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THE INFLUENCE OF FOAMED BITUMEN CHARACTERISTICS ON COLD-MIX ASPHALT PROPERTIES

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To

Allah,
Rasulullah,
Dienul Islam,

My Mother – Hj. Niek Djadmini,
In Memory of My Father – Martojo H.P.,
My Family–Anisah, Asmaa, Afifah, Fatin, Zakiy and Farah.

ABSTRACT

The increase of road infrastructure around the world and its impact on the environment requires that serious attention is given to building more sustainable pavement constructions. Foamed asphalt (FA) as an increasingly attractive cold asphalt mixture, is therefore becoming an important subject area for study.

The effect of foaming water content (FWC) on foamed bitumen (FB) characteristics has been identified in terms of maximum expansion ratio (ERm) and half-life (HL). ERm is the ratio between maximum foam volume achieved and the volume of original bitumen, whereas HL is the time that the foam takes to collapse to half of its maximum volume. The value of ERm increases with increasing FWC, while the HL value shows the opposite trend. In general, for FB 160/220 (FB produced using bitumen Pen 160/220), lower bitumen temperature produces higher ERm, whereas for FB 50/70, this trend is reversed. For FB 70/100 the trend was inconsistent.

FB properties (which depend on FWC) are concluded to have a moderate effect on FA performance. Mixing protocol and binder type are found to have a more dominant effect than foam properties. The effect of foam properties is only clearly defined in well mixed specimens, based on stiffness evaluation. The stiffness over various FWC values was found to be affected by a combination of ERm and apparent viscosity of the foam. Three zones of ERm values are proposed, namely a poor zone, a stable zone and an unstable zone. The poor ERm zone is between 3 and 8 (corresponding to wet foam quality, i.e. 52%-87% gas content), the stable zone is between 8 and 25 (for FA using FB 50/70 at 180oC) or 8 and 33 (for FA using FB 70/100 at 180oC), and beyond this ERm value (25 or 33) is the unstable zone. For FA using FB 160/200, no zone categories could be defined since no significant variation in stiffness was observed over the range of ERm values.

Finally, practical guidance for producing an optimised FA mixture has been proposed. This guidance consists of considerations related to mixer type and usage, selection of binder type, bitumen temperature, minimum and maximum application

limits of ERm or FWC, and suggestions are made to obtain the best chance of optimum performance in different climatic regions.

DECLARATION

The research reported in this thesis was conducted at the University of Nottingham, School of Civil Engineering, Nottingham Transportation Engineering Centre (NTEC), between February 2005 and January 2008. I declare that the work is my own and has not been submitted for a degree at another university.

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LIST OF ABBREVIATIONS

AASHTO	The American Association of State Highway and Transportation Officials
AC	Asphalt cement
atm	atmosphere
BC	Bitumen content
bit	Bitumen
BS	British Standard
C	Celcius
CBR	California bearing ratio
CMA	Cold-mix asphalt
cm	Centi metre (10^{-2}m)
cp	Centipoise (1 mPa.s)
CR	Contract Report
CSIR	Council for Scientific and Industrial Research
DBM	Dense Bitumen Macadam
DCP	Dynamic cone penetrometer
DD	Draft Development
DSR	Dynamic Shear Rheometer
ER	Expansion ratio
ERa	Actual expansion ratio
ERm	Maximum expansion ratio
Eq	Equation
FA	Foamed asphalt
FAM	Foamed asphalt mixture
FB	Foamed bitumen
FBC	Foamed bitumen content
FI	Foam Index
FL	Foam life
FWC	Foaming water content
FWD	Falling Weight Deflectometer
GPa	Giga Pascal (10^9 Pascal)
g	gram
HDM	Heavy Duty Macadam
HL	Half-life
HMA	Hot mix asphalt
hr	Hour
HRA	Hot Rolled Asphalt
Hz	Hertz
J	Joule
ITFT	Indirect Tensile Fatigue Test
ITSM	Indirect Tensile Stiffness Modulus
ITS	Indirect tensile strength
kg	Kilo gram (10^3g)
km	Kilo metre (10^3m)
kN	Kilo Newton (10^3N)
kPa	Kilo Pascal (10^3Pa)

LAF	Load area factor
LL	Liquid limit
LVDT	Linear variable differential transformers
Max	Maximum
MC	Moisture content
MDD	Maximum dry density
Mg	Mega gram (10^6 gram)
mJ	Milli Joule (10^{-3}J)
mN	Milli Newton (10^{-3}N)
mm	Milli metre (10^{-3}m)
m	Metre (10^3mm)
MPa	Mega Pascal (10^6 Pascal)
mPa.s	Milli pascal seconds (10^{-3}Pa.s)
MMC	Mixing moisture content
msa	Million standard axle
ms	Milli seconds
N	Newton
NAT	Nottingham Asphalt Tester
NPTF	Nottingham University Pavement Test Facility
NTEC	Nottingham Transportation Engineering Centre
OCC	Optimum compaction and workability content
OFC	Optimum fluid content
OFBC	Optimum foamed bitumen content
OMC	Optimum moisture content
Pa	Pascal
Pa.s	Pascal seconds
Pen	Penetration
PF	Percentage of fines
PG	Penetration grade
PI	Plasticity Index
PL	Plastic limit
PTF	Pavement Test Facility
QH	Quick hydraulic
QVE	Quick viscoelastic
RAP	Reclaimed asphalt pavement
RLAT	Repeated Load Axial Test
rpm	Revolutions per minute
RTFO	Rolling thin film oven
s, sec	second
SABITA	Southern Africa Bitumen and Tar Association
SH	Slow hydraulic
SMA	Stone Mastic Asphalt
SVE	Slow viscoelastic
TR	Technical Report
TRL	Transport Research Laboratory
UCS	Unconfined compressive strength
UK	United Kingdom
µm	Micron

UNCED	United Nations Conference on Environment and Development
VAR	Vacuum asphalt residue
VCL	Virgin Crushed Limestone
VMA	Void in the mix aggregate
VR	Viridis Report
vd	vertical deformation
V/P	Voltage/ pressure
W	Watt
WC	Water content
WC_{reduc}	reduction water content

LIST OF SYMBOLS

A	Area
A1, A2	Area under foam decay curve (seconds)
c	Coefficient or correction factor in FI concept (ERm/Era)
ΔT	Temperature difference
Δh	Horizontal deformation
D	Maximum size of aggregate
d	Diameter of specimen
E	Young's Modulus
E_b	Modulus of binder
E_{mortar}	Modulus of mortar
ε	Strain
ε_x, ε_y and ε_z	Strain in x, y and z direction
ε_t	maximum initial tensile strain
$\overline{\varepsilon_{hx}}$	Average horizontal tensile strain
$\varepsilon_{hx(\max)}$	Maximum horizontal tensile strain at the centre of the specimen
F	force
Fq	Foam quality (%)
Ø'	frictional resistance
$Ø_d$	Volume fraction of dispersed phase
K	Kelvin
k	Thermal conductivity (W/m.^{o}C)
L	Thickness
λ	Ratio of dispersed and continuous viscosity ($\frac{\mu_d}{\mu}$)
Ls	Latent heat of steam (or enthalpy of evaporation) (J/g)
M	Mass (grams)
Mw, Mb, Ms	Mass of water, bitumen, steam respectively (grams)
m	Slope of fatigue line
Mr	Resilient modulus
N	Number of cycles
N_{cr}	$N_{critical}$ (The N value at which the N/vd reaches its highest value
N_f	number of cycles to failure
n	Number of moles (mass/ atomic mass of compound)
ή	Absolute viscosity (Pa.s)
P	Load
Pr	Pressure in atmospheres (atm)
p	Percentage passing
Q	Heat energy (Joule)
Q_{b100}	The amount of transfer heat energy required by hot bitumen to reduce its temperature to 100^{o}C
Qw, Qb, Qs	Heat energy of water, bitumen and steam respectively (Joule)
R	Universal constant (~82.0545) (atm. Litre/mole. Kelvin)
Rc	Flow channel size
r	particle size (mm)
r_B	Bubble size

σ	total stress (kPa)
σ_s	Surface tension (mN/m or mJ/m^2)
σ'	effective stress (kPa)
σ_{vx}	Vertical stress across x-axis (compression)
σ_{hx}	Horisontal stress across x-axis (tension)
σ_{vy}	Vertical stress across y-axis (compression)
σ_{hy}	Horisontal stress across y-axis (tension)
$\sigma_{hx\ (max)}$	Maximum horizontal tensile stress at the centre of the specimen
$\sigma_{vx\ (max)}$	Maximum vertical compressive stress at the centre of the specimen
$\overline{\sigma_{hx}}$	Average horizontal tensile stress
$\overline{\sigma_{vx}}$	Average vertical compressive stress
σ_{fb}	tensile fracture strength of binder
τ	strength
S	Specific heat (or enthalpy) (J/g.^{o}C)
Sw, Sb, Ss	Specific heat of water, bitumen and steam respectively (J/g.^{o}C)
Sm	Stiffness of mixture (MPa)
S_b	Bitumen stiffness (Pa)
T	Temperature (^{o}C)
Tw, Tb, Tf	Temperature of water, bitumen and foam respectively (^{o}C)
t	Thickness of specimen
t	Time (seconds)
ts	Spraying time (seconds)
u	pore water pressure
μ	Viscosity of continuous phase
μ_d	Viscosity of dispersed phase
μ_e	Effective viscosity of dilute emulsion
μ_{ef}	Effective viscosity of foam
μ_l	Viscosity of liquid phase
V	Volume (litres)
v	Poisson's Ratio
Va	Volume of aggregate
Vb	Volume of bitumen
V_f	filler proportion
Vg	gas volume
Vl	Liquid volume
Vv	Volume of void
#	Sieve size

1 INTRODUCTION

The increase of road infrastructure around the world and its impact on the environment requires that serious attention is given to building more sustainable pavement constructions. Sustainability, defined as ‘meeting the needs of the present without compromising the ability of future generations to meet their own needs’ (WCED, 1987), comprises the following four aspects, i.e. better social life, environment protection, prudent use of natural resources and economic growth maintenance (Treleven et al, 2004).

The sustainability issue in pavement construction constitutes a strong incentive towards the use of cold mix asphalt technology worldwide. Foamed asphalt, as an increasingly attractive cold asphalt mixture, is therefore becoming an important subject area for study. It is reported that this mixture has been successfully implemented in many roads across the world especially in cold recycling.

Foamed asphalt mixture (FAM) has considerable advantages. The use of this mixture conserves aggregates and bitumen, decreases energy usage, minimises waste and reduces fuel consumption and greenhouse gas emission. This mixture can therefore significantly reduce the cost of construction. Engineering advantages include the possibility to use a wide variety of aggregates, the binder increases the strength compared to a granular material, exhibiting more flexibility compared to cement treated materials, giving faster strength gains compared to emulsion mixtures and possible early opening to traffic.

Unfortunately, the performance of FAM is still poorly understood. The overall behaviour during the curing process is not well understood; nor are the fundamental properties of stiffness, fatigue and deformation resistance fully defined. Moreover the foamed bitumen characteristics and their effect on mixture properties are also still unclear. It is noted that addressing the lack of understanding of how the binder works in the mixture is crucial for implementation. Foamed asphalt technology generally presents the perception that this type of mixture brings a significant risk due to its complicated behaviour. It is therefore necessary and timely to conduct research into FAM performance and hence to provide an up-to-date evaluation of foamed bitumen binder, its characteristics and the role it plays in mixture performance. It is expected that this research will contribute to unravelling the complications of foamed asphalt behaviour and to allowing this material to achieve the high requirements of a pavement material.

1.1 Background

1.1.1 What is foamed bitumen?

Foamed bitumen can be produced by injecting pressurised air and a small quantity of cold water into a hot bitumen phase in an expansion chamber. Soon after spraying into a special container, the bitumen foam expands rapidly to its maximum volume followed by a rapid collapse process and a slow, asymptotic return to its original bitumen volume.

For example, 500g of hot bitumen injected using 10g of cold water (2% of bitumen mass) normally results in foam with a maximum volume around 15-20 times that of the bitumen. The ratio between maximum foam volume achieved and the volume of original bitumen is termed the maximum expansion ratio (ERm). The ERm value is mainly dependent upon the amount of water added, namely the foaming water content (FWC). ERm increases with higher FWC. After reaching its maximum volume, the foam dissipates rapidly accompanied by steam gas escaping. The time that the foam takes to collapse to half of its maximum volume is called the half life (HL). In the above example, HL would normally be between 20-30 seconds. After a particular time (around 60 seconds), the foam volume reduces very slowly and asymptotically. During this phase, foam bubbles still survive even though the bitumen has become harder.

Regarding the generating process of foamed bitumen, it is supposed that foamed bitumen comprises air, steam gas, liquid bitumen and a little remaining water. When foam is investigated in a measuring cylinder, steam gas is seen clearly forming bubbles which are wrapped by liquid bitumen. The gas bubbles appear to be rising to the surface boundary (looking like water boiling) whilst the liquid bitumen descends during foam dissipation due to the gravity effect. Foam bubbles appear larger when foam volume reduces. It is also understood that bitumen temperature during foaming reduces significantly to around the boiling point of water. Thus, foam properties change in several ways with elapsed time, i.e. temperature drops, gas content reduces, volume decreases, density increases, and bubbles expand & collapses. The bitumen state also changes from liquid to foam, returns back to liquid, and then moves to a viscous and solid condition. Thus, foamed bitumen is an unstable material with complex properties.

During the foaming process, the key properties of bitumen change from bulk properties to surface properties. Molecules of surfactant (primarily contained in asphaltenes) are transported from the bulk of bitumen to the interface (between liquid bitumen and air gas), and form an adsorption layer on the interface (Barinov, 1990).

Referring to Schramm (1994) and Breward (1999), foam structures can be divided into two groups, i.e. wet and dry, depending on the proportion of liquid in the foam. Wet foam is foam with liquid volume fraction typically between 10-20%, whereas dry foam is foam with liquid volume fraction less than 10%. Bubble shapes in wet foam are approximately spherical, while in dry foam, the bubbles are more polyhedral. The bubbles structure significantly affects the foam rheology. Dry foam tends to have higher apparent effective viscosity (Assar and Burley, 1986) and elastic modulus (Weaire and Hutzler, 1999) than wet foam.

1.1.2 How is foamed bitumen used in road construction?

Foamed bitumen, having specific properties as described above, enables the coating of wet aggregates at ambient temperature to form foamed asphalt for road pavement material. This cold-mix asphalt (CMA) material is an alternative to hot mix asphalt (HMA), a ‘traditional’ mixture of aggregate and bitumen which is mixed at hot temperature.

The use of foamed bitumen in road construction using a cold system can be achieved either by ‘in-plant (ex-situ)’ or ‘in-place (in-situ)’ technology. In neither system is it necessary to heat the aggregate materials (either recycled or fresh aggregates) prior to mixing with foamed bitumen. In-plant mixing enables control of input materials and mixing quality and also the material produced can be stored for later use, whereas in-place treatment offers a cost effective and rapid form of road rehabilitation with relatively lower quality than in-plant mixing method.

