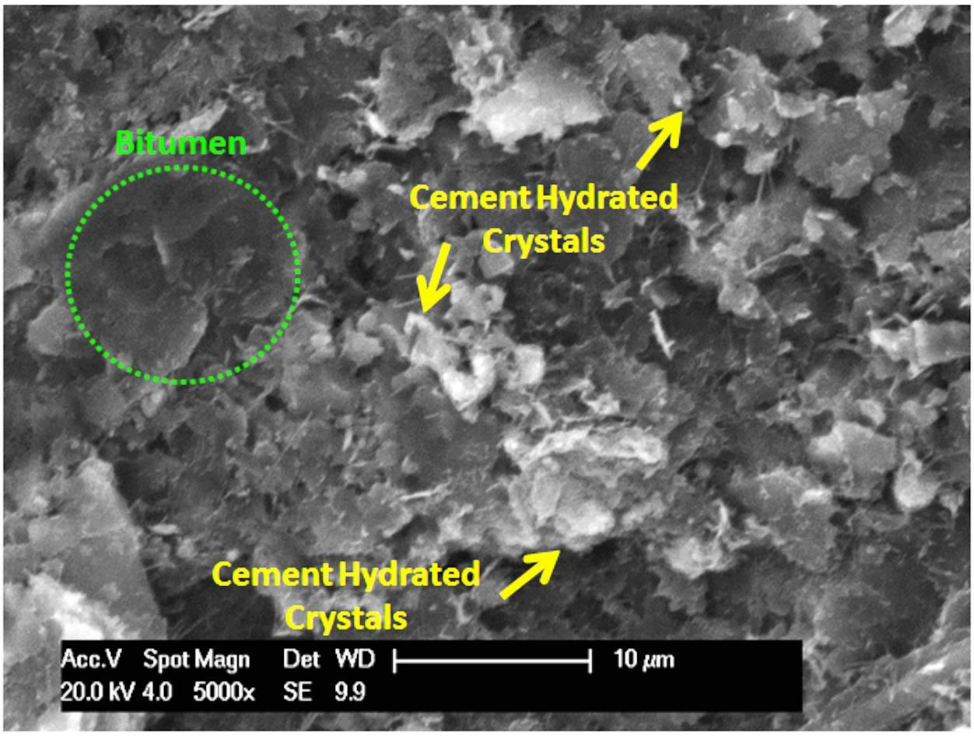


Figure 13.9 Microstructure of foam mix with addition of 2% cement containing high binder content, 4FB2C.

Further evidence can be seen in Figure 13.10, an image taken from another area of foam mix 4FB2C, presenting the main particle bonding at higher magnification. The bond in foam mix containing 4% foamed bitumen is made up of mainly bituminous foamed mastic, as compared to Figure 13.11, in which the foam mix containing 3% foamed bitumen displays structures of CSH gels as the main bonding material. Even though the two images do not have the same magnification levels, the areas are enough to compare the particle dominant bonding dominated in the foam mixes. Hence, it is an interesting point to note that not all foam mixes containing 2% cement content show dominating concrete-like properties, particularly when the foam mix is produced with high foamed bitumen content, 4%.

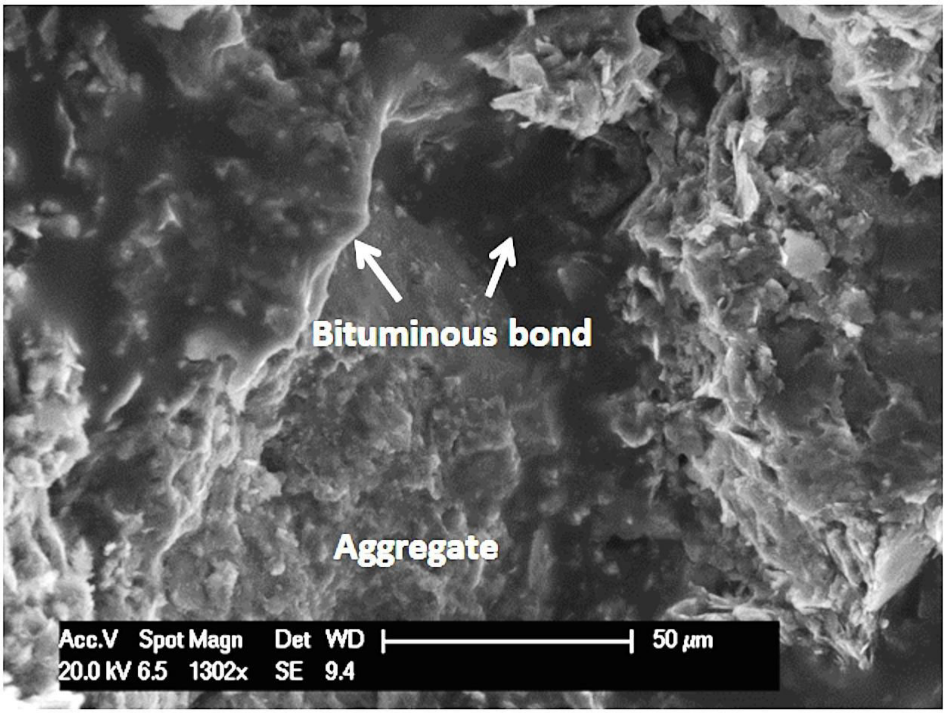


Figure 13.10 Bituminous bonding in the foam mix, 4FB2C

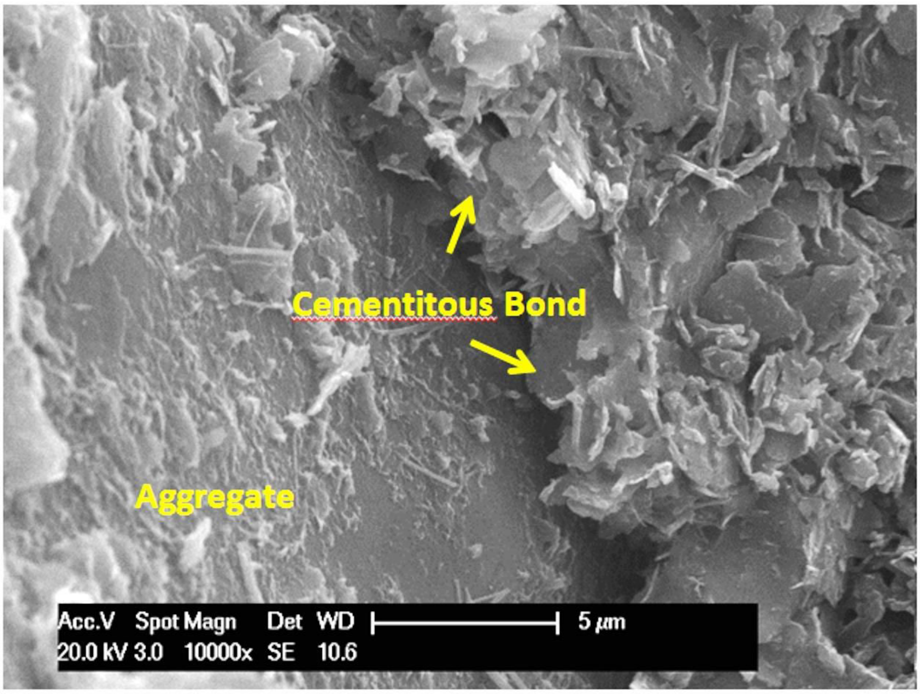


Figure 13.11 Cementitious bonding in the foam mix, 3FB2C

As mentioned earlier, foam mix has poor water resistance: this fact is clear in the sample SEM micrograph images of mixes after soaking as presented in Figure 13.12, which shows weak and broken bituminous bonds taken from various areas with different magnification levels. When water infiltrates the mix, it tends to break the bituminous bonds leaving spaces for water to channel. However, the addition of cement in the foam mix helps, as the water activates its hydration process and the CSH gels begin to occupy these water-filled spaces. Hence, cement addition in the foam mix improved stiffness significantly under wet curing compared to the foam mix with no cement.

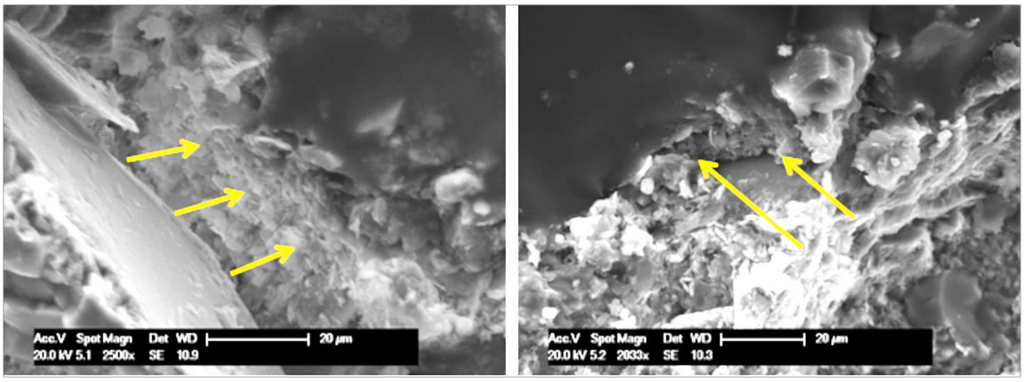


Figure 13.12 Cement interactions in repairing the damaged bonds in foam mixes

13.4.2.2 Microstructure of Coir Fibre Reinforced Foam Mix

From the laboratory evaluation, it was found that the addition of coir fibre as a reinforcement agent did not help the foam mix to gain stiffness and strength particularly in a wet environment, although they did delay the development of large cracks. The micrograph analysis reveals some points of weaknesses in the coir fibre-reinforced foam mix as shown in Figure 13.13.