The in-plant mixed process produces a material known as Foamix. The plant consists of hoppers for aggregate with a conveyor belt feeding into a pugmill. Spray bars are fitted for addition of water. Foamed bitumen is sprayed as the aggregate drops from the conveyor belt and proceeds to mixing in the pugmill to ensure that foam distribution within the mix is homogenous. The Foamix then drops onto a conveyor belt where it can be transported to a loading truck or to a stockpile (Maccarone et al, 1995). Currently, progress in the production technology for Foamix has led to the development of mobile mixing plants which can be located close to site in order to reduce the transport cost of materials. The feed materials may be virgin aggregates, road planings, marginal construction materials or combinations of these (Millar and Nothard, 2004). The produced Foamix material appears as moist particles that consists of coated fine aggregate and partly coated coarse aggregate.

The in-place process, also known as Foamstab, consists of recycling a distressed pavement by milling the road to certain depth (100mm to 300mm) using a heavy duty rotovator. The Foamstab process can be used to recycle distressed asphalt pavement and granular base and/or sub-base layers. The pulverised pavement is then injected with water followed by foamed bitumen sprayed into the recycler’s mixing chamber. The bitumen is continuously supplied to the rotovator from a road tanker and the two vehicles move in tandem along the site. The appearance of the materials after mixing is similar to Foamix. The mixed material can then be levelled, shaped and compacted to obtain a new flexible pavement. The process is carried out in a single-pass operation (Wirtgen, 2004).

Foamed bitumen can also be produced in a small mobile plant under laboratory conditions. Wirtgen WLB 10 foaming plant is specially designed to investigate foamed bitumen characteristics, and to generate FAM as well by attaching a mechanical mixer to the foaming unit. The foamed bitumen produced by this unit is similar to that produced by the foamed bitumen systems mounted on large recycling machines (Wirtgen, 2004).

The role of mixing in the process of generating foamed asphalt material is important since foamed bitumen collapses rapidly in seconds. Foamed bitumen should be produced at the best quality to ensure that the foam disperses as much as possible in the mixture. Aggregate moisture content should be predetermined properly (see Chapter 2 section 2.5.1.5) to ensure foamed bitumen is able to distribute onto the aggregate surface. Mixer capabilities (power, speed and agitator type) should be designed to guarantee the most homogenous foam dispersal in the mixture. During the mixing process, foamed bitumen properties play an important role in helping to produce the optimum end product performance.

FAM has most potential when used as a base course layer and placed between an asphaltic surface and granular layers in a road pavement. The ‘semi-bound’ pattern of aggregate-binder structure of FAM indicates its behaviour is between those of unbound and fully bound materials. As Brown (1994) states, there are three important mechanical properties related to the base course, namely: (1) stiffness, which is to ensure good load spreading ability; (2) fatigue strength, which is to prevent cracking under repeated traffic loading; and (3) resistance to permanent deformation, which is to eliminate rutting. Thus, it is necessary to characterise foamed asphalt in terms of those three fundamental properties for base application.

1.1.3 A brief history of foamed asphalt technology

Foamed asphalt technology was introduced firstly by Professor Ladis Csanyi of Iowa State University in 1956 (Csanyi, 1957). The effectiveness of foamed bitumen stabilised ungraded local aggregates such as gravel, sand and loess had been successfully demonstrated both in the laboratory and in the field. In this process, foamed bitumen was produced by introducing saturated steam at about 172 kPa (25 psi or 1.72 bars) through a specially designed and properly adjusted nozzle. The use of water as well as air and gases was also tried as a means of producing foamed bitumen; however, it was proved, at that time, that the use of steam was the simplest, most effective and efficient (Csanyi, 1959). It can be noted that the steam foaming system is very convenient for in-plant mixing where steam is readily available, but it has proved to be impractical for in-place operation due to the need for steam boiler equipment.

Subsequently, it was reported that foam technology had been successfully implemented in many road sections in Australia. Foamed bitumen was used to stabilise sand, gravel and crushed stone aggregates and to produce road wearing course, sub-base course or sub-base, either for a local road or a heavy duty pavement (Larcombe & Newton, 1968; Bowering, 1970; Bowering & Currie, 1973; Bowering & Martin, 1976a). The foaming process was successfully modified by Mobil Oil (Australia) in the 1960s so that the hot bitumen could be accurately and consistently blended with cold water in a suitable mixing chamber.

Foamed asphalt application then gained popularity and was widely accepted as a potential pavement material worldwide, for example in New Zealand, South Africa, United Kingdom, Norway, Germany, France, etc. (Ruckel et al., 1980 and Akeroyd, 1989). Reports of use also come from Kuwait, Iran, Saudi Arabia, China, Taiwan and Thailand since the 1990s.

Investigation into FAM properties has been conducted by many researchers. For example, Abel (1978), Ruckel et al. (1982), Jenkins et al (1999) and Saleh (2006a,b) have investigated intensively on the subject of foamed bitumen characteristics. Foamed asphalt properties have been investigated using many methods, in terms of: indirect tensile strength (Loizos et al, 2004; Jenkins et al, 2004); CBR, unconfined compressive strength, Hveem stabilometer and cohesiometer (Bowering, 1970); resilient modulus (Ruckel et al., 1980; Maccarone et al, 1994), Marshall Stability (Lee, 1981; Brennen et al, 1983); triaxial test (Shackel et al., 1974; Ebels & Jenkins,

2006) and permanent deformation using dynamic creep (He & Wong, 2007 and 2008).

A comprehensive set of guidelines concerning FAM and structural design can be found in CSIR Transportek reports CR-98/077 (Muthen, 1999) and CR-2001/76 (Long, 2001), Viridis report VR1 (Nunn and Thom, 2002), and Iowa Department of Transportation report TR-474 (Lee and Kim, 2003).

In the UK, Hubert (1988) reported that Lunedale road in Dartford (Kent) was recycled using foamed bitumen in 1986. The pavement exhibited good structural performance. Foamix material was also one of a suite of materials used in a full-scale trial of re-useable materials for road haunch repair (Potter, 1996) carried out by the Transport Research Laboratory (TRL). This work concluded that, after several months curing, the material attained similar stiffness and fatigue characteristics to those of conventional hot-mix bituminous materials. After two years in-service as a structural layer, a performance equivalency of unity with 28mm DBM100 was attributed to the Foamix. This compared with an equivalence factor of 0.7 for Foamstab material in the same trial. A pilot study was then carried out to compare the performance of Foamix and Foamstab materials, to indicate whether the improved quality of in-plant mixed material justified a reduction in pavement thickness (Nunn and Thom, 2002). This study reported that the quality of Foamix material was more uniform compared to similar materials which were mixed in-place. After a year in service, Foamix cores could be reliably removed from site; on the contrary, in-place mixed material was reported to be more problematic to core even after several years. Results from this study suggested that a long-term equilibrium stiffness of 2500 MPa was achievable, and a reduction of up to 20mm in thickness of a pavement with a design traffic of 10 msa could be achieved if in-plant mixed material replaces in-place mixed material. Subsequently, the design guide suggested by Milton and Earland (1999) was taken forward and was enhanced to accommodate higher design traffic (up to 80 msa) and the strength/stiffness characteristics of the recycled materials; this new design guide is presented in TRL Report 611 reported by Merill et al (2004).

1.1.4 Considerations in using foamed asphalt technology

The primary consideration in foamed asphalt application is to promote sustainable development as recommended by Agenda 21, the agreement of 178 governments in the United Nations Conference on Environment and Development (UNCED) held in Rio de Janerio (Brazil) 3-14 June 1992. The considered benefits include following:

- Increased energy efficiency since heating of aggregates is not necessary, and reducing materials transportation when an in-place system is applied.
- Conserving natural resources when using recycled asphalt and other waste materials.
- Preserving the environment due to lowering fuel consumption and greenhouse gas emission, eliminating disposal problems and developing sustainable construction.

Economic consideration is an important factor in implementing foamed asphalt technology. Construction cost can be reduced significantly since foam technology allows use of waste and local materials, it is not necessary to heat aggregates, and it can be applied using a relatively lower binder content. These benefits give advantage when compared to conventional hot mix asphalt. However, it is reported that foamed asphalt construction is more expensive than lime/ fly ash and cement stabilisation. The recycling process necessitates use of a specialised plant and equipment, which needs to be mobilised. Therefore, it may require that a relatively large size of project or a large volume of material is processed in order to achieve the expected benefits and cost effectiveness.

Engineering considerations should be considered properly. On most projects it is reported that stabilization using foamed bitumen is easily handled, readily compacted and it is possible to open to traffic directly. The binder increases the shear strength and decreases the permeability compared to a granular pavement. The material exhibits more flexibility and hence is relatively fatigue resistant when compared to cement treated materials. The use of water is less than for a bitumen emulsion mixture, giving faster strength gains, especially in the early stage. A foamed asphalt pavement can also tolerate heavy rainfall after construction with only minor surface damage under traffic. The possibility to use a wide variety of materials, such as recycled asphalt, soil/granular aggregates and other marginal or waste material should be accompanied by adequate quality control. Due to the variable nature of road materials, especially after the roads have been overlaid and inlaid a number of times in the past, it is often difficult to produce a consistent recycled product. Furthermore, not all pavements are suitable for recycling. Pavements with low subgrade strength, or weak granular or lower bituminous layers which are susceptible to permanent deformation, may not be suitable to be treated by recycling, certainly if the recycling is confined to the upper layers. A thorough site investigation is often necessary before deciding. Before using a recycled material it is necessary to assess the risk of early deterioration in relation to the potential benefits. These factors demand understanding of the long-term performance of recycled pavement structures as otherwise they may increase the “risk cost” associated with their use.

1.2 Problem Statement

The research problem tackled in this thesis was only stated following a comprehensive review and initial study of FAM, including a basic understanding of road pavement materials and their fundamental properties. The completed studies are presented in Chapters 2 and 3.

1.2.1 Executive summary of literature review

It is necessary to select the best foam quality prior to mixing with aggregate materials. This is achieved by investigating the foaming properties of the bitumen over the full range of foaming water content (FWC). Both a high maximum expansion ratio (ERm) and a long half-life (HL) are currently understood to be desirable – which means that a compromise has to be found. The guidance proposed by Wirtgen (2005) is that the minimum permissible values of ERm and HL should be 8 and 6 seconds respectively. Based on these, a technique for deriving an optimum FWC value has been proposed. In contrast, CSIR (Muthen, 1999) suggest minimum values of 10 and 12 seconds for ERm and HL. However, these methods, though useful for field applications, are not very rigorous, particularly since the relationships of both ERm and HL with FWC are non-linear.

As an alternative, Jenkins (1999) introduced Foam Index (FI) as a function of experimental parameters ERm and HL. The definition of FI is complex and is derived from an assumption that foam collapse follows the logarithmic decay equation. Jenkins suggested a minimum FI of 164 seconds for conventional cold mix applications. In general, it is expected that a maximum FI will occur at an optimum FWC giving optimised foam quality. Further description of the FI concept can be read in Chapter 2 section 2.4.1.2.

On the other hand, some studies have found that with some bitumens, the increasing ERm values caused by higher FWC values, were not necessarily accompanied by decreasing HL values. Lesueur et al (2004), He & Wong (2006) and Saleh (2006a) found that HL values decrease up to certain FWC and then remain constant. Such a trend results in FI values increasing continually with increasing FWC and hence an optimum FWC value becomes impossible to locate.

Saleh (2006a) proposed an alternative method in which foam viscosity was measured using a Brookfield rotational viscometer at several times, and an average foam viscosity over the first 60 seconds was calculated. The relationship between FWC and average foam viscosity was plotted and the minimum viscosity value was selected as the best foam composition (see Chapter 2 section 2.4.1.3). Saleh also found that foams that were categorised as poor-performing, according to their FI values, can still be effectively mixed with aggregates. It was thus proposed that the ability of a foam to form a well coated asphalt mix was directly related to its viscosity.

However, most studies of general foam (not necessarily bitumen foam) agree that the apparent viscosity increases with the gas content and decreases with shear rate (Assar and Burley, 1986). For example, Marsden and Khan (1966) in Heller and Kuntamukkula (1987) found that increasing gas content from 70% to 80% and 90% resulted in increasing the foam viscosity from 130cp to 210 and 280cp respectively. If the same applies to foamed bitumens, then viscosity should increase with increasing FWC and hence ERm.

Crucially, neither Jenkins nor Saleh discuss the correlation between foam characteristics and asphalt mix properties, a very significant omission. Therefore, in this study it was decided to investigate the parameters affect on both the foam properties and resultant mix performance.

1.2.2 Research Need

Obviously, the influence of foamed bitumen characteristics on mix properties remains a gap in knowledge and has become a research requirement in order to enhance the overall understanding of foamed asphalt application.

1.3 Research Objectives

This research was designed to meet the research need stated above. The expected outcomes were to complete a laboratory mixture design and to propose practical guidance to produce an optimised foamed asphalt mixture.

The general aim is to achieve a better understanding of foamed asphalt properties with emphasis on the influence of foamed bitumen characteristics. Some previous findings in terms of foamed bitumen characteristics will be reviewed and their correlation to mix properties will be explored.

The specific research objectives can be identified as follows:

1) Investigate foamed bitumen characteristics based on experimental and theoretical study.
2) Investigate foamed asphalt fundamental properties in terms of stiffness modulus, resistance to fatigue and resistance to permanent deformation.
3) Explore the influence of foamed bitumen characteristics on mix properties.

1.4 Scope of Work

The scope of work and limitations of this study are as follows:

Aggregate

In this study, for the main test programme, virgin crushed limestone (VCL) aggregate collected from Tarmac Dene quarry (UK) was used without the addition of any reclaimed asphalt pavement (RAP) materials, as is commonly used for FAM. The absence of old/aged binder is considered necessary to facilitate a clearer evaluation of the effect of foam properties on the mixture. Reclaimed asphalt pavement (RAP) was only used in the initial studies in order to investigate a number of problem areas.

Testing for mixture properties

The mechanical properties of FAM in terms of stiffness modulus, resistance to fatigue and permanent deformation were evaluated using the Nottingham Asphalt Tester (NAT), using the Indirect Tensile Stiffness Modulus (ITSM), the Indirect Tensile Fatigue Test (ITFT) and the Repeated Load Axial Test (RLAT) respectively.

Foamed bitumen type

In this study, foamed bitumen was produced using a laboratory mobile foaming plant type Wirtgen WLB 10, housed in the Nottingham Transportation Engineering Centre (NTEC) laboratory. The source of binder used was Shell bitumen.

Specimens used

All FAM samples (see section 5.3.1) were cylindrical specimens compacted using a gyratory compactor. The specimens were prepared based upon the laboratory mixture design, i.e. at optimum binder content (4% of aggregate mass) and an aggregate water content of 4.6%. Prior to testing, the specimens were cured at 40^{o}C for 3 days (to simulate approximately 6 months of field curing as proposed by Lee and Kim, 2003).

1.5 Structure of the Report

Chapter 1	states the background, research problem statement, research objectives, scope of works and structure of this report. The problem statement, although stated in Chapter 1 is essentially the outcome of intensive study described in Chapter 2 and 3.
Chapter 2	reviews foamed asphalt material as the core of this study. Both foamed bitumen characteristics and foamed asphalt mixture properties are reviewed extensively. General road pavement materials and their fundamental properties are also discussed at the start of the chapter as important background to this research project. Mixture design for foamed asphalt and industry experiences in the UK are discussed at the end of the chapter.
Chapter 3	presents an initial study to report properties of the materials used in this study, laboratory works and a pilot scale experiment in the Nottingham Pavement Test Facility.
Chapter 4	reports a theoretical and experimental study of foamed bitumen. The theoretical study is an exploratory work to characterise the physics of foamed bitumen based on general foam literature, whereas the experimental study covers investigation of maximum expansion ratio and half-life, and flow rate.
Chapter 5	presents an investigation into foamed asphalt fundamental properties in terms of indirect tensile stiffness modulus, resistance to permanent deformation and resistance to fatigue.
Chapter 6	analyses and discusses the relationship between foam characteristics and mix properties. This chapter builds a bridge to link the results of Chapter 4 and Chapter 5.
Chapter 7	proposes practical guidance to produce an optimised foamed asphalt mixture.
Chapter 8	states the main findings, conclusions and recommendations.

2 LITERATURE REVIEW

2.1 Introduction

Chapter 2 presents a literature review which is focused on general road pavement materials and foamed asphalt characterization. This will include a discussion about material properties and mixture design for foamed asphalt materials. An understanding of what road pavement materials are and how to characterise them is required in order to provide a correct understanding of foamed asphalt mixture (FAM) as a road material.

The full review on characterization of foamed bitumen and properties of foamed asphalt is designed to support the summary and problem statement given in Chapter 1. Mixture design for foamed asphalt materials is described at the end of this chapter to provide a correct understanding of the procedure to attain an optimum end product. This is included as an explanation of sample preparation for laboratory, required prior to conducting the initial study reported in Chapter 3.

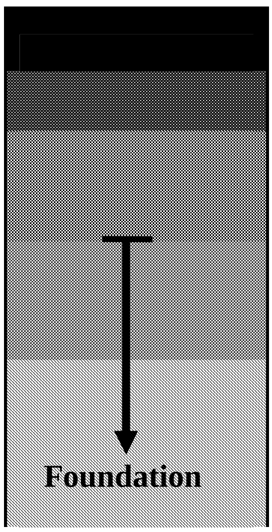
2.2 Road Pavement Materials

2.2.1 Pavement layers

A combination of road pavement materials is required to support traffic loads depending on the existing soil sub-grade, which does not generally have ability to support traffic directly. Commonly, pavements contain essentially the following four components (Figure 2.1):

- The pavement foundation, formed by compacted existing soil or imported fill (sub-grade), optional selected material (capping layer) and stabilized/ unbound granular material (sub-base).
- The base, that can be formed using bituminous material (e.g. dense bituminous macadam), rigid construction (Portland cement concrete), stabilized material (cement bound material, foamed asphalt mixture, etc.) or unbound granular material.

- The binder course, that can commonly be formed using bituminous material (e.g. dense bituminous macadam) and rigid construction (Portland cement concrete).
- The surface course, that is principally to provide a good riding quality (skid resistance, noise, spray, etc.) and prevent water infiltration. The surfacing can vary from a very thin surface dressing to 50mm dense bituminous mixture.



Pavement structure	Material type
Surfacing	A thin surface dressing / Asphalt
Binder course	Asphalt/ cement concrete
Base	Asphalt/ cement concrete/ stabilised road base/ unbound granular material
Sub-base	Bitumen or cement stabilised material/ unbound granular material
Sub-grade	In-situ material/ cement stabilised/ unbound granular material

Figure 2.1 - Typical structure of road pavement

The pavement foundation provides a platform onto which the more expensive upper layers materials are placed. In some cases, capping layer is used to provide a formation and support the sub-base course using relatively cheaper materials. Surfacing is normally designed to provide an adequate level of serviceability, whereas binder course is a structural layer. It can then be understood that the base and binder course, as intermediate layers, are primarily structural layers to spread the traffic load stress. Thus, in an ideal pavement as shown in Figure 2.2 (Brown, 2000) the important mechanical properties related to the materials used for these layers are stiffness, fatigue strength and resistance to permanent deformation (Brown, 1994). Stiffness is required to ensure good load spreading ability, fatigue strength will prevent cracking due to traffic loading and resistance to permanent deformation eliminates rutting.

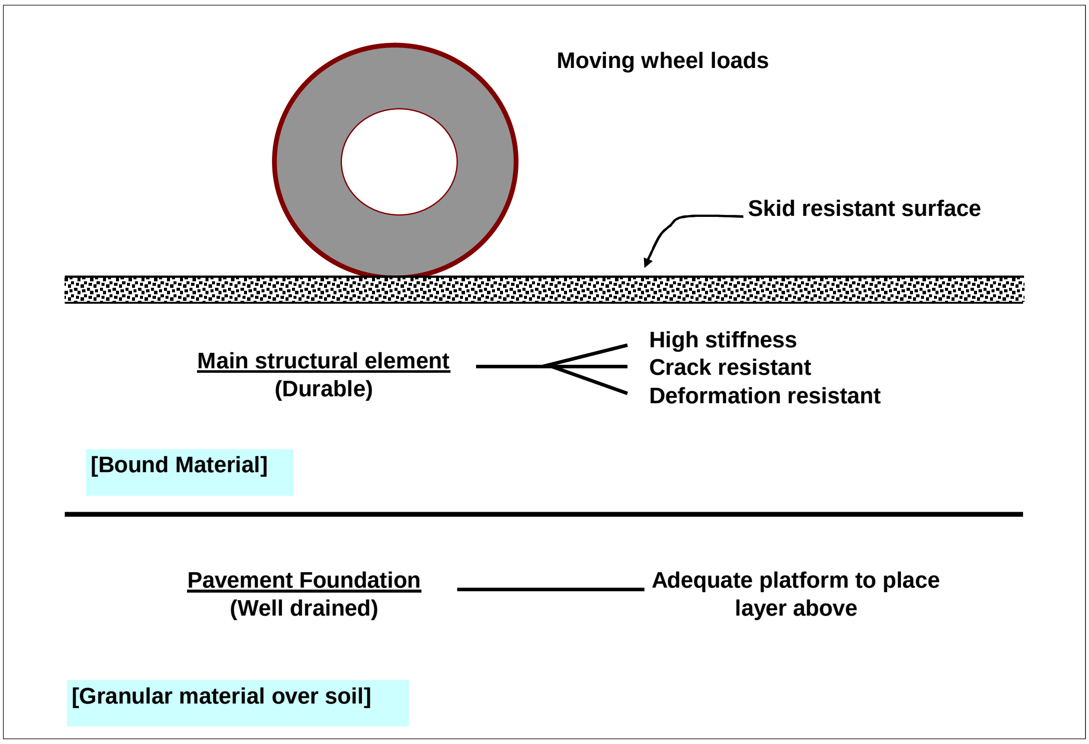


Figure 2.2 - The 'ideal' pavement (after Brown, 2000)

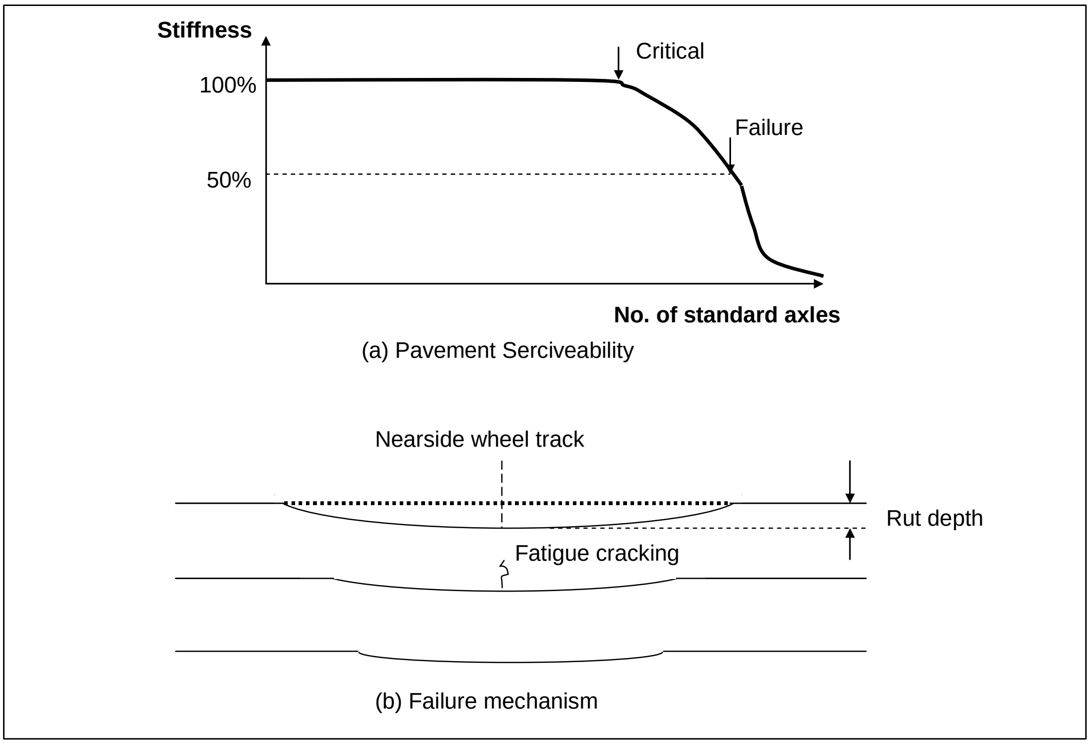


Figure 2.3 - Failure mechanism in pavement material

2.2.2 Failure mechanism

It is essential to understand how pavement materials fail under repeated traffic loading. Pavements do not fail suddenly, but gradually deteriorate over a period of time. The deterioration is caused by both the magnitude and the number of load applications onto the pavement, which induce stresses and strains in the structure. This gradual deterioration takes the form of rutting and cracking, as shown in Figure 2.3b, as the two principal failure mechanisms in a road pavement, described in Brown and Brunton (1986) and Brown (1996).

2.2.2.1 Cracking

Cracking can initiate at the pavement surface and propagate toward the bottom of the layer for thicker pavements. Surface cracking is also possible at the shoulders of ruts. However, for thinner pavements, cracking is commonly assumed to initiate at the bottom of the bound layer, where the maximum tensile stress and strain occur, and then to propagate towards the surface under repeated load applications (Brown, 2000). There are conflicting requirements between the need to avoid crack initiation, which requires a high asphalt stiffness, and to minimize crack propagation, which requires a low stiffness (Brown, 1996).

Brown (2000) also stated that the stiffness of a pavement can be utilized as a failure indicator since it changes with time and trafficking. Serious deterioration starts off at a point sometimes known as the ‘critical condition’ of the pavement, with failure at, typically, 50% of initial stiffness (Figure 2.3a). The critical point used to be defined in the U.K. as a 10mm rut, at which point strengthening of the pavement is normally required.

2.2.2.2 Rutting

Brown (2000) stated that rutting is the sum of permanent deformations in various pavement layers. A single wheel load applied to a pavement material results in deformation or strain. Most of this will be recovered when the load goes away, but a small part still remains. Over time, with a large number of load applications, these small irrecoverable strains accumulate to form permanent deformation, which manifests as rutting in the pavement. Rutting may be the result of permanent deformation from various layers, and is often divided into structural rutting (granular and base layers) and non-structural rutting (surfacing).

Structural rutting might be due to the impact of poor foundation layers. The strength of foundation materials (τ) is mainly affected by the effective stress (σ') and the angle of frictional resistance ($\text{Ø}'$) of granular materials since $\tau = \sigma' \tan \text{Ø}'$. The effective stress can be increased by decreasing the pore water pressure (u) since $\sigma' = \sigma - u$ (note: σ is total stress), for example by draining the soil. It is noted that rutting in the foundation might be caused by the applied stress being large or by the pore water pressure being high. Therefore, it is important to ensure that the upper layer can provide good load spreading to reduce stress in the foundation and to construct well-drained foundation layers (Dawson, 2000).

The resilient stress–strain response of foundation materials to trafficking is an important factor determining the thickness and stiffness of higher layers. It is known that granular materials, under load, also result in resilient (recoverable) and plastic (irrecoverable) strains. Large resilient strains can lead to premature fatigue failure of higher layers, whereas a large plastic strains result in rutting. It is well known that granular materials exhibit a stress-dependent modulus, and that the modulus decreases as the live load stress level increases. At a certain high stress level, the material fails. For deep layers, the stress levels in foundation materials rarely reach failure. Sufficient load spreading has to be provided by upper layers so that the stress level is kept sufficiently far from failure. The failure concept within foundation materials has been described by Dawson (2000).

2.2.3 Fracture mechanism in binder-aggregate mixture

Thom and Airey (2006) introduced the fracture phenomenon as a failure mechanism in a pavement at the microscale. This concept was applied to the phenomenon of fracture between filler and binder in a mortar, and between graded aggregate and binder in an asphalt mixture. The fracture mechanism both in the granular matrix and in the asphalt under repeated loading was also discussed. Overall, this concept may improve understanding not only of the fracture mechanism of the mixture, but also of how aggregate and binder interaction develops mixture strength and response to loads. This concept will be useful in understanding how foamed asphalt develops mixture stiffness in a case where not all aggregate particles are coated by binder.

2.2.3.1 Fracture phenomenon in mortar

In mortar, fracture is assumed to occur either at the interface between the bitumen and a particle or within the bitumen phase of a mortar. It is also assumed that the fracture surface will try to form at right angles to the direction of principal tensile stress and that it will follow the contours of particles as necessary, establishing the shortest path. Figure 2.4 shows a schematic of a fracture surface in the case of a spherical particle(s).

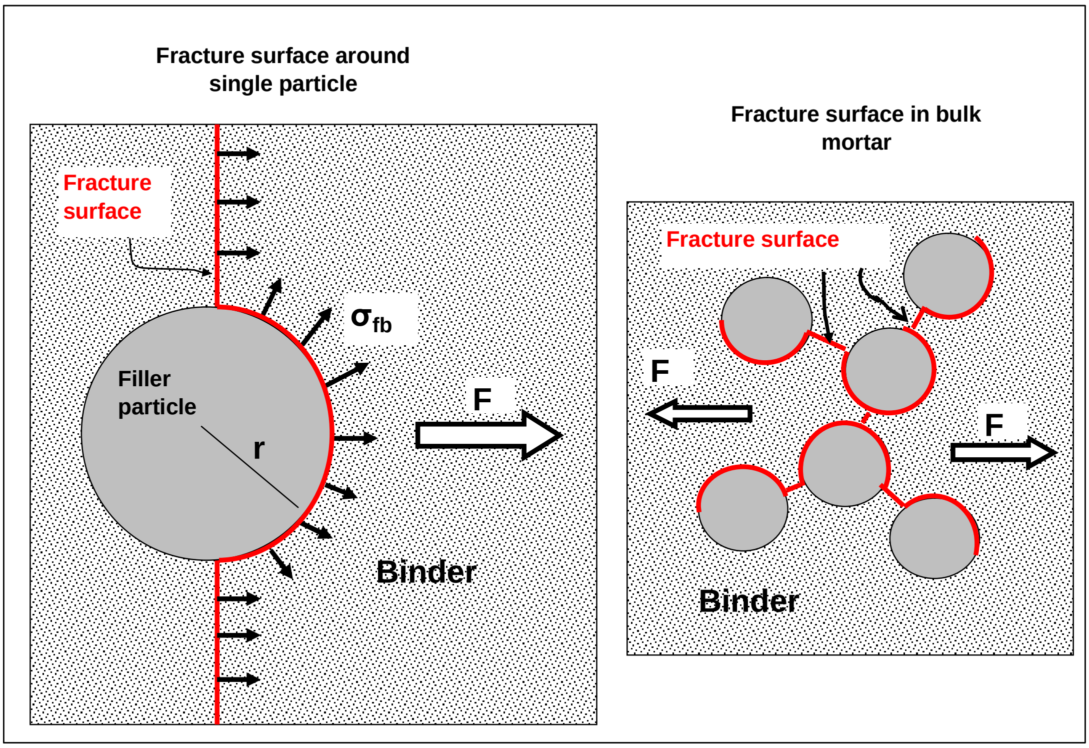


Figure 2.4 - Schematic representation of fracture surface in mortar (After Thom and Airey, 2006)

Thom and Airey predicted that the average force (F) to cause fracture over the face of a particle depended on particle size (r) and tensile fracture strength of binder (σ_{fb}). The strength of mortar was predicted to be a function of the filler proportion (V_f): $\sigma_{f(mortar)} = (1+ 2V_f)\ \sigma_{fb}$. The strain value was estimated by dividing the strength by the modulus (E_{mortar}). For mortar with low filler content, E_{mortar} was found to be given by E_b / (1- V_f), where E_b is stiffness of binder. At higher filler content, frictional interaction between filler particles would increase the effective modulus; therefore the modulus of mortar could be defined as $E_{mortar} = [E_b / (1- V_f)] * [1+V_f /(1-V_f)]$.