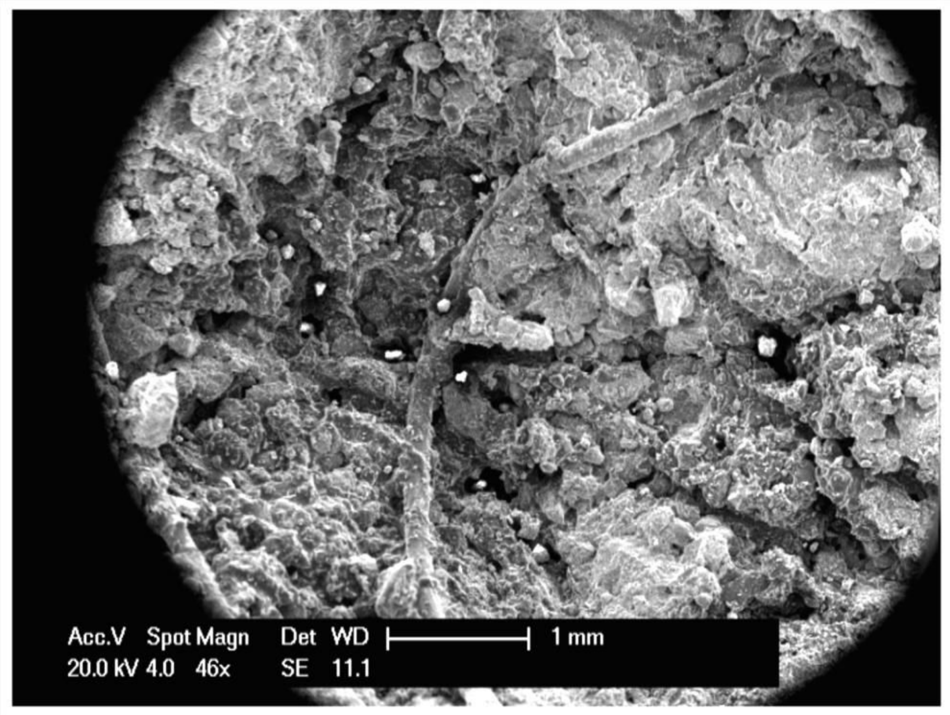


Figure 13.13 Microstructure of coir fibre reinforced foam mix

Figure 13.14 shows a poor bonding of coir fibres in the foam mixes where non-continuous bonding can be observed from this micrograph. The non-continuity generated voids in the mixes that, in the wet environment, would allow more water to infiltrate and deteriorate the mixture as discussed in Chapter 9.

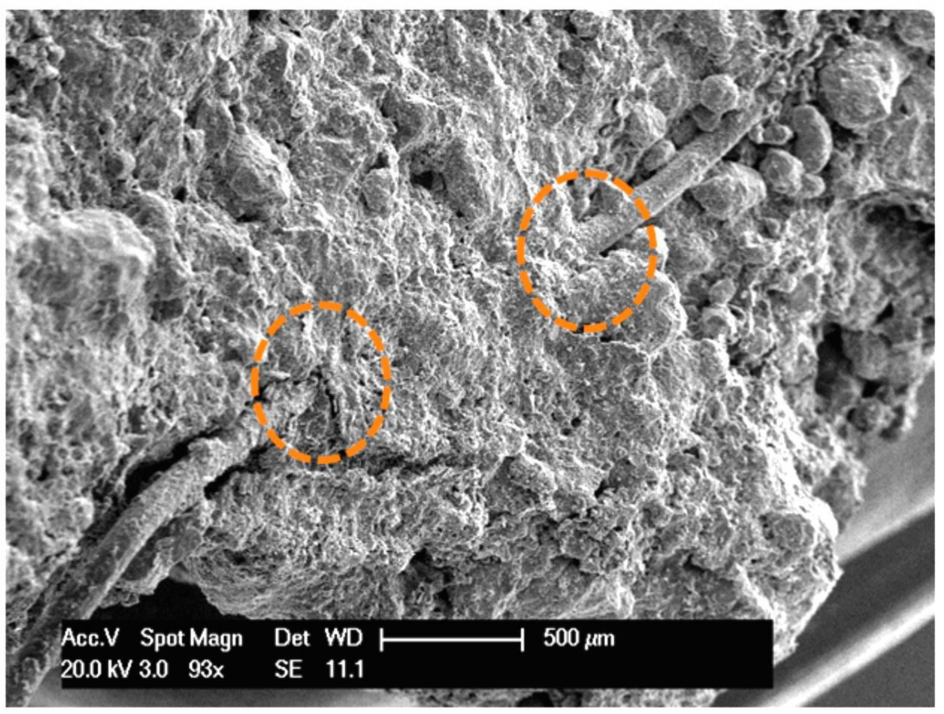


Figure 13.14 The non-continuous bonding of coir fibre in the foam mixes

Figure 13.15 and 13.16 present micrographs of coir fibres coated with bituminous mastic at different areas forming a random network across the foam mixes. The addition of coir fibres leads to an increase in the surface area in the mixes reducing the amount of bituminous mastic bond available to hold the aggregate skeleton. It can be observed that some fibres seem to protrude from the matrix.

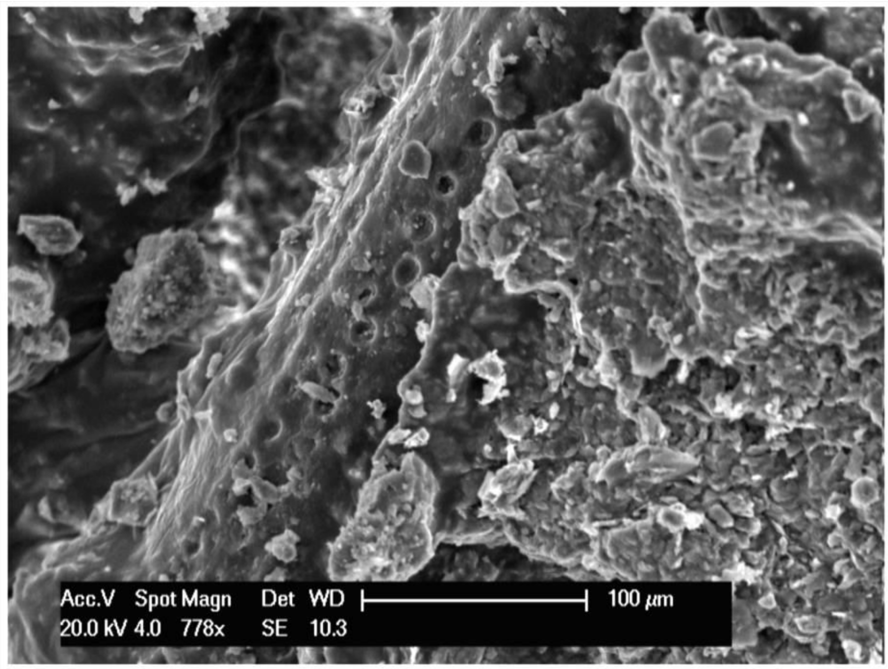


Figure 13.15 Coir fibres coated with bituminous mastic in the matrix

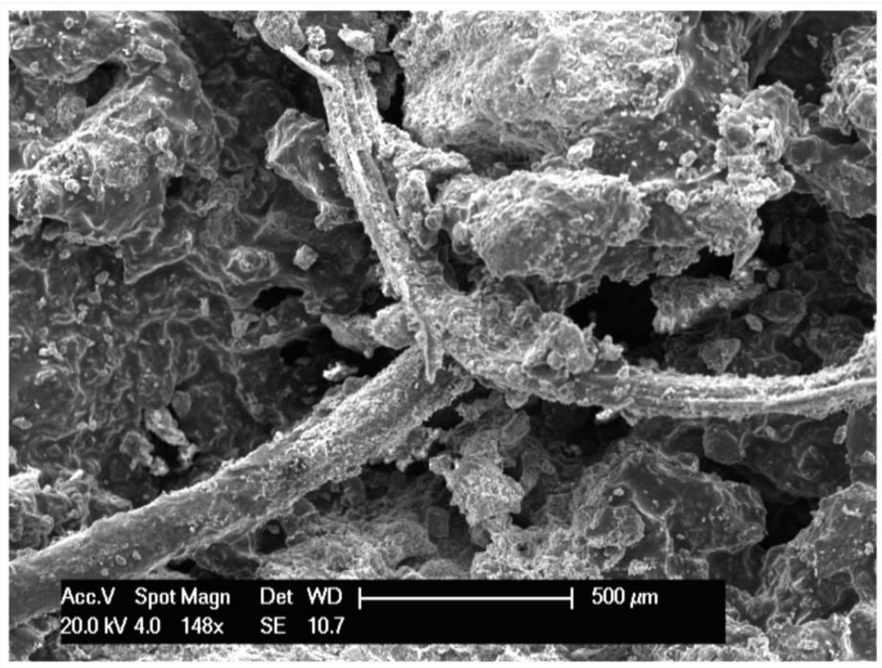


Figure 13.16 Coir fibres coated with bituminous mastic forming a random network in the foam mixes

Due to the lesser, effective, content of bituminous mastic, some fibres might have detached from the matrix as seen in Figure 13.17. Traces of detached fibres can be observed with the concave 'print' of the fibres. The results indicate that the interfacial bonding between the fibre and the matrix is weak.