2.2.3.2 Fracture phenomenon in asphalt mixture

In an asphalt mixture, at the point of contact between aggregate particles, the binder film is squeezed out to zero thickness allowing direct contact. Normal inter-particle motion then occurs, whether shear motion, rotation or separation. This motion will be resisted by the binder surrounding the point of contact. If the binder is absent, it means in granular matrix, inter-particle movement within the aggregate can occur giving a low modulus. Thus, the presence of binder in the granular matrix resists inter-particle motion and increases the modulus. The fractures probably take place when the binder experiences infinite or near-infinite strain as particles move relative to each other. The binder will fail in a zone of sufficient extent such that the binder outside that zone is able to resist the stress/ strain induced as result of local inter-particle motions.

Under repeated load, a granular material develops both recoverable and irrecoverable strain. That part of the inter-particle motion forming irrecoverable strain will be slightly different from one cycle to another cycle due to the slight particle rearrangement. In general, the amount of particle rearrangement and irrecoverable strain will decrease with increasing number of load cycles as the granular matrix reaches an increasingly stable interlocking state.

Similarly, a reduction of irrecoverable strain in asphalt under repeated load obviously occurs. However this irrecoverable strain is caused by increasing number of fractures that occur around all particle contacts where inter-particle motion takes place, and an increasing freedom of motion; therefore this causes a reducing stiffness modulus. Furthermore, asphalt fatigue is known to be strain-dependent. Thom and Airey considered that fatigue failure would occur when the fractures became continuously linked. The number of cycles to achieve failure is primarily dependent on the degree of strain induced. The stress required to reach this strain is affected by loading rate and temperature and will reduce with increasing number of fractures.

It is likely that, according to this concept, the cracking will potentially start either at the weakest bonds between aggregate particles and binder or where the stress in the material is highest.

2.2.4 Types of road materials

Pavement materials can be categorized into two groups namely bound and unbound materials. The bound materials generally use bitumen as the binder, to produce bituminous mixtures that give a flexible response, or use Portland cement to produce concrete mixtures that give a rigid response. Unbound materials are normally a combination of coarse, medium, fine aggregates and filler, producing a granular mixture. Bound materials are commonly used for surface and base course layers that require material to have resistance to horizontal compressive and tensile strain, whereas unbound materials are mostly applied as base course and sub-base that require material to be resistant to horizontal tensile strain and vertical compressive strain respectively (Lister et al, 1982).

Regarding how bituminous materials are mixed, there are two known groups, namely hot mix asphalt (HMA) and cold mix asphalt (CMA). HMA is a mixture in which all materials are mixed at high temperature. Bitumen heating is required to decrease its viscosity to enable coating of the aggregate and mixture workability. Heating of the aggregate ensures no water is present inside the aggregate and makes bitumen adhere to its surface more easily. Examples of HMA are Dense Bitumen Macadam (DBM), Hot Rolled Asphalt (HRA), Stone Mastic Asphalt (SMA), etc. Unlike HMA, CMA is a mixture of aggregate and binder that is mixed at ambient temperature. This method is generally more economical, less disruptive and environmentally friendlier than HMA. Emulsion and foamed bitumen are examples of CMA binders. Applying CMA may be a greater risk than using HMA (Thom, 2005) due to the inherent nature of the material such as lower binder content, higher void content, poorer mechanical properties and limits on early trafficking. This is probably the reason why the use of

CMA lags behind HMA applications. But, it is quite certain that the use of CMA can be improved with a better understanding of material properties and if a much clearer estimate of the material quality can be provided.

The following sections describe bitumen emulsion (a CMA material) and cement bound granular mixture (as the principal alternative to CMA). Foamed asphalt descriptions are detailed in sections 2.4 and 2.5.

2.2.4.1 Bitumen emulsion

The mixture of aggregate and bitumen emulsion is one example of CMA that contains water inside the material. Water comes from three sources: water within the aggregate, water incorporated in the emulsion, and pre-wetting water added to the aggregate before the addition of emulsion. This water is used to achieve uniform coating during mixing.

Bitumen emulsion is a bitumen-water suspension in which the bitumen is dispersed throughout the water phase in the form of discrete globules, typically 0.1 to 5 microns in diameter, which are held by electrostatic charges stabilized by an emulsifier (Read and Whiteoak, 2003). Bitumen emulsion can be manufactured by a colloid mill machine in which the hot bitumen (100 to 140^{o}C) is poured into the mill and broken into small globules and then becomes coated with the emulsifier to give it electrical charge. The working system of emulsion is caused by the presence of charge surrounding the bitumen surface, which means the droplets repel each other (Thom, 2005). The bitumen proportion in the road emulsion is normally in the range of 50-70% by weight for the requirements of storage stability and viscosity (Leech, 1994).

The functions of emulsifier are it makes emulsification easier by reducing the interfacial tension between bitumen and water, prevents coalescence of droplets and dictates setting rate and adhesion (Read and Whiteoak, 2003). In the emulsion, the ionic portion of the emulsifier is located at the surface of the bitumen droplet. In cationic emulsion, this will impart a positive charge to the bitumen droplet surface, whereas in anionic emulsion, this will impart a negative charge.

The bitumen emulsion stability can also be well described by the DLVO (Derjaguin-Landau-Verwey-Overbeek) theory (Lesueur and Potti, 2004). According to this theory, inter-droplet interactions are the sum of an electrostatic repulsion and a Van-der-Walls attraction. When the repulsion overcomes the attraction, the droplets are prevented from approaching each other and the emulsion does not break. On the other hand, when the attraction overcomes the repulsion, the droplets tend to contact and then coalesce.

The aggregate coating process can be described in the three steps below:

- Initial position: emulsifier molecules located both in the water and on the droplet surface,
- Breaking process: aggregate with negative charge on its surface absorbs some of the ions from the solution and causes weakening of the charge on the droplet surface,
- Coating process: When the charge on the surface droplet is sufficiently depleted, rapid coalescence will take place and the aggregate surface is easily adhered to by the bitumen.

Bitumen emulsion mixture can be applied using three types of aggregate grading (Ibrahim, 1998): open graded mixture, semi-dense mixture and dense graded mixture. Open graded mixture has a high porosity and is easier to deform, semi-dense mixture has greater cohesion and is less permeable, whereas dense graded mixture has high cohesion and good resistance to deformation. Aggregate grading has a significant effect on the mixture stiffness and a fine dense grading appears to give the highest stiffness modulus. Lesueur and Potti (2004) suggested in their proposed mix design that the aggregate grading should be selected to allow a good compactability and to provide aggregate reactivity and surface area in accelerating the breaking process.

A high water content is a huge help for emulsion distribution in the mixing process and hence to achieve a good coating of the aggregate. On the other hand, an excess of water at compaction will reduce density and other properties (Ibrahim, 1998). Therefore, it is recommended to use a compromise water content which can give both a uniformly coated aggregate in the mixing process and maximum density in the compaction process.

2.2.4.2 Cement bound granular mixture (CBGM)

In the UK a mixture of aggregate and Portland cement is called a cement bound granular mixture (CBGM) and it is one of the hydraulically bound mixture (HBM) family (MCHW-1 and 2, 2007; and BS EN 14227-1: 2004). CBGMs are extensively used as sub-base and base courses in flexible pavements and they constitute the principal alternative to cold mix asphalt materials.

According to BS EN 14227-1: 2004 CBGMs are classified in terms of their cement content, water content, and strength and/or elastic modulus of the hardened mixture. The cement and water content are determined by a design procedure and/or experience with mixture produced using its proposed constituents. The design procedure should demonstrate that it is suitable for achieving both compliance with the mechanical properties requirements and any site density requirements. The minimum cement content is dependent upon the maximum nominal aggregate size, whereas the water content being suitable for compaction depends on the aggregate size, the cement content, the climatic condition on site, the distance of transportation, the equipment being used for compaction etc (BS EN 14227-1: 2004). CBGM is classified by the strength properties of the job standard mix either by (a) the characteristic compressive strength of specimens at an age of 28 days (system 1), or (b) the characteristic direct tensile strength or the indirect tensile strength and the modulus of elasticity of specimens at an age of 28 days (system 2).

Based on the classification system as described in BS EN 14227-1 (2004), MCHW-1 (2007) categorizes CBGMs into three groups, i.e. CBGM A (broad graded aggregate), CBGM B (close graded aggregate) and CBGM C (graded mixture). This grouping system is based on the aggregate type and hence the strength. This system replaced the old system in which the mixtures were called cement bound materials (CBMs) and were categorized into classes based on their 7 days cube compressive strength i.e. CBM1, CBM2, CBM3 and higher classes. CBM1 and CBM2 were intended to be used for sub-base material, whereas CBM3 and higher classes were used for base course materials (MCHW-1, 1993). CBGM A is approximately equivalent to CBM2 and CBGM B and C are approximately equivalent to CBM3, 4 and 5.

Aggregate grading and maximum size are important in relation to CBGM properties. The grading requirements of materials for different categories of CBGM are specified in the MCHW-1: 2007. Two maximum aggregate sizes can be used i.e. 60mm (zone A graded) and 20mm (zone B graded). If the larger size is used, the mix requires lower water content so that, for a given workability and cement content, the water cement ratio can be lowered with a consequent increase in strength (Neville, 1995).

Cement content is the most important factor influencing a CBGM’s strength, as well as shrinkage cracking (Pendola et al, 1970). The amount of cement varies from 3% (clean aggregate) to 10% (soil) and is directly related to the resulting strength. The determination of a CBGM’s cement content to achieve a particular strength is conducted by means of cube compressive strengths over varying cement contents at their respective optimum water contents. The procedures for preparing, curing and testing of the specimens are described in BS EN 13286-50 to -53, using test methods conforming to BS EN 13286-40 to -43. The type of cement may be Portland cement, Portland blast furnace slag cement or Portland PFA cement (TRL 386, Milton and Earland, 1999).

Shahid (1997) has investigated the mechanical properties of CBGM. The strength property was found to be sensitive to variations in mix water content and degree of compaction. Elastic stiffness of CBGM both in tension and in compression, which have equal value, was influenced by the type and quality of aggregate and had a linear relationship with its strength. The Poisson's ratio value, as a function of the aggregate grading and cement content, varied from 0.14 to 0.24. Two stages of cracking were also studied. The primary cracking consisted of transverse cracks due to shrinkage and thermal effects, whereas the secondary cracking consisted of longitudinal cracks due to traffic loading.

As specified in TRL 386 (Milton and Earland, 1999), Portland cement as a binder for in-situ recycling works to produce a flexible composite pavement structure, is adaptable to most conditions although the thermal cracking is a danger in stronger mixes. It has the following advantages: (a) it is readily available at reasonable cost, (b) it is generally suitable for mixing with a wide range of pulverized aggregates comprising existing pavement materials, (c) it is tolerant of contamination by plastic fines and organic material, (d) it is tolerant of adverse wet weather (often the case in UK), and may be used for either cement stabilized or cement bound materials.

In application of cement, specialist spreaders are vital which should incorporate a control system to ensure that the rate of spread is achievable to a target accuracy of ±0.5% of the specified spread rate. The rate of spread is measured by weighing the amount of material retained on each of five trays or mats of known area laid in the path of the spreading machine (TRL 386, Milton and Earland, 1999).

2.3 Fundamental Properties of Bituminous Materials

2.3.1 Stiffness Modulus

Stiffness of bituminous material is defined as the ratio of uniaxial stress and the corresponding strain. It depends on temperature and loading time due to the visco-elastic behaviour of bitumen that affects the strain response. Table 2.1 shows the stiffness behaviour and the main parameters which affect it. For intermediate conditions, when visco-elastic behaviour occurs, the parameters are in a transitional state (Brown, 2000).

Figure 2.5 shows that bitumen stiffness and mixture properties, i.e. Void in the mix aggregate (VMA), significantly affect the mixture stiffness. The effect is different for different conditions of bitumen. It was found that a bitumen stiffness of approximately 5MPa is the border between elastic and visco-elastic region (Brown, 2000).

Table 2.1 - Stiffness behaviour of bituminous materials

Condition	Type of behaviour	Parameters affecting performance
Low temperature or short loading time	elastic	Bitumen stiffness (S_b) and Void in the Mix Aggregate (VMA) of mixture
High temperature or longer loading time	viscous	S_b, VMA and aggregate properties (type, grading, shape and texture), compaction and confining conditions

The term 'visco-elastic' is normally used to indicate that bituminous materials have both viscous and elastic behaviour; viscous at high temperature and elastic at low temperature. Elasticity means the material deforms instantaneously when loaded and recovers immediately and completely when unloaded whereas viscous material deformations build-up when loaded and irrecoverable strain remains when unloaded.

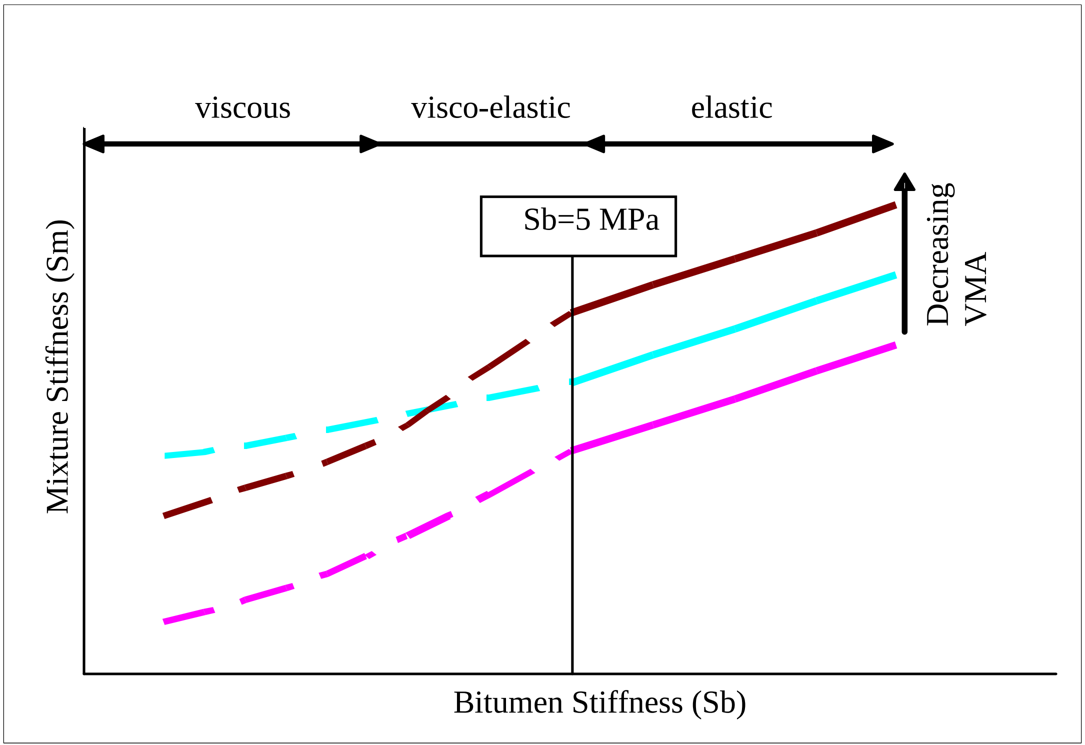


Figure 2.5 - Relationships between Bitumen Stiffness and Mixture Stiffness (Brown, 2000)

Figure 2.6 illustrates a sample of material with length L and cross section area ab, subjected to a direct normal load P in tension in the x direction. This uniaxial loading causes a direct stress σ = P/ab and strain $\varepsilon_x = x/L$. The modulus of elasticity (also called Young's Modulus), E, is defined as shown in Eq. 2.1. Elastic modulus has units of stress, e.g. N/m^2 or Pa. It can be seen in the figure that when the material is loaded, not only does it get longer, but it also decreases its cross-sectional area. The resulting deformations in the y and z directions, when divided by the corresponding dimensions, a and b, yield strain ε_y and ε_z. The ratio of these strains to ε_x is defined as 'Poisson's Ratio (v)'. Poisson's Ratio is dimensionless and generally ranges between 0.1 and 0.5. A value of 0.5 implies that no volumetric change is taking place since increasing dimension in the x direction is compensated by decreasing dimension in the two other directions. For bituminous materials, their modulus varies with temperature and loading time, therefore the term 'stiffness modulus' is used instead of elastic modulus. Normally the stiffness of bituminous mixtures ranges between 1GPa and 10GPa, whereas Poisson's Ratio is around 0.35.

$$E = \frac{\sigma}{\varepsilon} \qquad \textbf{...Eq. 2.1}$$

Where:
E = Elastic modulus
σ = Applied stress
ε = Resultant strain

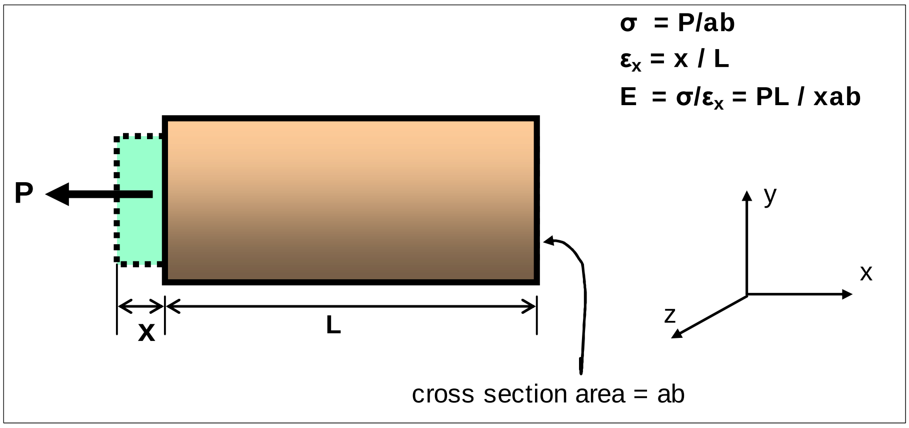


Figure 2.6 - Elasticity of material

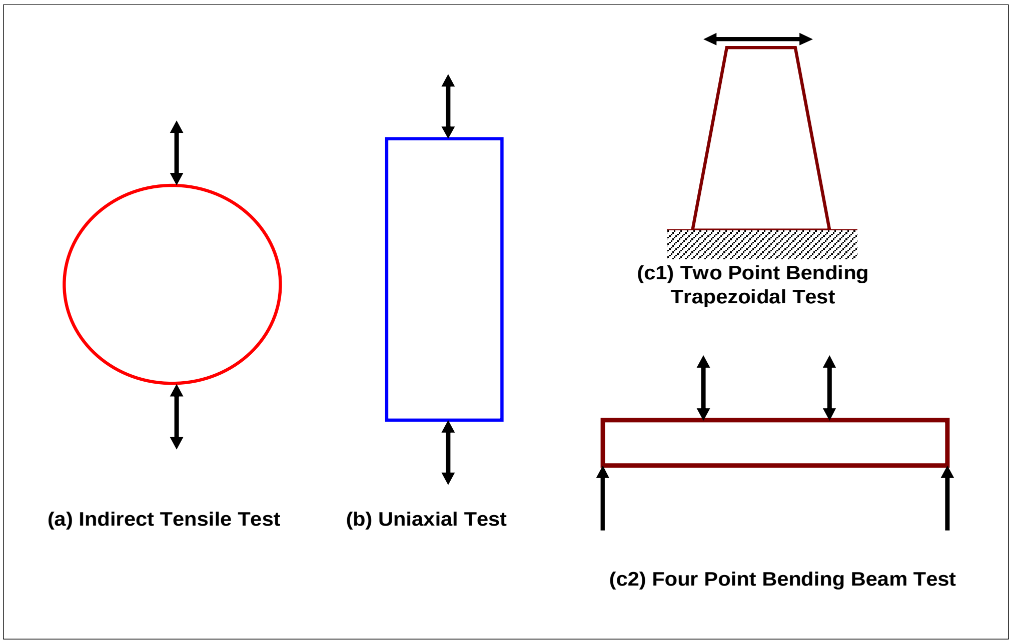


Figure 2.7 - Laboratory test methods for elastic stiffness

Stiffness modulus is an important property for bituminous base course layers. Increasing the elastic stiffness improves load-spreading ability, thus reducing the peak stress transmitted to the sub-grade.

Common laboratory test methods to measure stiffness under visco-elastic conditions are: (1) the repeated load indirect tension test, (2) the uniaxial repeated load test and (3) repeated load beam tests. Figure 2.7 shows those three methods including two kinds of beam test systems. In the UK, the first method, namely the Indirect Tensile Stiffness Modulus (ITSM) test, has been widely used. It has been confirmed that the ITSM test results give a good correlation with other test methods such as bending beam (Cooper and Brown, 1989). Stiffness measurement may also be utilised to indicate mixture quality such as temperature susceptibility, water sensitivity (adapted from BS EN 12697-12: 2003), damage and ageing.

The ITSM test, which was selected for use in this study, can be performed in the Nottingham Asphalt Tester (NAT) in accordance with BS DD 213:1993; it is known as a non-destructive method (Cooper and Brown, 1989). This testing method uses cylindrical specimens (100mm or 150mm in diameter) that may be prepared in the laboratory or sampled from the field. Figure 2.8 shows the typical test configuration for ITSM.

Table 2.2 shows typical NAT stiffness moduli at 20^{o}C for various materials (Widyatmoko, 2002). It can be seen that stiffness values depend not only on binder stiffness and aggregate packing, but also on age of materials. Normally, aged specimens have higher stiffness than fresh materials. Where it is found to be lower, this may indicate damage e.g. by micro-cracking. Read (1996) has also summarized the factors affecting the ITSM, as listed in Table 2.3.

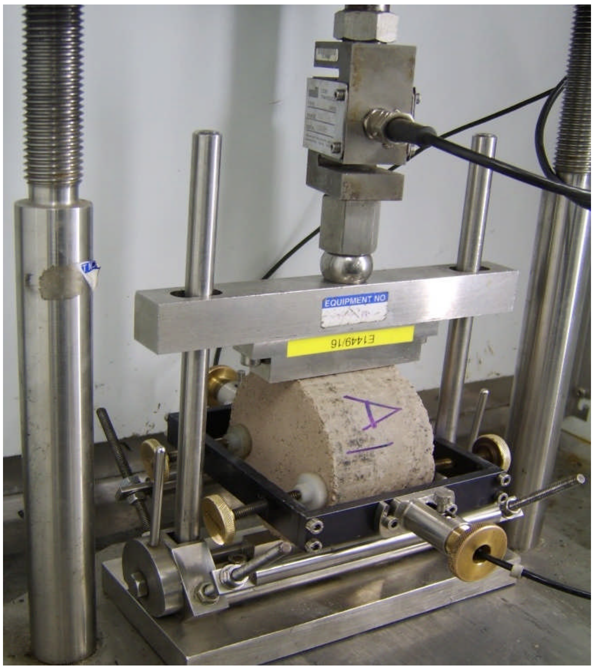


Figure 2.8 - ITSM test configuration

Table 2.2 - Typical stiffness moduli for standard materials at 20^{o}C

Material	Typical stiffness (MPa)	
	Fresh material	Aged material
HRA Wearing Course (50pen)	2000 – 3000	x 1.5
HRA/ DBM (100pen)	1000 – 2500	x 1.75
DBM50	2400 – 5000	x 1.5
HDM50	3100 – 6700	x 1.5

Table 2.3 - Factors affecting the ITSM (Read, 1996)

Factors	General effect on ITSM
Specimen temperature during test	High temperature → low stiffness modulus Low temperature → high stiffness modulus
Loading frequency	Low frequency → low stiffness modulus High frequency → high stiffness modulus
Stress amplitude	High stress → low stiffness modulus Low stress → high stiffness modulus
Poisson's Ratio (assumed)	Low value → low stiffness modulus High value → high stiffness modulus
Bitumen grade (for a particular mixture type)	High pen → low stiffness modulus Low pen → high stiffness modulus
Bitumen content (for a particular mixture type)	Highest stiffness modulus is achieved at or very near the optimum binder content
Bitumen modifiers	Use of bitumen modifiers can increase/ decrease the stiffness of mixture that depends on the modifier type. Modifier is generally used to improve the mixture characteristics rather than its stiffness modulus. Styrene butadiene styrene (SBS) is an example of polymers that can increase the elasticity of asphalt (Yildirim, 2007)
Void content (for a particular mixture type)	High air voids → low stiffness modulus Low air voids → high stiffness modulus (for some mixtures very low air voids can result in a reduction in stiffness modulus)
Aggregate type and gradation	Crushed rocks generally results higher stiffness than gravels. For continuously graded materials the larger the aggregate the higher the stiffness. For all mixture the higher the quantity of coarse aggregate the higher the stiffness.

If testing is not feasible, the stiffness of a particular mixture at any temperature and loading time can be estimated by an empirical method. Two examples of such techniques are the University of Nottingham method (Brown and Brunton, 1986) and the Shell Nomograph (Bonnaure et al, 1977). The University of Nottingham method (Figure 2.9) requires bitumen stiffness (in MPa) and voids in mix aggregate or VMA (in %); the data required for the Shell Nomograph, as shown in Figure 2.10, are bitumen stiffness modulus (in Pa), bitumen volume (in %) and aggregate volume (in %). These two methods can only be applied for a minimum bitumen stiffness of 5MPa and, for the University of Nottingham method, values of VMA between 12% and 30%. These two methods also assume that the grading and properties of aggregate affect the elastic stiffness of the mixture since they influence the packing characteristics and hence the compaction of material (Read and Whiteoak, 2003).

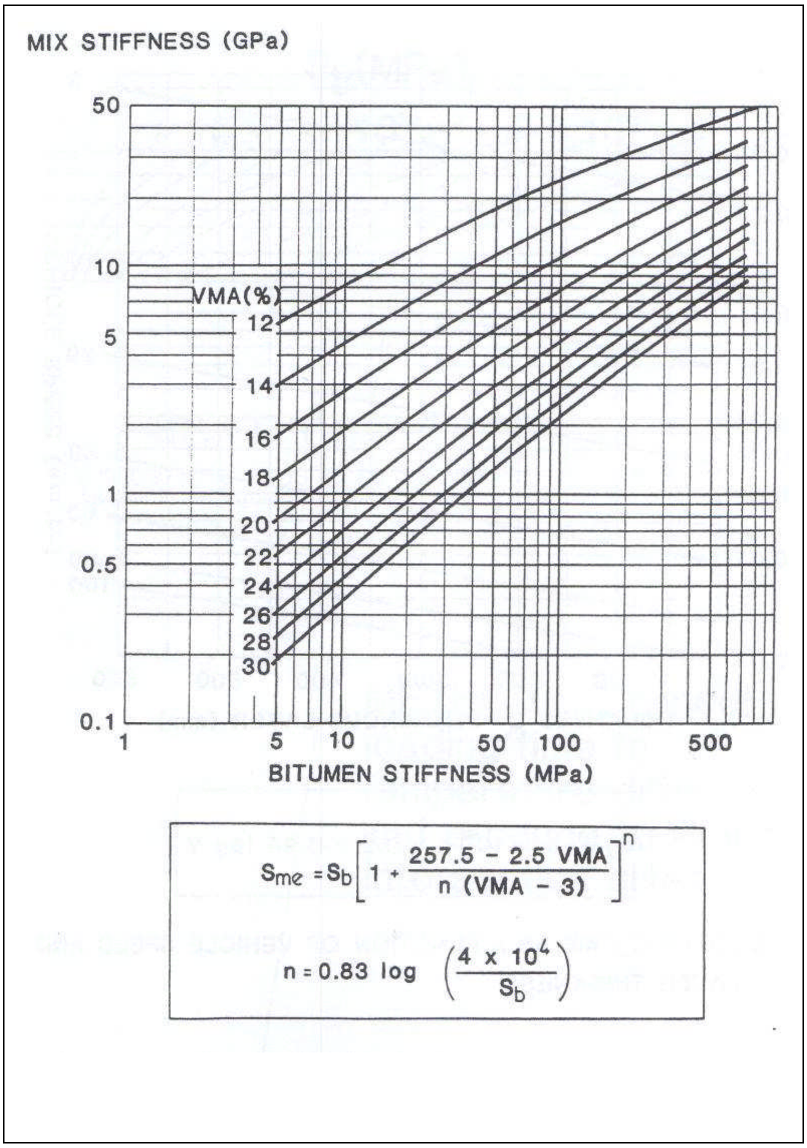


Figure 2.9 - The University of Nottingham method to predict stiffness of mixture (Brown and Brunton, 1986)