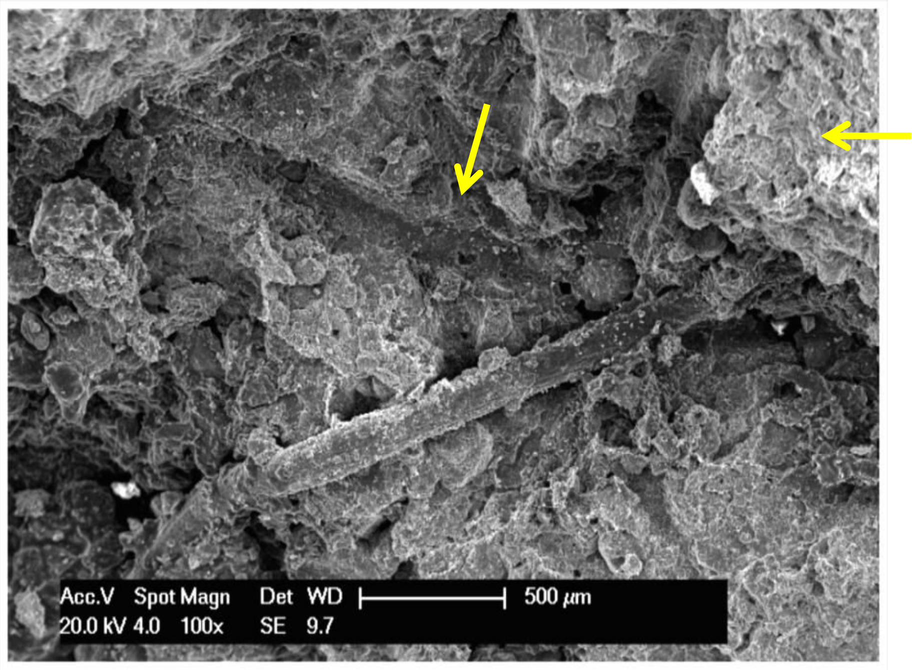


Figure 13.17 Concave 'print' of the coir fibres

The preliminary laboratory investigation of coir fibre-reinforced foam mix revealed that it has the capability of delaying the development of large cracks. The images in Figure 13.18 show some induced cracks due to the test loadings. When the load is applied to the coir fibre-reinforced foam mix, the matrix began to crack after which stress appears to have transferred to the fibres. This can be seen by the bridging effect of the fibres as shown in Figure 13.19. Although this is evidence of the fibres hindering the development of larger cracks in the mixes, they are not feasible for use as reinforcement in the foam mixes due to their poor interfacial bonding in the matrix. The poor interfacial bond would generate more voids resulting in severe damage particularly in the wet environment as discussed in the Chapter 9.

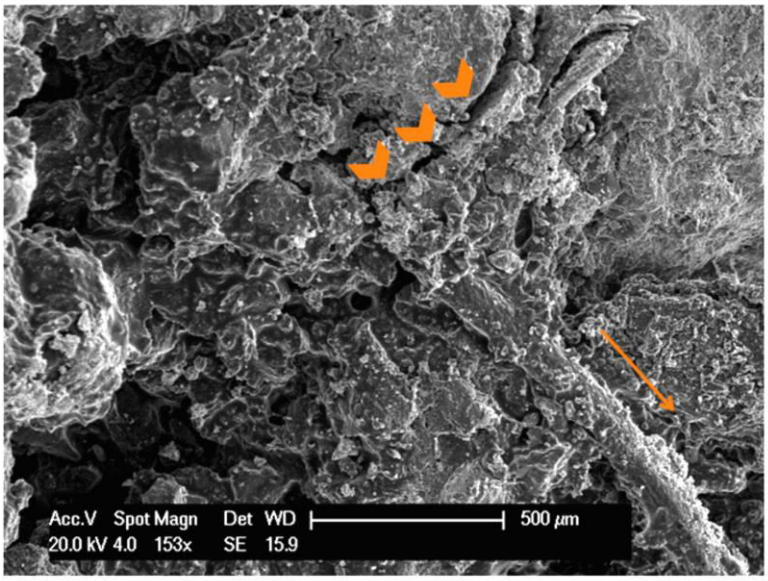


Figure 13.18 Cracks in the foam mix

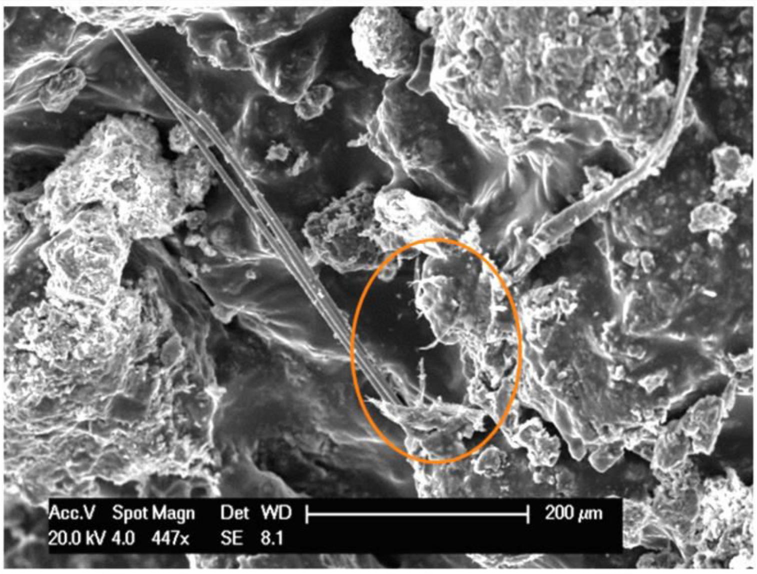


Figure 13.19 The pulling and bridging effects of the coir fibres

13.5 SUMMARY

Two imaging tools were used in this study namely, digital image analysis software and scanning electron microscope.

The low frequencies of small mastic blob areas in the 3FB mix provide an opportunity for an additive, such as cement, to occupy the open spaces in the mix. In contrast, the high frequencies of small mastic blobs in the 4FB mix indicate the good distribution of foamed bitumen resulting in a flexible and ductile bituminous behaviour.

Cement, when dispersed in water, undergoes a hydration process forming calcium hydroxide, calcium silicate hydrates and ettringite. The products tend to fill the void spaces to form a dense heterogeneous matrix. When water is reintroduced, the hydration process can be reactivated in the presence of residual cement, providing opportunity to repair the damaged bonds in the mix.

Generally, an addition of high cement content i.e 2%, produced more cement-hydrated products resulting in a higher stiffness mixture; however, it only applies to the foam mix containing 3% foamed bitumen content. When 4% foamed bitumen content is selected to produce foam mix with 2% cement, the benefits of foam mix (being flexible) remain.

Although the coir fibres have the capability to prevent the development of the initiated cracks, the poor adhesion to the foam mix matrix with the fibres might not improve the moisture sensitivity of the foam mix.

CHAPTER 14 PAVEMENT ANALYTICAL STUDY

14.1 INTRODUCTION

This chapter focuses on the application of foam mix for a potential pavement structure in Brunei Darussalam. The main objective is to predict the pavement life expectancy when the selected foam mix is used as a road base layer, in terms of the relationship to the numbers of traffic loads expressed as Equivalent Standard Axle Load (ESALs). The selected foam mix would be the preferred mix as identified by the previous laboratory tests. The methodology and analysis are discussed in subsequent sections. Bitumen Stress Analysis in Roads (BISAR) was the main software program used that computes comprehensive calculations to provide stress and strain values in a selected pavement configuration. General inputs and assumptions are discussed in this chapter.

14.2 PAVEMENT FAILURE CRITERION

Expressed simply, failure of a pavement can be said to be induced by the stresses from the traffic wheel loadings causing fatigue cracking of the bound asphalt layer and rutting, which is contributed to by all layers (Henkelom & Klomp, 1962) as shown in Figure 14.1. Fatigue is characterised by excessive horizontal tensile strain, ε_t, at the bottom of asphalt layer that leads to cracks that propagate upwards towards the surface. Rutting is characterised by the excessive vertical compressive strain, ε_z, at the top of the subgrade layer. These are the main critical strains in typical flexible pavement structures.

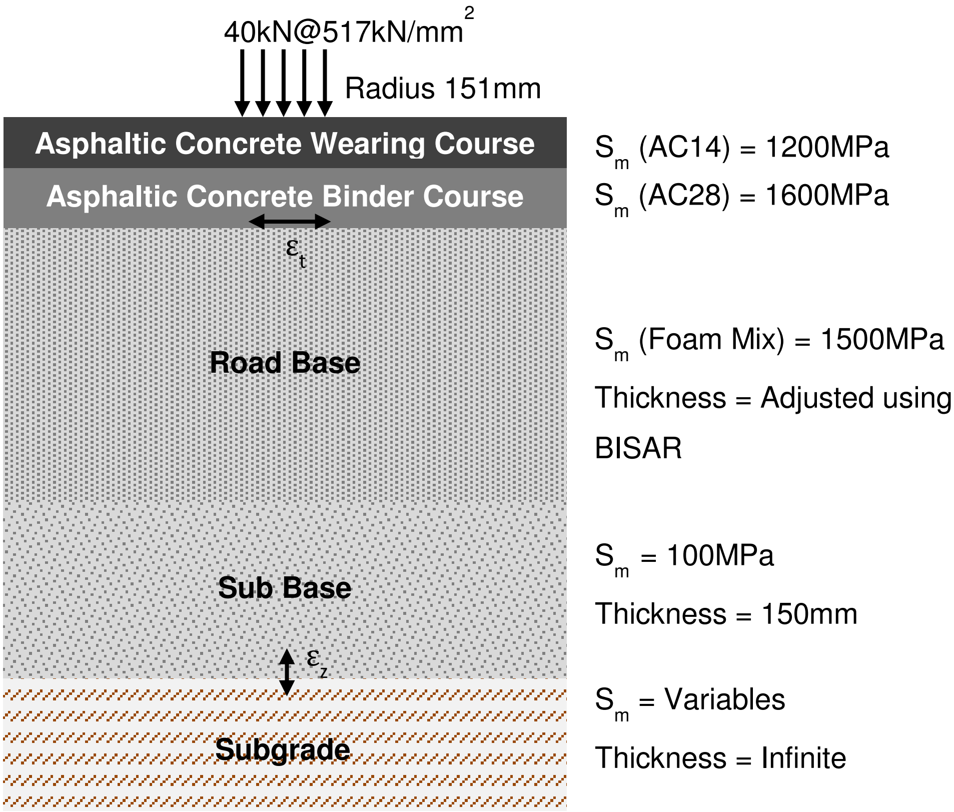


Figure 14.1 Typical pavement configuration and critical strains

Research on the fundamental properties of road pavements or full-scale trials have been carried out by many developed countries which has led to the development of pavement design methods, including specific relationships to calculate these critical strains and their allowable limit values. Table 14.1 shows examples of models developed to predict the pavement life in terms of number of standard axles under specific local conditions (Powell et al, 1984 and Asphalt Institute, 1982). These are derived from the relationship between allowable strain and the number of load applications to each failure mode (i.e fatigue cracking or rutting) on conventional pavement structure types. There are several assumptions and limits of applicability of the relationships illustrated in Table 14.1 and neither could be directly applied to the pavement sketched in Figure 14.1.