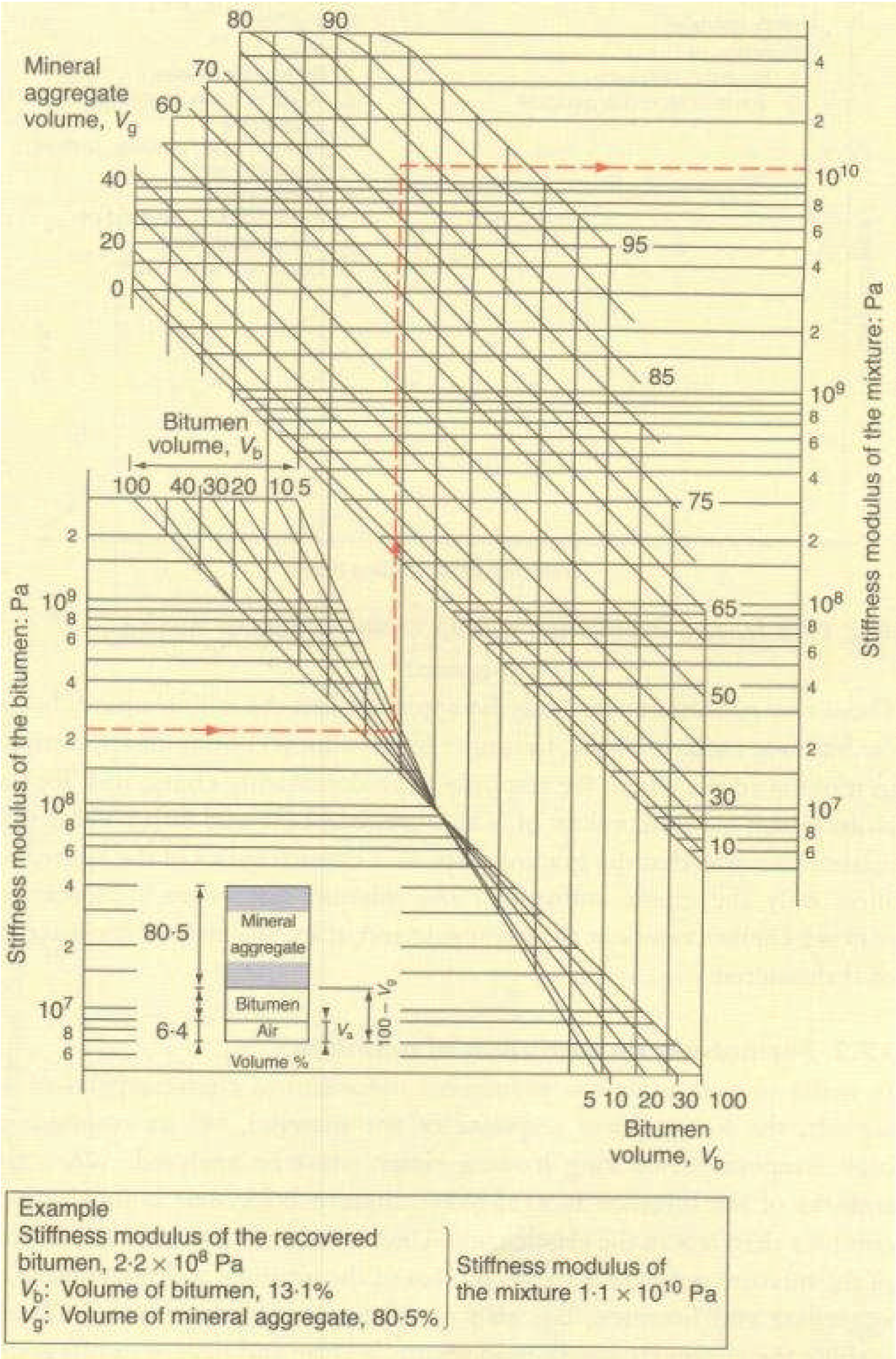


Figure 2.10 - Shell Nomograph to predict the stiffness of mixtures (Bonnaure et al, 1977). Figure adopted from Read and Whiteoak (2003).

2.3.2 Resistance to Permanent Deformation

Rutting is a common failure form for flexible pavements in which material from under the wheel path flows and compacts to form a groove or rut. Rutting is influenced by mixture properties i.e volumetric composition and material properties.

The principle of rutting development can be seen in Figure 2.11. In an idealised response of a bituminous mixture, when the load has been removed there is a small amount of irrecoverable plastic deformation. Although this deformation is small, the effect is cumulative and after a large number of load cycles a rut will develop.

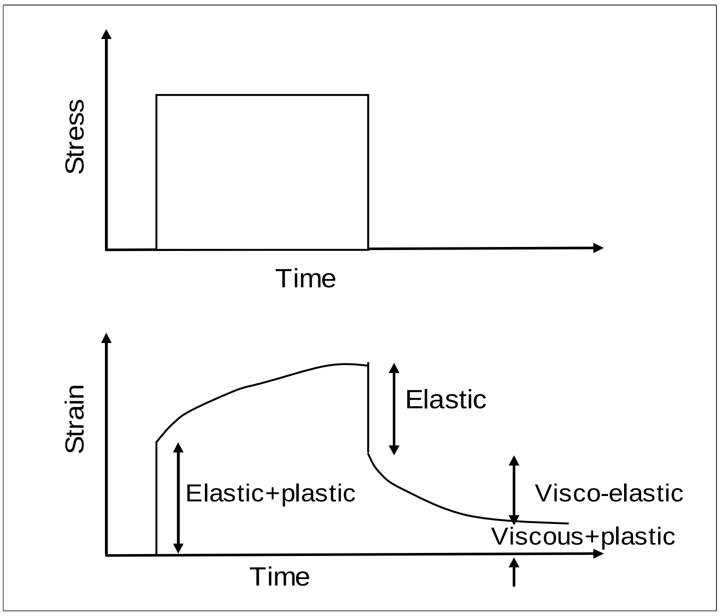


Figure 2.11 - Idealised response of a bituminous mixture

Two major mechanisms of rutting are densification (compaction) due to the repeated loading and plastic shear deformation due to the repeated action of shear and tensile stress. If a pavement has been well compacted during construction, further densification during rutting is unlikely, and permanent deformation is principally due to shear flow (Eisenmann and Hilmer, 1987).

Airey (2002a) described the effect of volumetric composition on the permanent deformation. The volumetric composition of a mixture is primarily determined by the aggregate grading, binder content and the compaction level. The relationship between voids in mix aggregate (VMA), air void (V_v) and bitumen content (V_b) can be seen in Figure 2.12(a). The V_v decreases as the V_b increases. The minimum VMA can be obtained at an optimum binder content. This corresponds to the point when the aggregate particles are most closely packed and, hence, give maximum resistance to permanent deformation. As the bitumen content increases past this point the binder films around the aggregate particles become thicker until all the voids are filled with bitumen and the resistance to shear flow is reduced.

Figure 2.12(b) shows the effect of compaction level on volumetric composition. If the level of compaction increases, the air void decreases. It should be noted that at a minimum VMA (maximum rutting resistance), if the compaction level increases the optimum binder content decreases.

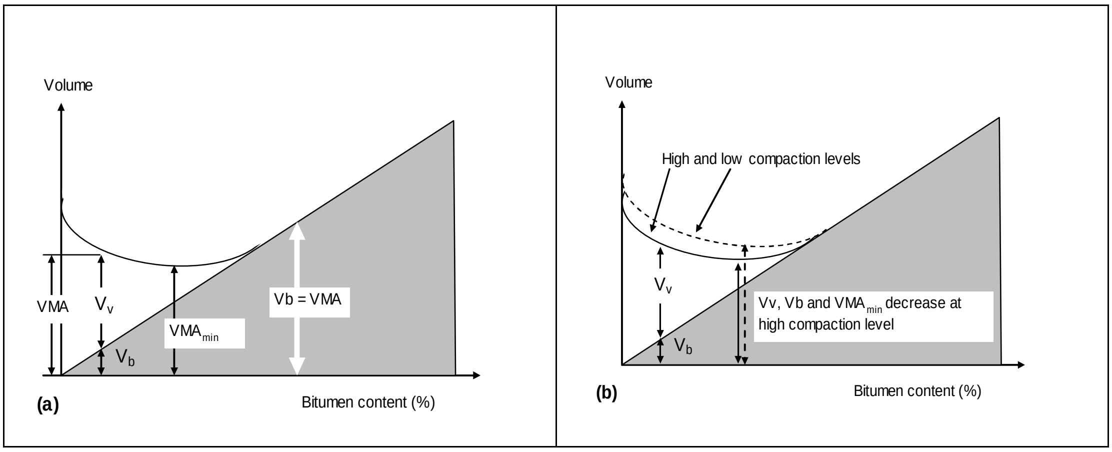


Figure 2.12 - Effect of bitumen content and compaction level on the volumetric composition

Brown (1967) found that aggregate grading and particle characteristics affect significantly the resistance to permanent deformation. Angular and rough crushed aggregates show better resistance to permanent deformation than smooth and rounded aggregates. It is noted that an angular and rough crushed aggregate mixture needs a lower binder content.

The most widely used mechanical tests for assessing permanent deformation characteristics are the repeated load axial test (RLAT) and the repeated load triaxial test. The RLAT applies a pulsed load to simulate the traffic. The RLAT protocol can be seen in BS DD 185: 1994. Figure 2.13 shows the configuration of the RLAT.

Figure 2.13 - RLAT test configuration using NAT apparatus

2.3.3 Resistance to 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 (Read, 1996).

Flexible pavement structural layers are subjected to continuous flexing under traffic loading which creates the repeated stresses, and therefore strains. The magnitude of the tensile strain is dependent upon the stiffness modulus and the nature of the pavement. These tensile strains are around 30 – 200 microstrain under a standard (80 kN) axle load at the bottom of the main structural layer in typical pavement construction (Kingham, 1973). Under these conditions, the possibility of fatigue cracking exists and, consequently, fatigue is one of the failure criteria considered in pavement design.

Fatigue characteristics of specific mixtures over a range of traffic and environmental conditions should be known in order to design asphalt pavements. In the laboratory, fatigue tests can be carried out under simple flexure (in beam tests), direct axial loading and diametral loading (Read, 1996). The method of performing a simple fatigue test is to apply loading to a specimen in the form of an alternating stress or strain of certain amplitude and to determine the number of applications of load to fail the specimen. Fatigue tests can, therefore, be applied using two main modes of loading i.e. stress controlled (constant load during test) and strain controlled (constant deformation). The results are usually expressed as the relationship between either stress or initial strain and the number of load repetitions to failure. The fatigue failure of a specific asphalt mixture can be characterized by the slope and relative level of the stress or strain versus the number of load repetitions to failure on a log-log plot.

Figure 2.14 (left) shows fatigue lives for the same material (30% coarse aggregate, nominal size 14 mm) at different temperatures. It can be seen that the lines are not quite parallel with longer lives at lower temperature. However, it has already been noted that mixture stiffness is dependent on temperature and loading time. If the results are replotted in terms of strain (using Eq. 2.4) as shown in Figure 2.14 (right), then they become approximately one line. It means that strain can be used as a failure criterion in which the effects of temperature (and also loading time) can be accounted for by their effect on stiffness. This is known as the 'strain criterion' (Airey, 2002b).

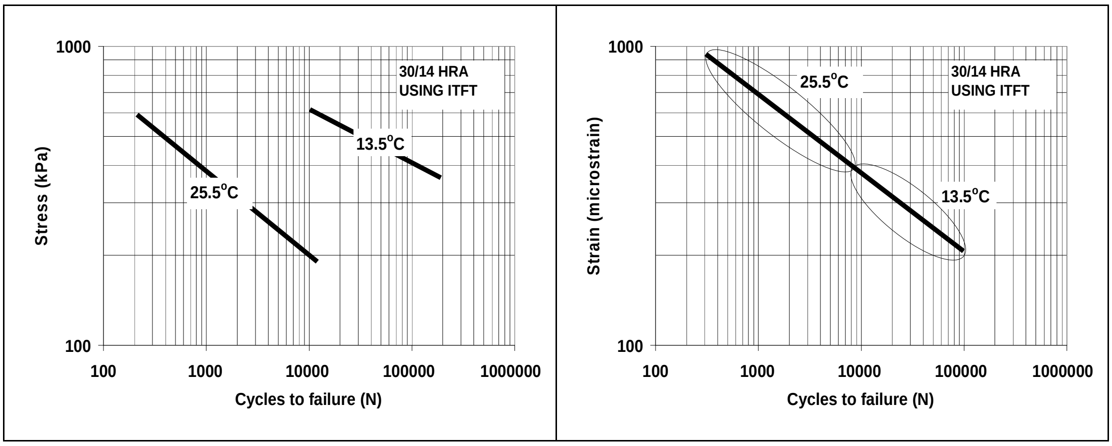


Figure 2.14 - Typical Fatigue lines under different temperature conditions based on (left) Maximum tensile stress and (right) Maximum initial tensile strain (after Read, 1996).

Fatigue life can be characterized in terms of crack initiation and crack propagation. Read (1996) proposed that this can be found when the test results are plotted as cycle number (N) v N divided by vertical deformation as shown in Figure 2.15. Therefore, fatigue life can be approached based on either crack initiation or crack propagation, in which the latter results in a steeper fatigue line.

The general relationship defining the fatigue life is as shown in Eq. 2.2. Coefficient *m* defines the slope of the strain-fatigue life line and for many mixtures has a value of approximately 5 or 6. Softer grades of bitumen give steeper lines than hard grades.

$$N_f = c \times \left(\frac{1}{\varepsilon_t} \right)^m \qquad \textbf{...Eq. 2.2}$$

where:

N_f number of cycles to failure

ε_t maximum initial tensile strain

c, m material coefficients

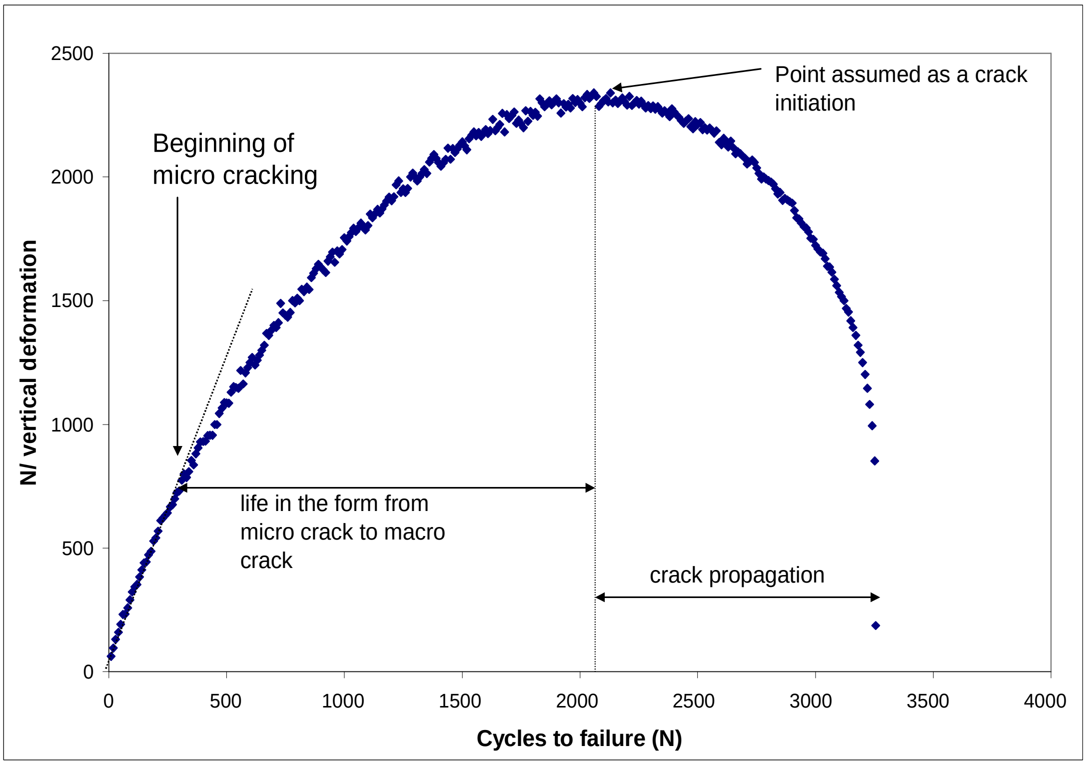


Figure 2.15 - Crack initiation and crack propagation in fatigue test (after Read, 1996)

In the U.K., the indirect tensile fatigue test (ITFT) under the Nottingham Asphalt Tester (NAT) in accordance to BS DD ABF: 2000 has been introduced. Figure 2.16 shows the configuration of ITFT.