Table 14.1 Examples of design criteria equations to limit rutting and fatigue strain in pavement design

Design Criteria Equation	Reference
Rutting: $\varepsilon_z = A\left(\frac{1}{N}\right)^b$ ε_z = Vertical compressive strain at the top of subgrade N = Number of standard wheel loads in the design life A and b = constants (vary for different versions of equation)	TRRL 1132 (Powell et al, 1984)
Fatigue: $\varepsilon_t = C\left(\frac{1}{N}\right)^d$ ε_t = Horizontal tensile strain at the bottom of asphalt C and d = constant (depend on asphalt characteristics)	
Rutting: $N_f = 1.365 \times 10^{-9}(\varepsilon_z)^{-4.477}$ N_f = Number of load applications to failure ε_z = Vertical compressive strain at the top of subgrade	Asphalt Institute (Asphalt Institute, 1982)
Fatigue: $N_f = 0.0796(\varepsilon_t)^{-3.291}(E)^{-0.854}$ ε_t = Horizontal tensile strain at the bottom of asphalt E = Elastic Modulus of asphalt concrete	

14.3 METHODOLOGY

In a mechanistic design approach, the structural properties of the pavement materials are the primary data input, expressed in terms of stiffness modulus and Poisson's ratio. Therefore it is paramount to know realistic stiffness values of different asphalt mixes and granular materials for the pavement design. Known limiting strain values for both fatigue and rutting criteria are also essentials to design the pavement thickness in such a way that the potential failures would be avoided.

Unfortunately there is no available data on the property of the pavement materials in Brunei Darussalam to obtain their own limit fatigue and rutting strain criteria under local conditions. For the purpose of this analytical study, the new revised Malaysian pavement structural design manual (JKR Malaysia, 2013) was selected as a guideline. It should be noted that the manual has limitations in terms of loading conditions and the materials particularly aggregate type i.e. granite which is conventional in Malaysia. However, the types of pavement materials stated in the manual have similarities with the ones currently being used in Brunei Darussalam such as the asphaltic concrete AC 14, AC 28 and the manual takes into account the similar climatic conditions.

The aim of this analytical study is to develop a simple relationship between the foam mix layer thicknesses (applied as road base), the expected traffic in equivalent single axle loads (in millions of ESALs) and subgrade stiffness (in MPa). In order to conduct the study, allowable limiting strain in the pavement structure has to be assumed to avoid the potential for significant pavement distress. This assumption was obtained from a back-calculated analysis of the predesigned pavement structures included in the Malaysian design manual.

A layered elastic program known as Bitumen Stress Analysis for Roads (BISAR) was used to calculate the stress induced by a specified wheel loading to identify the critical strains in the pavement structure. The program is, however, based on linear elastic theory in which the layers in the pavement structure are considered to be homogenous and isotropic. Without considerable further study, the consequences of these computational limitations cannot be fully determined. Nevertheless the effects should be minimised since the use is intended to evaluate comparable pavement structures and not to provide an absolute design, as discussed below.

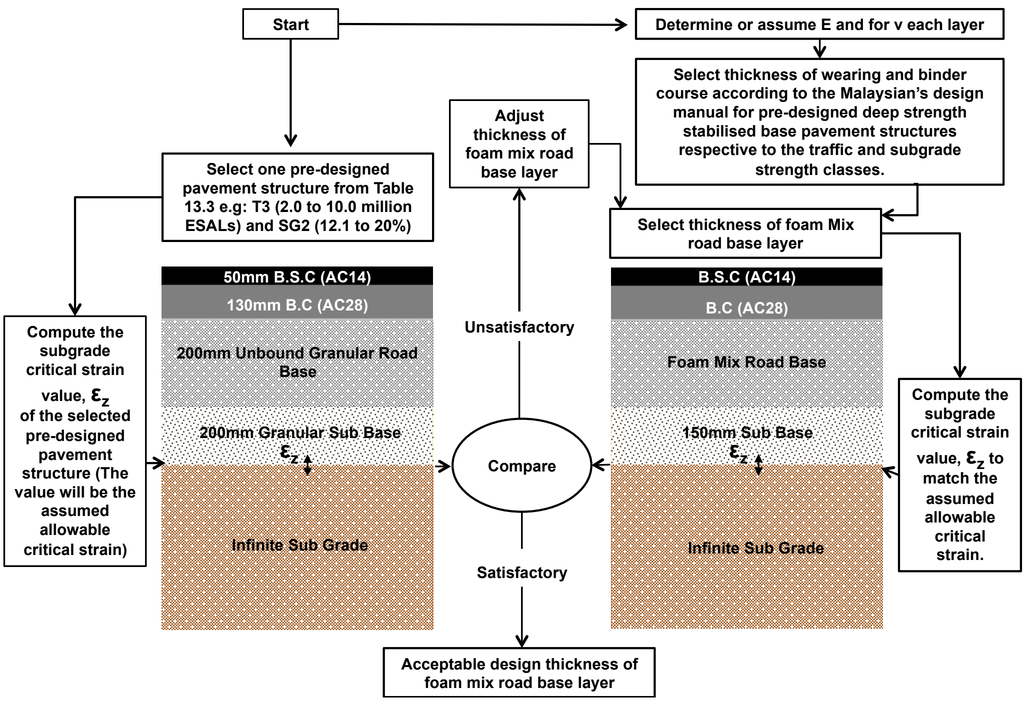


Figure 14.2 Analytical design study flow diagram (Exemplar pavement structures shown for T3 and SG2, see Table 14.3)

The tensile strain calculations (at the bottom of a conventional hot asphalt binder course layer) may not be applicable to a pavement that incorporates a road base formed of foam mix, which could be considered as some pseudo-high quality granular material in which the stiffness modulus value was similar to that of the hot asphalt surface layers. The conventional approach would effectively have analysed the foam mix road base as a hot asphalt mix resulting in low tensile strain values at the bottom of the layer. Furthermore if all the hot and foam mix asphalt layers are treated as a very thick asphalt, then the tensile strain at the bottom of the foam mix layer cannot be expected to relate closely to actual fatigue behaviour of the foam mix. The foam mix is a partially bound material; therefore, there are inevitable cracks in the structure that either affect the fatigue life or make it irrelevant. In this analytical study, the fatigue protection of the conventional asphalt layers was taken into account by obtaining the thickness of the surface layers from the pre-designed deep strength stabilised base pavement structure as mentioned below. Therefore, this analytical study will be based only on the assumed rutting (or compressive strain) criterion.

The pavement analysis performed in this study is summarised in a flowchart diagram, Figure 14.2, and the detailed procedures were as follows:

i. Select nine pre-designed conventional flexible pavement structures, one each for the nine combinations of three subgrade classes (SG2, SG3 and SG4) and three traffic classes (T2, T3 and T4), from the catalogue in the Malaysian standard ATJ5/85 (JKR Malaysia, 2013). The matrix is as shown in Table 14.2 and 14.3.

ii. The stress induced by each of the three traffic classes on those nine pre-designed pavement structures was calculated using BISAR to compute the critical vertical compressive strain on top of the three subgrade classes.

iii. The computed vertical compressive strain, ε_z was assumed to be the allowable rutting strain value, design criterion, to its traffic and subgrade category. In principle, the computed horizontal tensile strain, ε_t, should be assumed to be the allowable value for design in a similar manner. However, for the reasons explained in the previous paragraph reliable values of ε_t could not be computed so an alternative approach was adopted (see step v below).

iv. Using BISAR, the stabilised Brunei Darussalam sandstone foam mix road base layer (that replaced the aggregate layer formed of expensive imported high quality stone) (Figure 14.2) was adjusted until its actual vertical compressive strain reached (or was less than) the assumed allowable compressive strain value (see Table 14.4). This would then provide the minimum thickness of road base layer for the respective traffic and subgrade classes.

v. As an alternate to computations based on ε_t (see iii above), the asphaltic concrete surface thickness is extracted from the pre-designed deep strength stabilised base pavement structure catalogue in the Malaysian design manual respective to the traffic and subgrade class.

vi. Thus, when the minimum foam mix thickness satisfied the criterion (assumed allowable strain value), the related pavement configuration was considered as a valid design.

Further explanation for these steps is provided in the following pages.

Table 14.2 Traffic and Sub-grade classes extracted from the Malaysian's manual ATJ5/85 (JKR Malaysia, 2013)

Traffic Categories (ESAL=80kN)		**Classes of Sub-grade Strength (Based on CBR) used as elastic modulus input values**		
Traffic Category	Design Traffic (ESAL x 10^{-6})	Sub grade Category	CBR %	Elastic Modulus (MPa)
T2	1.1 to 2.0	SG2	12.1 to 20	120
T3	2.1 to 10.0	SG3	20.1 to 30.0	140
T4	10.1 to 30.0	SG4	>30.0	180

Table 14.3 Nine pre-designed conventional flexible pavement structures extracted from the Malaysian design manual ATJ5/85 (JKR Malaysia, 2013).

Traffic Category	**Subgrade Category**	**Thickness (mm)**			
		Asphaltic Concrete AC14 (BSC)	Coarse Bituminous Mix AC 28 (BC)	Crushed Aggregate Road Base (CAB)	Sub-base Course (GSB)
T2	SG2	140	-	200	100
	SG3	120	-	200	100
	SG4	100	-	200	150
T3	SG2	50	130	200	200
	SG3	50	130	200	150
	SG4	50	130	200	100
T4	SG2	50	150	200	200
	SG3	50	150	200	150
	SG4	50	150	200	100

14.4 GENERAL INPUTS AND ASSUMPTIONS

BISAR requires a number of inputs and assumptions to adequately characterise a pavement structure and to compute its response to loading on selected pavement configurations. These inputs are simplified in Figure 14.1 and the detailed descriptions are as follows:

14.4.1 Loading Condition

The total number of standard axle loads is a parameter used for design of a new pavement. It is commonly quantified as an equivalent number of single axle dual wheels, each imposing a load of 80kN (TRL, 1993 and JKR Malaysia, 2013). For easy analysis, a single wheel loading system was used to make the critical horizontal position exactly under the wheel load leading to a slightly more conservative design. Given that the parameters being designed were comparative to reference pavements, this simplification should be insignificant. Therefore, the total force by one wheel applied on the pavement surface was 40kN and it is used as the load input in BISAR. The tyre pressure is assumed to be uniformly distributed over a circular area, 517kN/m^2 (Robinson and Thages, 2004). This gives a radius of the contact pressure of the load about 151mm. From this loading condition data, the BISAR program calculates the area of contact, the force applied and hence, the stresses and strains in selected positions.

14.4.3 Material Properties

The properties of the pavement materials were carefully selected by the author to match the common pavement materials being used in Brunei Darussalam as described below. A Poisson's ratio of 0.35 was assumed for all layers.

Asphaltic Concrete

In the conventional pavement structure in Brunei Darussalam, the asphaltic concrete layers are mainly used to form wearing and binder courses. The wearing course is the top surface layer mainly made up of nominal aggregate sizes of 14mm (labelled as AC14). The lower surface layer, binder course, has nominal aggregate sizes of 28mm (labelled AC28) in accordance to GS1 (CPRU, 1998). The AC14 and AC28 were referred as bituminous surface course (B.S.C.) and binder course (B.C.) in the Malaysian pavement design manual. Bitumen penetration grade 80/100 has been the conventional type being used for forming the asphaltic concrete layers. Therefore, the stiffness moduli input for AC14 and AC28, as shown in Table 14.4, were 1200MPa and 1600MPa respectively at a temperature of 35°C as defined in the Malaysian design manual ATJ5/85 (JKR Malaysia, 2013) for the analysis of the pavement design.

Table 14.4 Stiffness modulus parameters of the Asphaltic Concrete materials taken from the Malaysian design manual ATJ5/85 (JKR Malaysia, 2013)

Pavement Layer	**Material Description**	**Elastic Modulus (MPa)**
Wearing Course (refer as Bituminous Surface Course, B.S.C.)	AC 14	1200
Binder Course (refer as B.C.)	AC 28	1600

The asphaltic concrete is mainly designed to act as surface layer in conventional Brunei Darussalam pavement structures and is made up to a thickness of 100mm comprising wearing and binder course, normally 40mm and 60mm respectively. It is considered to be rather thin considering the need to withstand heavy traffic volumes that could potentially induce excessive tensile strains at the bottom of the asphalt layer leading to the formation of cracks. Due to the limitation in the available fatigue data, Table 14.5 presents the thickness requirements for conventional asphaltic concrete surface layers, extracted from the Malaysian design manual, for stabilised base pavement structures respective to the traffic and subgrade classes. Such stabilised base materials seem closest to the cement-treated foam mixes developed in the current study.

Table14.5 Thicknesses of B.S.C. and B.C. extracted from the pre-designed deep strength stabilised base (JKR Malaysia, 2013)

Subgrade Class	**Pavement Layer**	**Traffic Class**			
		T2	**T3**	**T4**	**T5**
SG2	B.S.C.	120	50	50	50
	B.C.	-	100	150	160
SG3	B.S.C.	100	50	50	50
	B.C.	-	100	140	140
SG4	B.S.C.	100	50	50	50
	B.C.	-	100	130	140

Road base

Two types of road base materials are considered in this analytical study for comparison as described below:

Unbound Granular Road Base

Wet mix crushed aggregates were selected for the unbound granular base layer for back calculation analysis of the Malaysian conventional flexible pavement structure. The stiffness modulus was 350MPa as stated in the manual.

Foam Mix Road Base

The foam mix investigated in this research study for use as road base material to substitute the unbound imported crushed good quality aggregates is the one treated with 1% cement content, 4FB1C. Its stiffness modulus is obtained from the durability test in Chapter 11, 1500MPa. This value was chosen since it was the stiffness modulus retained by the foam mix after it was subjected to two cycles of dry and wet conditions (Figure 11.5).

Granular Sub-base

The granular sub-base materials may consist of crushed stone or gravel of low stiffness property. It acts as a working platform above the subgrade. Therefore, a stiffness modulus of 100MPa was chosen and the thickness of 150mm was selected for the sub-base layer to reflect the practicality of site application (U.K. Highways Agency, 2009).

Subgrade

Weak ground is very common in Brunei Darussalam and subgrade mainly comprises imported fill or stabilised materials to achieve certain CBR strength preferably more than 10%. In this analytical study, subgrade stiffness was the main variable parameter. Its thickness was assumed to extend to infinity at a relatively weak to good condition as shown in Table 14.2. Subgrade stiffness, E, has been determined by converting its CBR strength using equation (Powell et al, 1984) as below:

$E = 17.6(CBR)^{0.64}$ Equation 14.1

where CBR is expressed as a percentage and ε_z is estimated in Megapascals (MPa).

14.5 ANALYSIS RESULTS AND DISCUSSION

Table 14.6 presents the BISAR output of compressive strain values, ε_z, for the predesigned conventional pavement structures shown in Table 14.3, calculated at the top of the subgrade. These values were assumed to be the allowable limit compressive strains against rutting.

The table also presents the minimum thickness results for the selected foam mix modelled as a road base layer. The thickness of the foam mix layers was adjusted so as to give an "Actual compressive strain" value (see Table 14.6) no greater than the assumed allowable limit strain. Hence a minimum thickness requirement of foam mix road base could be indicated respective to the traffic and subgrade class.

Table 14.6 Computed compressive strain (Assumed allowable rutting strain in this study) for the nine pre-designed conventional flexible pavement structures from Table 14.3 and the foam mix road base thickness with its actual compressive strain

Traffic Category	Computed Values	Subgrade Category			
		SG1	*SG2*	*SG3*	*SG4*
T2	Compressive Strain (Assumed allowable rutting strain 10^{-6})	446.7	341.4	400.2	371.8
	Actual Compressive Strain ($x10^{-6}$)	423.6	325.5	389.1	365.3
	Foam Mix Road Base Thickness (mm)	160	160	110	90
T3	Compressive Strain (Assumed allowable rutting strain 10^{-6})	313.1	238.6	244.0	235.6
	Actual Compressive Strain ($x10^{-6}$)	312.3	231.4	239.1	230.8
	Foam Mix Road Base Thickness (mm)	220	230	200	170
T4	Compressive Strain (Assumed allowable rutting strain 10^{-6})	N/A	217.9	221.9	213.5
	Actual Compressive Strain ($x10^{-6}$)	N/A	212.1	218	209.2
	Foam Mix Road Base Thickness (mm)	N/A	250	220	190
T5	Compressive Strain (Assumed allowable rutting strain 10^{-6})	N/A	217.9	221.9	213.5
	Actual Compressive Strain ($x10^{-6}$)	N/A	212.1	218	209.2
	Foam Mix Road Base Thickness (mm)	N/A	250	220	190

The relationship between the foam mix road base thicknesses over increasing subgrade stiffness can be seen in Figure 14.3. Generally as the subgrade stiffness improves, there is a reduction in the minimum thickness requirement of the foam mix road base. At higher traffic classes, T4 and T5, the subgrade has to be stabilised to reach a minimum of 120MPa (JKR Malaysia, 2013). For strong subgrade SG4 (180MPa), the foam mix road base layer requires a minimum of 100mm to withstand light traffic class T2 (about 1 - 2millions ESALs) while a minimum thickness of 120mm is required to withstand heavy traffic class T4 (up to 30millions ESALs).

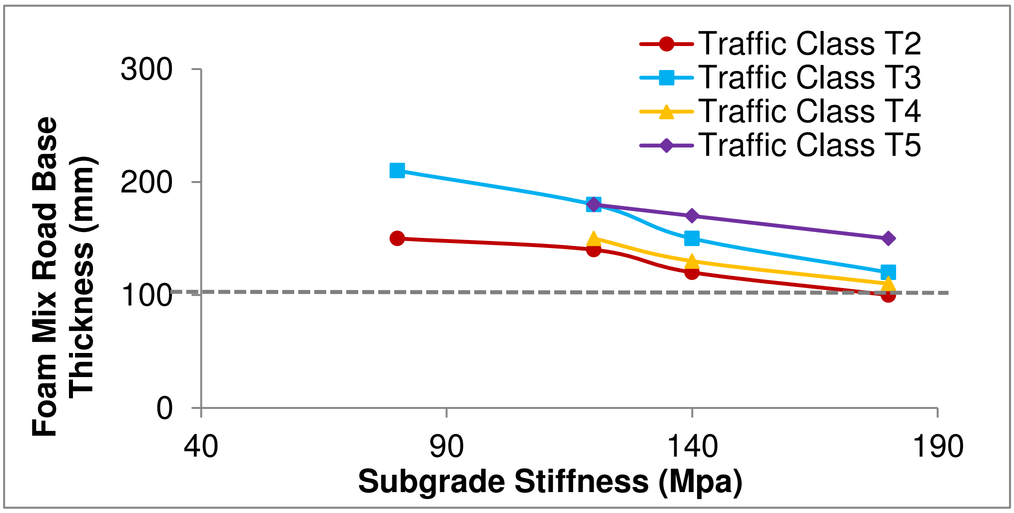


Figure 14.3 Relationship of foam mix road base thickness and subgrade stiffness

It can be observed that there is a significant reduction in the required thickness of foam mix road base from traffic class T3 to T4. The reason can be illustrated in the subsequent figures (Figure 14.4, 14.5, 14.6 and 14.7) which show the relationship between the foam mix road base and asphaltic concrete surface layer thicknesses with increasing expected traffic volumes.