Clearly, not only is fatigue affected by stress induced in the pavement and loading type (mode, pattern and rest periods), it also depends upon mixture variables such as stiffness, bitumen content, bitumen properties, air voids, aggregate and filler (Read, 1996).

Reportedly, fatigue resistance tends to increase with higher mixture stiffness in stress control mode of loading (Read, 1996), higher bitumen volume (Gibb, 1996), higher bitumen stiffness (Sousa et al, 1998), lower void volume (Brown, E.R. et al, 2001), higher fines content (Monismith et al, 1985), finer gradation (Sousa et al, 1998) and flakier aggregate shape (Read, 1996).

Figure 2.16 - Indirect tensile fatigue test (ITFT) configuration

2.4 Foamed Asphalt Material

2.4.1 Foamed bitumen characteristics

2.4.1.1 Maximum expansion ratio and half-life

Foamed bitumen is in essence a steam gas-hot bitumen mixture that can be generated by injecting pressurised air and a small quantity of cold water into a hot bitumen phase in an expansion chamber as shown in Figure 2.17. Soon after expulsion from the expansion chamber, the bitumen foam expands quickly to its maximum volume and remains for seconds followed by a rapid collapse process and then returns slowly to its original bitumen volume.

As described in Chapter 1, foamed bitumen is characterised in terms of maximum expansion ratio (ERm) and half-life (HL). These two experimental foam characteristics can be investigated using a laboratory foaming machine, e.g. the Wirtgen WLB 10. This is a mobile laboratory plant that is designed specially for investigating and producing foamed bitumen at laboratory scale. The foamed bitumen produced is collected in a special steel measuring cylinder and then its maximum increased volume is measured by a dipstick. The ratio of this value to the original bitumen volume (known) is calculated as ERm, whereas the time needed by the foam to collapse to half its maximum volume is recorded as the HL. Figure 2.18 illustrates the testing method to measure ERm and HL.

ERm and HL are dependent parameters which are influenced by foaming water content (FWC), bitumen type and bitumen temperature (Brennen et al., 1983) and also by machine setting, i.e. water and air pressure, and nozzle size (Castedo Franco and Wood, 1983). Ruchel et al. (1982) added that the size of measuring cylinder also affected the foam parameters. Softer bitumen types and the higher bitumen temperatures generally produce better foam quality. Abel (1978) added that acceptable foaming was only achieved at temperatures above 149^{o}C. However, others have reported that softer and higher temperature bitumens do not always have better quality (Ruenkrairergsa et al., 2004 and He and Lu, 2004). This is because foam with lower viscosity and relatively low surface tension will be more likely to collapse prematurely before reaching its maximum volume than a higher viscosity foam (He and Wong, 2006).

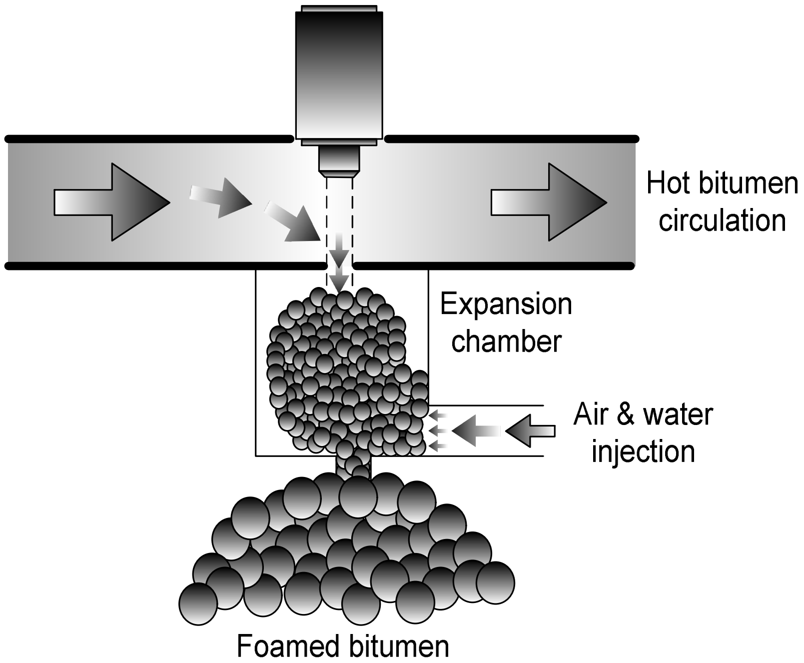


Figure 2.17 - Foamed bitumen produced in an expansion chamber

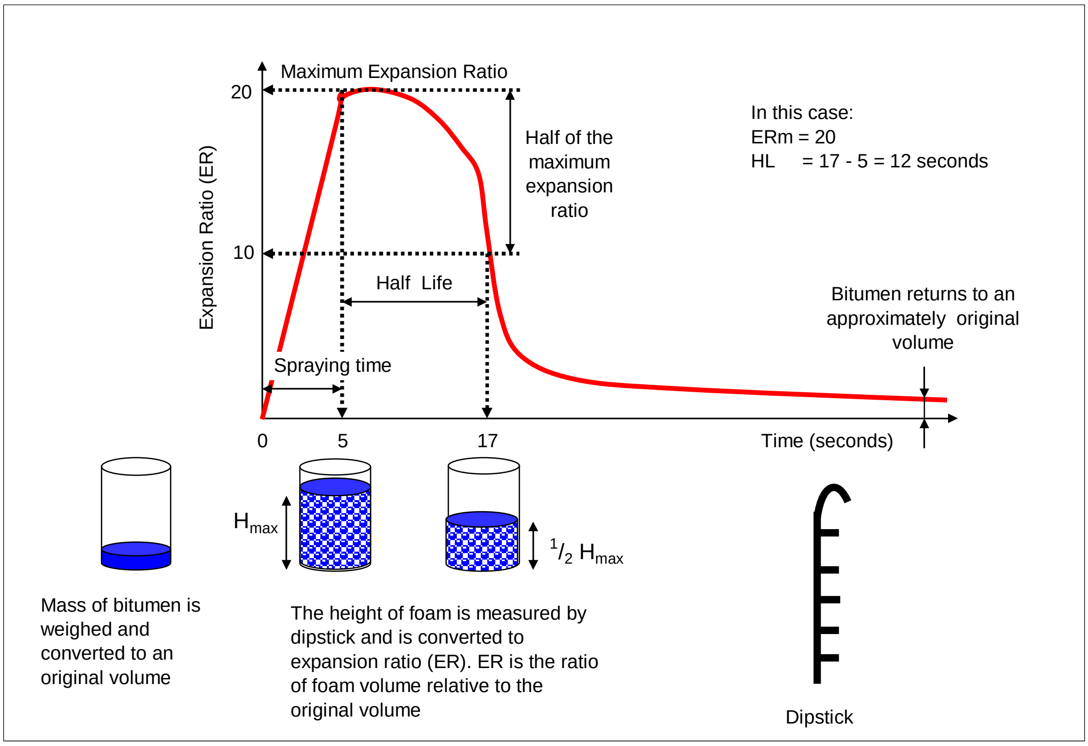


Figure 2.18 - Illustration for measurement of maximum expansion ratio and half-life

Figure 2.19 shows typical characteristics of foamed bitumen in which foaming water content (FWC - % by bitumen mass) has the greatest effect on maximum expansion ratio and half life (Jenkins, 2000). It can be seen that ERm increases with increasing FWC, whilst the HL values tend to decrease.

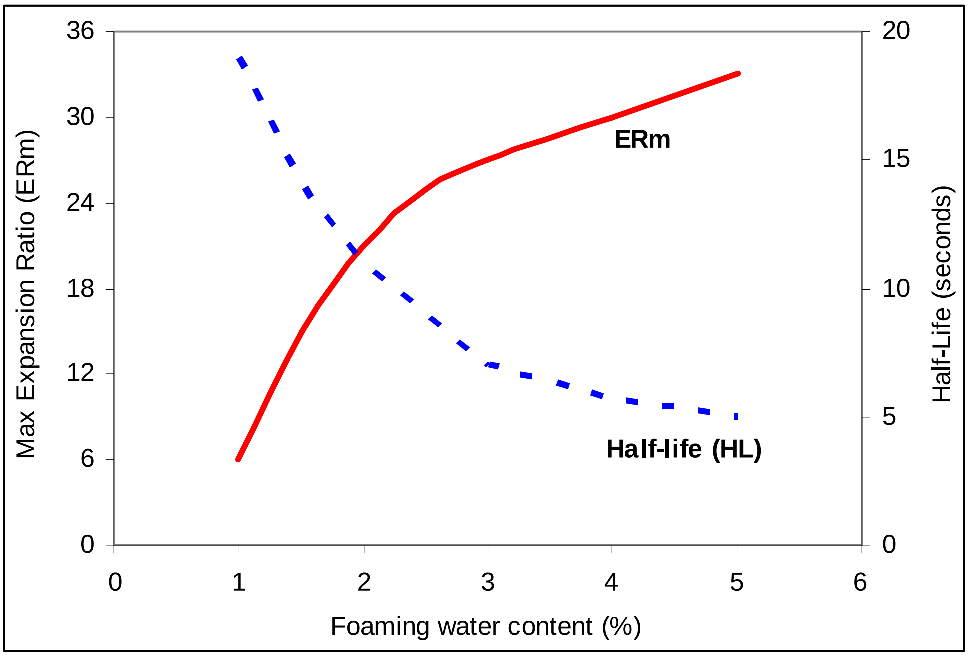


Figure 2.19 - Characteristics of foamed bitumen in terms of maximum expansion ratio and half-life

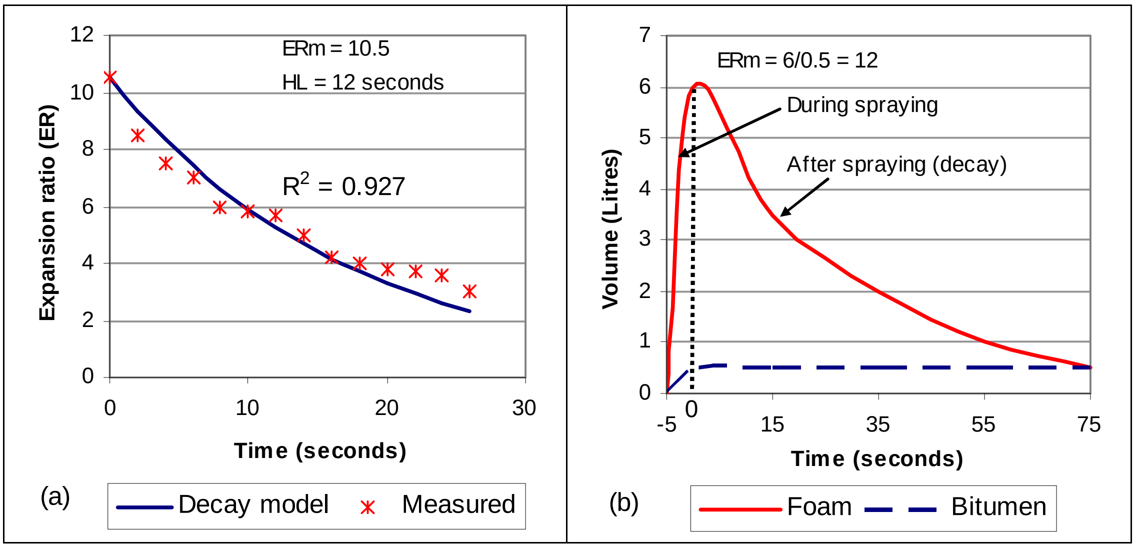


Figure 2.20 - Calibration of foam decay model (a) and curve of measured foam decay (b) (after Jenkins, 1999)

2.4.1.2 Foam decay and the foam index (FI) concept

Jenkins (1999) considered foam decay as another important factor of foamed bitumen properties. Foam decay is the collapse of foamed bitumen with time. Reduction of foam temperature due to contact of the bubbles with ambient air (steel container or aggregates) at lower temperature is one of the factors affecting foamed bitumen decay.

Jenkins, therefore, developed a foamed bitumen decay model by adapting equations for isotope decay as shown in Eq. 2.3. The isotope decay equation with respect to time is: Ln x = -kt. The negative sign in this equation indicates that the x values decrease with time. Thus, the k value is: -Ln x ÷ t. At t = HL the x value is a half of the initial value (ERm); therefore the k value can be written as: k = Ln (ERm÷0.5ERm) ÷ t or Ln2÷HL.

$$ER(t) = ERm^{*}\, e^{\frac{-Ln2}{HL}*t} \qquad \textbf{… Eq. 2.3}$$

where,

ER(t) = Expansion Ratio with respect to time after foam discharge

ERm = Maximum Measured Expansion Ratio (immediately after discharge)

HL = half-life (seconds)

t = time measured from the moment all foam is discharged (seconds)

The decay model (Eq. 2.3) has been statistically calibrated with measured foam decay giving a correlation coefficient of 0.927 (see Figure 2.20a). In most cases, the bitumen has been decaying for up to 5 seconds before the expansion ratio is measured, see Figure 2.20(b) i.e. the maximum expansion ratio measured, ERm, is not the actual maximum expansion ratio, ERa, of the foam; or ERm≠ERa. Using the foam decay relationship incrementally on foamed bitumen during discharge from the spray nozzle, the actual maximum expansion ratio ERa required to yield the measured maximum expansion ratio ERm in the laboratory can be back-calculated. It is not possible to measure the actual expansion ratio due to the decay during discharge; but it is possible to back-calculate it.

The relationship between ERa and ERm is shown graphically in Figure 2.21. Given t_s (time of spraying of the foamed bitumen) and HL (half-life), the correction factor c (=ERm/ERa) can be used to obtain the actual expansion ratio (ERa) from the measured expansion ratio (ERm).