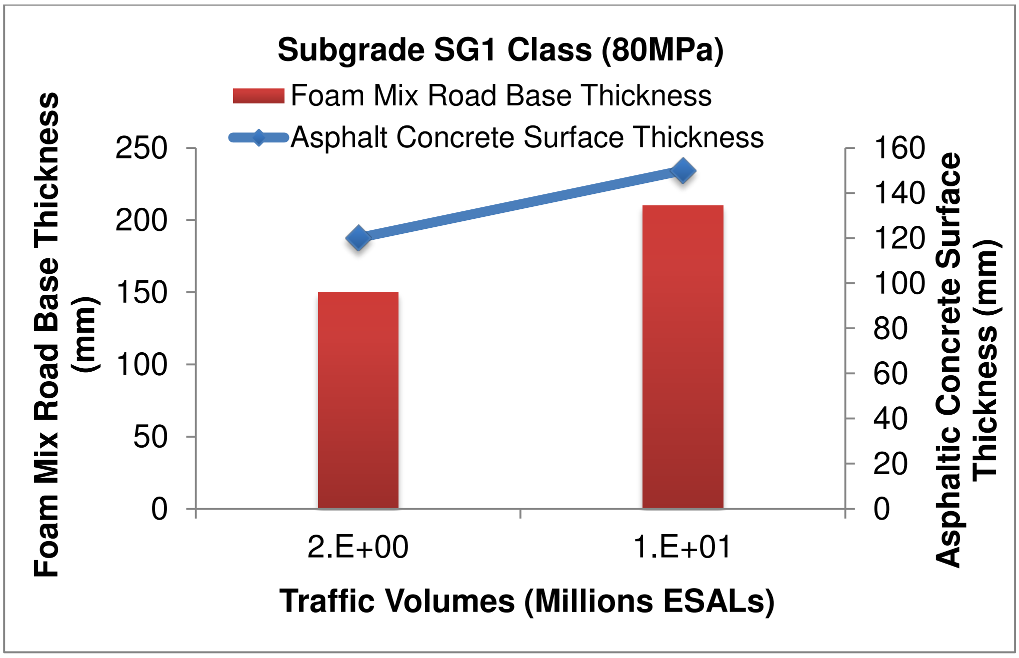


Figure 14.4 Relationship between the effective foam mix road base and asphaltic concrete thickness with increasing traffic volumes

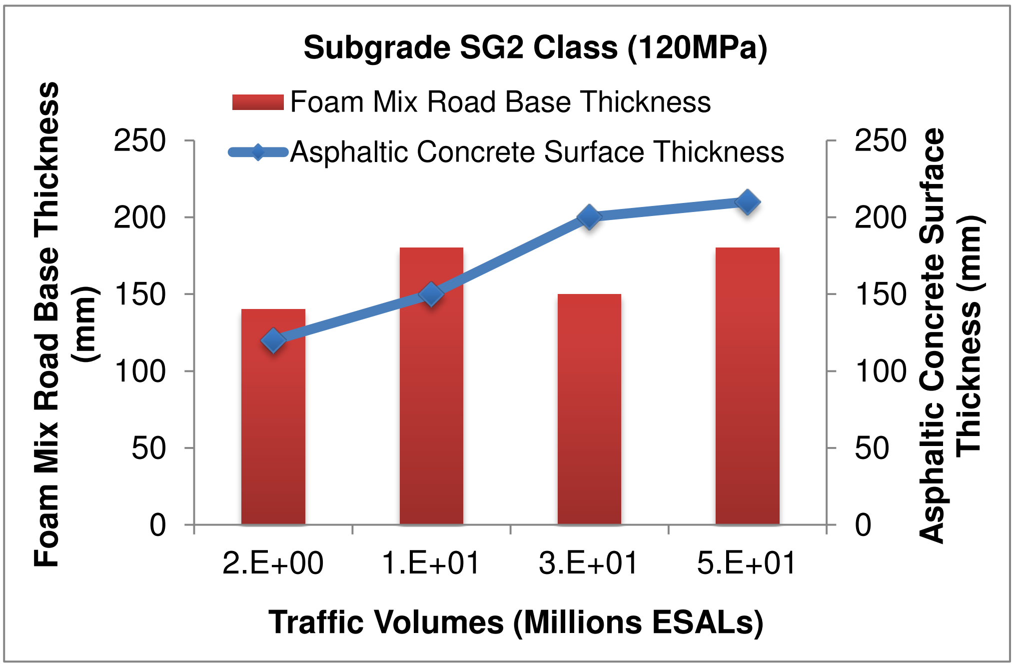


Figure 14.5 Relationship between the effective foam mix road base and asphaltic concrete thickness with increasing traffic volumes

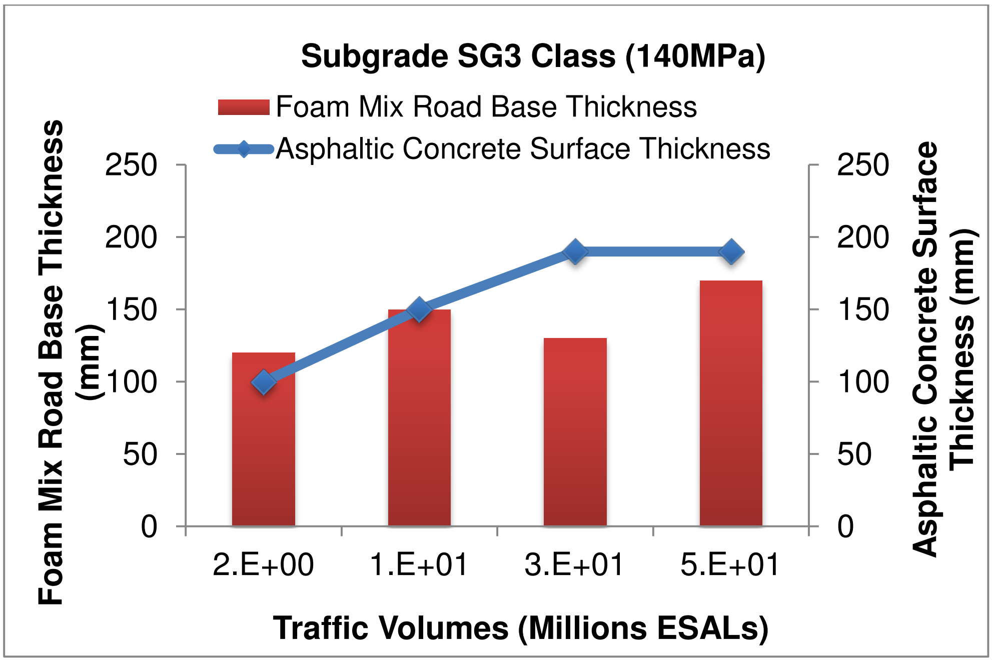


Figure 14.6 Relationship between the effective foam mix road base and asphaltic concrete thickness with increasing traffic volumes

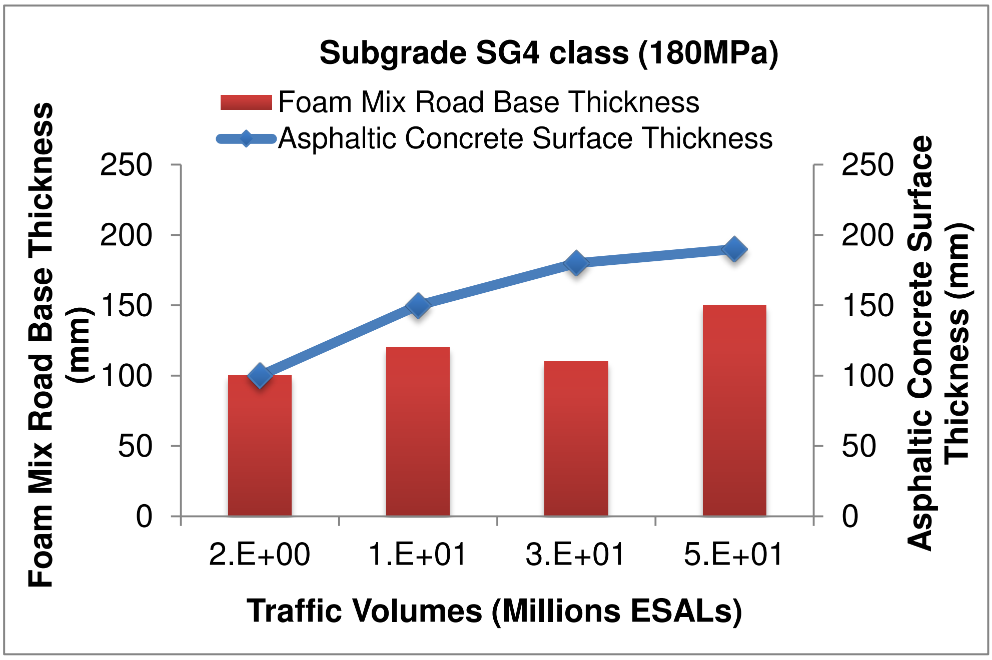


Figure 14.7 Relationship between the effective foam mix road base and asphaltic concrete thickness with increasing traffic volumes

The expected traffic and foam mix road base thickness relationship for each subgrade class is shown in Figure 14.4, 14.5, 14.6 and 14.7. Subgrade class SG1 (80MPa) (see Figure 14.4) could only accommodate low traffic volumes, from traffic class T2 up to T3 (10millions ESALs) for which the foam mix road base thickness increases from 150mm to 210mm. The same trends could also be observed on other subgrade with strengths more than 80MPa as shown on Figure 14.5, 14.6 and 14.7. For subgrade stiffness of 180MPa (SG4), as the expected traffic increased from T2 to T3, the foam mix road base layer thickness increased from 100mm to 120mm.

In contrast, when the expected traffic volume increases to 30millions ESALs (T4), the thickness of foam mix road base decreases. For this increase in traffic the increase in thickness of the asphaltic concrete surface layers more than compensates for any decrease in the foam mix road base thickness. If step v in Section 14.3 is a reasonable one (and this cannot be completely confirmed), then it appears that the T4 pavement may be experiencing the potential of fatigue cracking as the main failure mechanism. This relationship can be seen in Figure 14.5, 14.6 and 14.7 on subgrade stiffnesss of 120MPa, 140MPa and 180MPa respectively.

On the other hand, there is no significant increase in the thickness of the asphaltic concrete surface layer as the expected traffic escalates to about 50million ESALS (T5). Instead there is a need to increase the minimum thickness of the foam mix road base layer. For this increase in traffic reducing rutting may be the controlling mechanism. The same trends can be observed for all other subgrade classes more than SG1.

For very heavy traffic of 50 x 10^6 ESALs (T5), a minimum road base thickness of 180mm with foam mix of 1500MPa is required to be constructed on a subgrade class SG2. For high subgrade stiffness, SG4, a minimum thickness of 150mm only for foam mix road base is required as well as thinner surfacing.

From this analytical study, although the results are hypothetical, they suggest that the foam mix stabilised road base indicated as providing the best protection to the subgrade will have a minimum thickness less than that required for unbound road base. However, the results can only be used with caution because the stiffness modulus may change after applications of repeated loads and environmental factors that could cause deterioration leading to low stiffness modulus particularly in areas of high moisture subgrade (Chen et al, 2011). On the basis of their study, the limiting foam mix base modulus with 50mm surface thickness was 1000MPa to reduce cracking and moisture sensitivity. Therefore, it is important to note that the laboratory stiffness modulus values may be higher than the ones applied in the field. For comparison, the stiffness modulus of bitumen stabilised materials stated in the Malaysian design guidelines is 1200MPa (JKR Malaysia, 2013).

14.6 SUMMARY

This chapter has focussed on the analytical pavement design of a pavement structure with foam mix modelled as a road base layer to predict pavement performance under loading. Since Brunei Darussalam's pavement materials have never been tested to determine the stiffness modulus values therefore, the material properties for the asphaltic concrete and base layers were chosen from the Malaysian structural pavement design manual. This is due to the same types of hot asphalt mixes, AC14 and AC28, and also the conventional bitumen type 80/100 being used in that standard as in

Brunei Darussalam. Furthermore, both countries have similar climatic conditions.

The limiting vertical subgrade strain value for the rutting failure criterion was back-calculated from the pre-designed conventional pavement structure extracted from the Malaysian design manual. The foam mix road base thickness was adjusted so that the strain induced by the loading stress is still within the limiting strain value.

Since the conventional asphaltic concrete layer surfacing is normally designed to be 100mm thick, therefore the thickness of the asphaltic concrete for pavements containing foam mix was extracted from the predesigned deep strength stabilised pavement structures to reduce the potential risk of fatigue failure which could not readily be estimated by other means and might not even be relevant.

Generally foam mix road base layers reduce in thickness as the subgrade stiffness increases. At high traffic volumes, the foam mix road base reduced due to the increase in the thickness of the asphaltic concrete surface layers required to protect against the fatigue cracking. In contrast, as the traffic volumes increase to more than 10million ESALs, the thickness of foam mix reduced due to the increase in asphaltic concrete surface layers. The foam mix road base thickness increases, however, as the expected traffic volumes reach 50million ESALs.

The conventional thickness of asphaltic concrete surface layer, 100mm, in Brunei Darussalam is considered to be too thin to withstand the heavier traffic loadings. Hence, it was decided to adjust the asphaltic concrete thickness based on the thicknesses extracted from the Malaysian design manual, for pre-designed deep strength stabilised base pavement structure giving surfacing thicknesses of 190-210mm for the heaviest traffic.

CHAPTER 15 CONCLUSIONS, IMPLICATIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

15.1 INTRODUCTION

This chapter makes conclusions regarding the mix design process, laboratory curing, the influence of various additives and moisture levels as well as the durability of the foam mixes.

It follows with implications of using foamed bitumen as a treatment on sandstone aggregates for field application, that highlight the issue of moisture levels, aggregate gradation, short-term stiffness and durability of materials in the alternate dry-wet conditions.

The last part is the recommendations for future research that would extend this research study to understand the interaction of sandstones and foamed bitumen as well as the co-treatment agent and which address routine application for roads in Brunei Darussalam.

15.2 CONCLUSIONS

15.2.1 Mechanical Properties

Stiffness Response

Utilising marginal aggregates such as sandstones can impair the good performance of bituminous foam mixes particularly in a wet environment. A small amount of cement greatly enhances the properties of bituminous foam mixes such as stiffness, rutting and moisture resistance.

The presence of high cement content causes a dramatic increase in the foam mix's stiffness at low foamed bitumen contents. It then exhibits concrete-like characteristics rather than bituminous characteristics. In contrast, the bituminous characteristics still dominate the foam mix when the mix has high foamed bitumen content.

Bituminous foam mixes are partially coated materials, which have different components such as unbound coarse aggregate skeleton, uncoated fines and foamed mastic. When treated with cement, cementitious bonds will be an additional component in the mixtures. It can be observed that the behavioural impact of any component on the stiffness modulus can be masked by the impact of a more dominant element in the mixes, both in dry and wet conditions. In this case, a potential reason for the contrasting behaviour of the two cement-treated foam mixes could be the relatively cementitious dominant behaviour of the foam mix with 2% cement contents as compared to that of the ones with less cement, 1%, which exhibited a more bituminous dominant response.

When mixing moisture is low (equal to 60% of OMC), the stiffness could be gained by the presence of the matrix suction and aggregate interlock. But the mixture could produce high variability particularly for the cement-treated foam mixes. At an MMC of 70% of OMC, the standalone bituminous foam mix (4FB) performs at its optimum, thus its bituminous characteristics become dominant so that it could only allow 1% cement addition to enhance its stiffness properties but did not increase its performance much with the addition of more cement. The slight increase in MMC to 80% of OMC has an increased effect on the stiffness modulus value of foam mixes with 2% cement due to presence of extra amount of water to react with the extra cement, adding cementitious bonds.

In this study, the MMC can influence the mix design of the foam mix due to the variation in stiffness responses. Hence, it is important to indicate a rational range of MMC in order to obtain a consistent mix that would have a suitable stiffness response.

However, it is not sufficient, when determining MMC for mixing and compaction of foam mix, particularly when cement is added, to use an OMC (of virgin aggregates) criterion alone. The stiffness modulus of foam mix with or without cement has to be included because the response can vary over a range of MMCs.

Rutting

Based on the findings, the foam mix with 1% cement content is indicated to have the best rutting resistance of all the foam mixes. Its good performance in the wheel tracking test indicated that the addition of cement enhanced the high temperature stability (60°C) of the foam mixes.

In the early serviceable life of a pavement, densification seems to continue under the tracking of the wheel load for most of the solely foam mixes, whereas, the cement-treated foam mixes seem to exhibit non-increasing deflection values following early-life rutting.

Improper mixing could lead to a bad distribution of foamed bitumen binder that might cause the localisation of soft foamed mastic along the wheel track resulting in rapid deformation.

Coir Fibres as Reinforcement Agent

From the previous preliminary laboratory results, coir fibre is concluded to not improve the performance of foam mix and the mix is very sensitive to wet environment. It has a poor resistant property to permanent deformation.

15.2.2 Microstructure Behaviour of Foam Mixes

In foam mix with high foamed bitumen content, 4%, the small mastic blobs have a higher frequency than the ones with low foamed bitumen contents, 3%, leading to mix having stronger bituminous characteristics. Therefore, the 4% foam mix only allows small amount of additive such as cement to fill some voids in the mix whereas the foam mix with 3% foamed bitumen content would allow many more opportunities for the cement to occupy the spaces delivering a weak concrete-like structure.

At the same cement contents, the bonding in a foam mix containing low foamed bitumen content is dominated by the hydrated cement products whereas the foam mix containing high foamed bitumen content exhibits a performance dominated by the bituminous mastic.

The addition of coir fibres generates a porous microstructure. The fibres tend to be detached from the mastic leaving concave marks indicating a poor bonding between the mastic and the fibres.

Cement hydrated products formed in the cement-treated foam mixes results in a denser structure that is less susceptible to water.

15.2.3 Climatic Factors Affecting the Engineering Properties

Humid Curing Study

The bituminous foam mix alone tends to be weak during its early life and the utilisation of low quality aggregate, such as sandstones, further weakens the foam mix. From the laboratory humid curing study, the stiffness of foam only mix reached no higher than about 1000MPa after more than a week under humid curing. The low stiffness modulus indicates that it has a high risk of moisture damage particularly in the event of rainfall during the curing period.

In this case, the addition of cement as the co-treatment agent is vital to assist and overcome the shortcomings of standalone foam mixes. The addition of 1% cement has a marked beneficial effect on the humid curing rate and as well as on the ultimate stiffness of foam mixes, becoming higher than the foam only mixes although the mix might experience micro-cracks over time. With the high stiffness modulus achieved over a short period of time, it would make it less susceptible to water and hence it can allow for the early road opening to traffic. Furthermore, the cement-treated foam mix could retain its stiffness modulus greater or equal to 2000MPa in wet conditions.

One of the issues that emerges from the study is the short-term stiffness. The rapid development of the cement-treated foam mix at early age, has demonstrated a mix that can be suitable for a humid environment. With a small amount of cement, 1%, added, the stiffness gained about 3000MPa by the 4th day of the humid curing period. This finding indicates that the rapid curing period of such a mix could allow for early trafficking. It also has a low risk of moisture damage because its high stiffness modulus would sustain a slight reduction in wet conditions as explained in the Chapter 6.

The curing of standalone foam mixes was simply achieved by water evaporation, a process that the inclusion of cement did not seem to influence significantly. For the cement-treated foam mix, the cement used up water, that might have been lost to the environment after about 2 months, in the cement hydration process.

It is interesting to note that the cement-treated foam mix did not remain at its maximum stiffness modulus value of 8000MPa for a long period, eventually decreasing after approximately 8 months. This was probably caused by the repeated ITSM tests inducing micro-cracks in the mix, although cement paste carbonation could be an alternative reason. It is hard to be certain of the main cause because when comparing it with the foam only mix, the foam only mix eventually reached a plateau modulus, giving no comparable loss of stiffness for a material that cannot experience carbonation and that is intrinsically more flexible, with bituminous bonding throughout the mix.

Dry-Wet Cycles

It is clear that cement additive resulted in, by far, the best of all mixes by reducing the moisture susceptibility of the foam mixtures. It is surmised that, in having the foam mixes submerged under water, the residual cement re-activates the hydration reaction to form crystals filling more voids and repairing any damaged bonds. The worst performance of foam mixes was obtained with the pre-blended wet fix bitumen, particularly when the aggregates were wet.

The durability of foam mix is found to be low after the alternating dry and wet conditioning. The study has shown that the addition of cement (as little as 1% by mass of dried aggregates) can produce a durable pavement material compared with the performance of mixes containing other additives such as hydrated lime, Wet Fix and coir fibres. Thus, the foam mix containing the optimum foamed bitumen content, 4%, with 1% cement would be the most suitable material to be implemented in a wet tropical climatic region, such as that experienced in Brunei Darussalam.

15.3 ANALYTICAL PAVEMENT DESIGN

Analytical study of pavement design has been conducted to study the theoretical design life of a selected pavement configuration in which foam mix was modelled as road base layer.

In outline, a proposed thickness of foam mix for a variety of support conditions and traffic loading has been developed providing a means of replacing large volumes of expensive aggregate import into Brunei Darussalam with foam mix using far more economic, locally available, stones.

Although this analytical design study is based on limiting the compressive strain on the top of subgrade, it should be noted that the fatigue performance of asphaltic concrete or bound layer is of paramount importance in pavement design. The most cost effective pavement design option must take into account both the stiffness and fatigue performances. Nonetheless, the thickness values of the asphaltic concrete in the selected pavement configuration in this study were extracted from the Malaysian design manual as a function of the traffic and subgrade stiffness.

However, due to the lack of realism in the laboratory test set up, procedures and control conditions, the design requires to be calibrated against observed pavement performance.

15.4 IMPLICATIONS FOR FIELD APPLICATIONS

Generally the stiffness modulus of foam mix tends to be weak during its early life but develops gradually over time until it reaches the ultimate stiffness value. However, in the field, environmental factors such as moisture and humidity level vary and further prevent the foam mixes from reaching their ultimate stiffness value in a short time.

Adding more cement can enhance the stiffness modulus of foam mix but the disadvantage is that the foam mixes can become brittle. However, this observation is dependent on the foamed bitumen content and moisture level. When low foamed bitumen contents are used to produce the foam mix, the foam mix provides more spaces or voids for the hydrated cement products to fill resulting in more concrete-like materials as compared to the mix with high foamed bitumen content which maintains a more flexible bituminous-like characteristic.

The addition of higher cement content in the foam mix is prone to produce a less workable mix and, hence, a heterogeneous mixture particularly when mixing at low moisture levels, due to the fast drying effect. The cementitious bonds may dominate the resulting mix structure causing it to become more rigid. However, at the moisture level at which the non-cement treated foam mixes perform optimally, the higher cement content (about 2% by mass of dried aggregates) does not significantly enhance the stiffness above that achieved by the addition of a smaller amount of cement.

In the field, variations in the moisture levels are likely to happen due to environmental factors such as temperature, wind and humidity. Therefore, it would be hard to control the moisture level to its desirable value. On a hot sunny day, a high temperature can increase the evaporation process of mixtures on the surface, hence it speeds up the drying effect. On the other hand, the high humidity environment would slow down the evaporation activity within the mixture particularly under the surface. The variations would escalate with the addition of cement as it generates hydration processes that react with water in the mix. It would result in further increases in the non-uniformity of moisture level across the mix. This situation would make it difficult to control compaction across the mix in order to achieve an optimum and homogenous mix. Improper compaction could cause pre-mature damage such as ravelling and rutting. Therefore, it is crucial to verify the moisture level measured in the laboratory against the moisture measured in the field condition. The timing is also important such that upon mixing with water, compaction must follow immediately so as to minimise loss of designed mixing water.

The design of aggregate gradation has to take into account the possible degradation of aggregate particles due to the compaction process particularly when utilising the low quality aggregate types. More fines may require a higher amount of foamed bitumen to ensure an optimum foam mix design.

When cement is added, either as a replacement of fines or an additional mass, it has to be taken into account in the mix design evaluation. This study emphasised that when the cement content is additional to, rather than replacing, the fines, added more cement, could be more likely to produce a heterogeneous mixtures, particularly when the moisture level is low.

15.5 RECOMMENDATIONS FOR FUTURE RESEARCH

The surface chemistry between the foamed bitumen and mineral aggregates, particularly siliceous aggregates like sandstone, needs to be further investigated. It will help to understand the failure mechanism at the micro-scale when additives such as cement, hydrated lime and also pre-blended base bitumen with WF are used. Perhaps it will reveal alternative ways to improve the performance by changing the mix design such as gradation and moisture level, and techniques of mixing, to get the best out of these additives.

It would also be interesting to investigate the durability of foam mixes by changing the pH of the water. Rainwater can be somewhat acidic and this may impair the performance of cement-treated or foam-only mixes in a wet environment. This situation may also apply in the presence of sub-surface water that tends to seep into the pavement materials.

Further research to understand the effect of different components in foam mixes, particularly when cement is added at different mixing moisture levels, is recommended by extending the range of moisture level to 100% of OMC (by mass of dried aggregates). A microscopic study is recommended alongside this so as to see changes in the microstructure of foam mixes at different moisture levels.

Although in this study, the coir fibre is concluded to be unsuitable as the reinforcement agent in foam mix, it would be interesting to look at possible ways to improve the bonding in the foam mix. This might be by increasing the binder or by pre-treated the coir fibres.

As the findings are merely based on laboratory conditions, therefore it is recommended to perform a pilot scale trial using the same materials to find any shortcomings and a way to overcome them. This can assist in the development of detailed pavement design guidelines on the application of foam mix particularly to countries with wet and humid climates.

For improved thickness design of pavements incorporating foam mixes in Brunei Darussalam, it is recommended that the other layers are better characterised and that a fuller study of fatigue cracking of conventional hot mix asphalt surfacing over foam mix layers is fully investigated. A further need for fatigue research is to validate the proposed thickness designs – performance with respect to climate, traffic and subgrade support conditions.

APPENDIX A CALCULATIONS OF MIXING MOISTURE CONTENT (MMC)

Equation A. 1 (Wirtgen, 2004)

$$\boldsymbol{M_{sample} = M_{air-dry} / \left[1 + \left(\frac{W_{air-dry}}{100}\right)\right]}$$

Equation A. 2 (Wirtgen, 2004)

$$\boldsymbol{W_{add} = 1 + \left(0.5W_{omc} - W_{air-dry}\right)}$$

Equation A.3 (Wirtgen, 2004)

$$M_{add} = \frac{W_{add}}{100} \times (M_{sample} + M_{cement})$$

where

W_{add} = water to be added to sample or mixing moisture content (% by mass)

W_{omc} = optimum moisture content (% by mass)

$W_{air\text{-}dry}$ = water in air-dried sample (% by mass)

M_{water} = mass of water (g)

M_{sample}= dry mass of sample (g)

M_{cement}= mass of lime or cement to be added (g)

$$\boldsymbol{W_{add} = 1 + (0.5W_{omc} - W_{air-dry})}$$

$$\boldsymbol{W_{add} = 1 +} \ [\boldsymbol{0.5}(\boldsymbol{6}\%) - \ \boldsymbol{0}\%]$$

$$\boldsymbol{W_{add} = 1 +} \ [\boldsymbol{3}\%]$$

$$\boldsymbol{W_{add} = 4}\%$$

Therefore, $\%MMC = \frac{1}{6} \times 100 = 67\% \ (\sim 70\%)$

Lee (1981) stated that the mixing and compaction moisture content is in the range of 65% to 95%.

Note: that the aggregate samples obtained from the quarry site were oven dried upon its delivery to the laboratory. Therefore, there is no water in the dry mass of aggregate used in the laboratory experiments i.e $W_{air\text{-}dry} = 0$ and $W_{omc} = 6.0\%$.

APPENDIX B SUMMARY OF THE DATA INPUT IN BISAR PROGRAM

Data Input	Values	References
Load (kN)	40	
Radius (mm)	151	
Layer 1 Thickness (mm)	Based on the thickness of the pre-designed ‘Deep Strength – Stabilised Base’	ATJ5/85 (2013)
Stiffness Modulus (MPa)	1200	
Poisson’s Ratio	0.35	
Layer 2 Thickness (mm)	Based on the thickness of the pre-designed ‘Deep Strength – Stabilised Base’	ATJ5/85 (2013)
Stiffness Modulus (MPa)	1600	
Poisson’s Ratio	0.35	
Layer 3 Thickness (mm)	*Calculated using BISAR*	
Stiffness Modulus (MPa)	1500	Chapter 11
Layer 4 Thickness (mm)	150	UK Road Foundation Design
Stiffness Modulus (MPa)	100	ATJ5/85 (2013)
Subgrade Stiffness (MPa)	80,120,140	

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