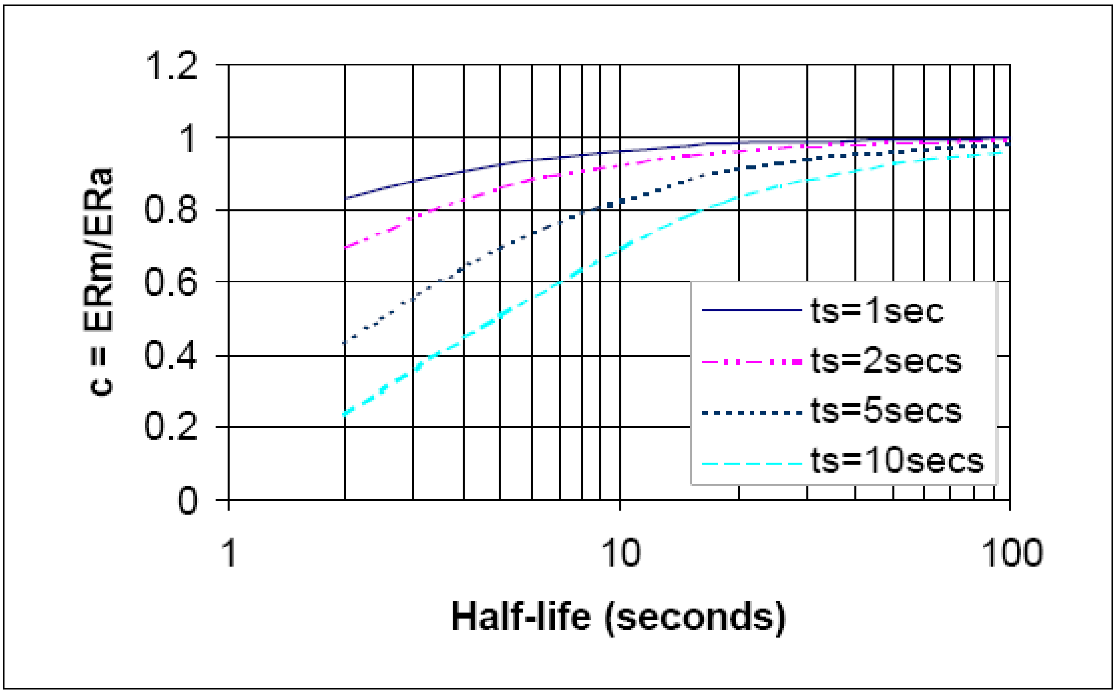


Figure 2.21 - Relationship between actual and measured maximum expansion ratio (Jenkins, 1999)

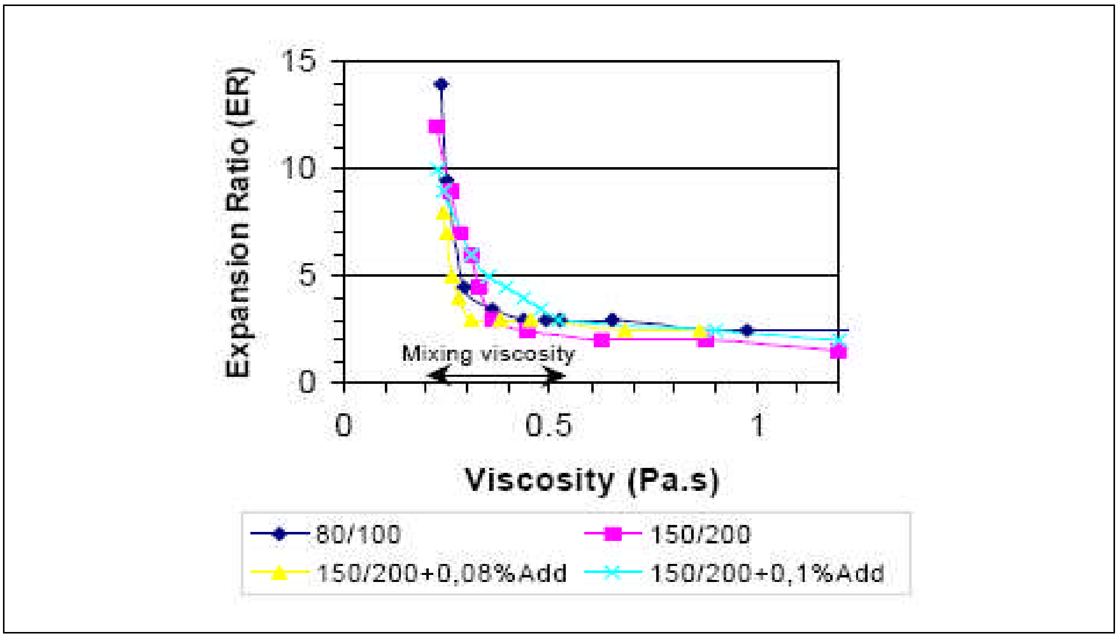


Figure 2.22 - Viscosity of foamed bitumen at different expansion ratio levels measured using a hand-held viscometer (Jenkins, 1999)

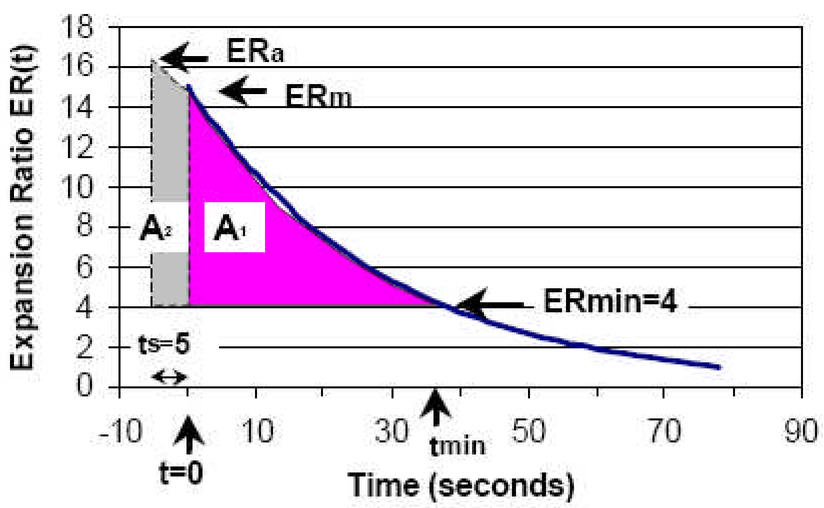


Figure 2.23 - The Foam Index (FI) calculation for a given foaming water content, where FI = A1 + A2 (Jenkins, 1999)

Considering that acceptable mixing takes place at viscosities between 0.2 and 0.55 Pa.s (Read and Whiteoak, 2003), Jenkins (1999) argued that the expansion ratio of the foam should at least be ER = 4 for adequate mixing of all foamed bitumens (see Figure 2.22). This value is then utilised as the minimum value for calculating the area under the curve (Foam Index value), see Figure 2.23.

Eq. 2.4 shows the calculation of the Foam Index (FI) as the sum of A_1 and A_2 (Figure 2.23).

$$FI = A_1 + A_2$$
$$= \frac{HL}{\ln 2}\left(4 - ERm - 4\ln\left(\frac{4}{ERm}\right)\right) + \left(\frac{1+c}{2c}\right) * ERm * t_s \qquad \textbf{...Eq. 2.4}$$

where

FI = Foam Index

HL = Half-Life

ERm = Maximum Expansion Ratio

c = Correction factor (see Figure 2.21)

t_s = spray time

As an example, Table 2.4 shows a comparison of foam index values of standard and non standard bitumen in accordance with CSIR (Muthen, 1999). The result indicates that the FI approach may be more sensitive in assessment of foam quality than CSIR specification.

Table 2.4 - Foam index of standard and non standard bitumen in accordance with CSIR (Muthen, 1999)

	Bitumen type-1	Bitumen type-2
ERm	15	10
HL (seconds)	10	12
$c = ER_m/ER_a$	0.83	0.86
FI (seconds)	165.1	94.5
Note: The bitumen type-1 would be discarded by CSIR specification since HL < 12 seconds. However this bitumen results in higher FI which shows a better foaming performance than the standard bitumen. A bitumen with a higher FI is able to store more energy in the foam whilst temporarily in the mixing viscosity range, than a bitumen with a lower FI.		

2.4.1.3 Foamed bitumen viscosity

Jenkins (2000) and Saleh (2006a) have reported the results of foamed bitumen viscosity investigations. Jenkins's investigation was to find the minimum foam expansion ratio related to the acceptable mixing viscosity, whereas Saleh's investigation was to find the optimum foam properties based upon their minimum viscosity.

As shown in Figure 2.22, Jenkins found that the foam viscosity decreased with increasing expansion ratio (ER). It can be seen that as ER decreases from about 15 to 5 the viscosity increases slightly; however the change of viscosity is sharp below ER of 5. It should be noted that decreasing ER in decaying foam is accompanied by decreasing foam temperature; hence increasing viscosity should be also affected by decreasing temperature. In general, during the first 60 seconds, foam viscosity will be in the range 0.25 Pa.s to 0.5 Pa.s. Jenkins measured foam viscosity by immersing the spindle of a hand-held viscometer in decaying foam. This means the measurement was conducted soon after the foam started to collapse, when its temperature was reducing. Each curve shown in Figure 2.22 is the result of a single foam (one FWC application), however bitumen temperature and foaming water content were not stated.

Saleh measured foam viscosity using a Brookfield rotational viscometer. The foam was directly collected and measured at several times over a period of 320 seconds, and an average foam viscosity over the first 60 seconds was calculated as shown in Figure 2.24. It appears that the trend line was developed over the measured viscosity values to enable an adjustment to be made to the average values over the first 60 seconds. The results were in line with the Jenkins's results as shown in Figure 2.22, in that for single foam during the decaying process, when ER and foam temperature are decreasing, foam viscosity increases with elapsed time. Saleh found that viscosity of foamed bitumen during the first 60 seconds was in the range of 0.2 to 0.6 Pa.s (for foam generated using bitumen Pen 80/100 and FWC 2.5%).

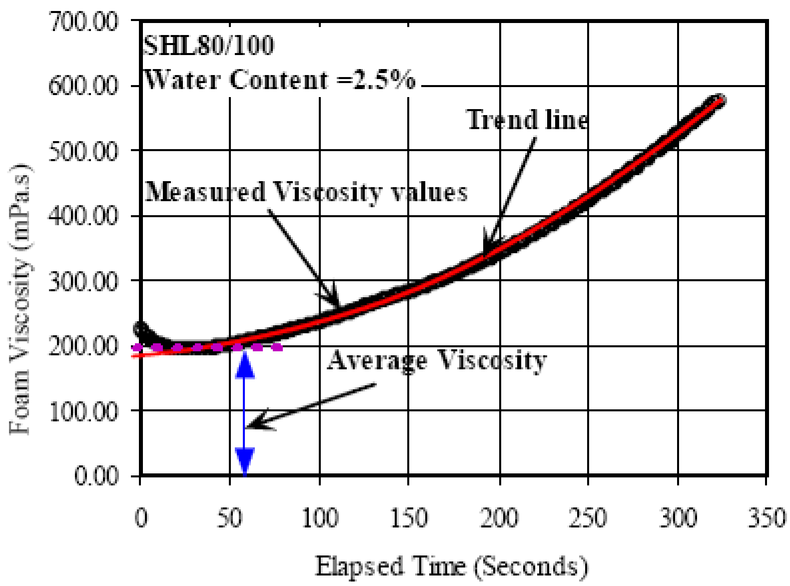


Figure 2.24 - Foamed bitumen viscosity measured using a Brookfield rotational viscometer against elapsed time for single foaming water content application (Saleh, 2006a).

All the average viscosities of foams generated using selected bitumen grades and temperatures at different FWC have been identified. The minimum viscosity value in the full range of FWC was selected as the best foam composition. It was thus proposed that the ability of a foam to form a well coated asphalt mixture was directly related to its viscosity. It was found that foam viscosity decreased to a minimum of 190mPa.s when FWC was at 2.5%. As FWC increased from 2.5% up to 4.5% the viscosity increased up to 900 mPa.s.

2.4.1.4 Determining the best foamed bitumen quality

A bitumen foam that achieves a higher ERm and a longer HL is currently understood to be of better quality and to result in better asphalt mix properties. Unfortunately, ERm and HL values show opposite trends and this makes the selection of an optimum foam difficult.

From experience, effective foam usually has a maximum expansion ratio between 10 and 15, which may be produced by injection of between 1 and 3 percent cold water. Various empirical guidance have been proposed as shown in Table 2.5 to limit ERm and HL and a method to select the optimum foam properties was introduced by Wirtgen (2005) as shown in Figure 2.25. This method is more likely to be useful for field applications. But as a research tool, selecting the best foam quality based on the mid point FWC (between minimum ERm and HL values) is not very rigorous due to these parameters not being linearly related.

Table 2.5 - Minimum application limit of ERm and HL

Method	Minimum ERm	Minimum HL
CSIR (Muthen, 1999)	10	12 seconds
TRL Report 386 (Milton & Earland, 1999)	10	10 seconds
Wirtgen (2005)	8	6 seconds
Chiu and Huang (2002)	8	8 seconds

It was for this reason that Jenkins (2000) introduced the Foam Index (FI) as a function of ERm and HL (see section 2.4.1.2). Higher ERm and longer HL result higher FI. Foamed bitumen quality could therefore be characterised using single parameter, i.e. the FI value. Foam with higher FI was understood to have better properties. The optimum foam characteristics could therefore be obtained from the full range of FWC for any one bitumen type at one temperature (Figure 2.26).

However, the FI concept can not be applied for all bitumen types. With increasing ERm values bitumens do not always show continually decreasing HL values, as found by Lesueur et al. (2004), He & Wong (2006) and Saleh (2006a), which means there is no clear definition of optimum FWC, FI values increase continually with FWC as shown in Figure 2.27.

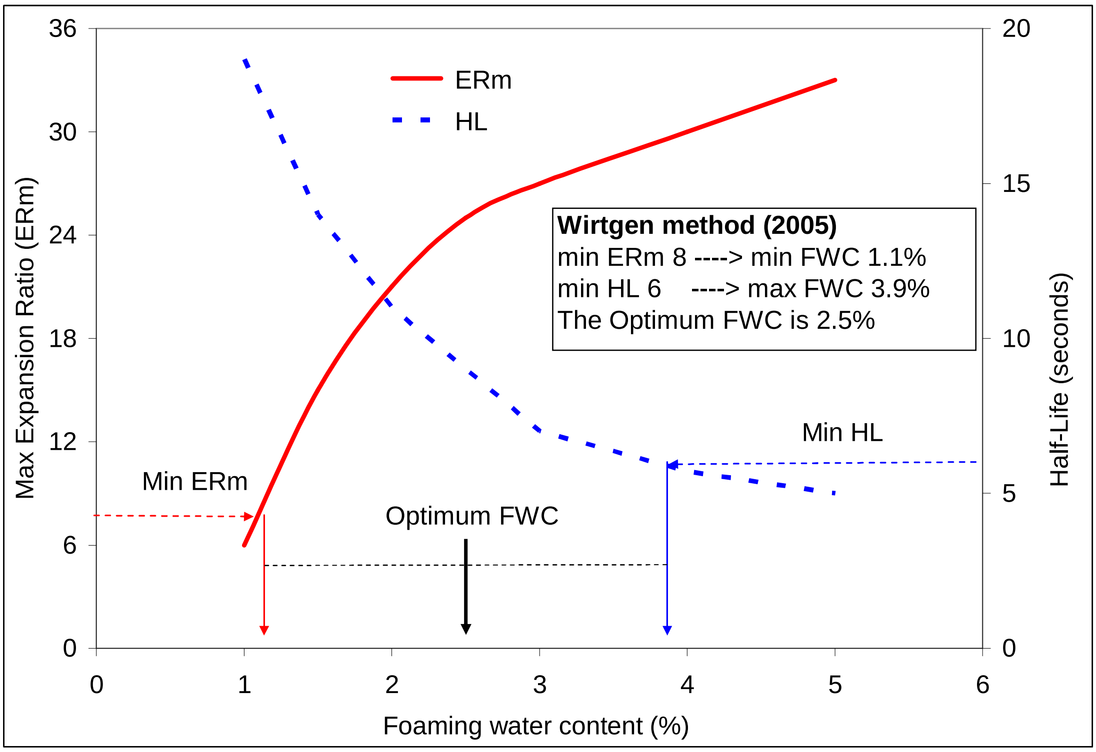


Figure 2.25 - The Wirtgen method to select the best foam quality (Wirtgen, 2005)

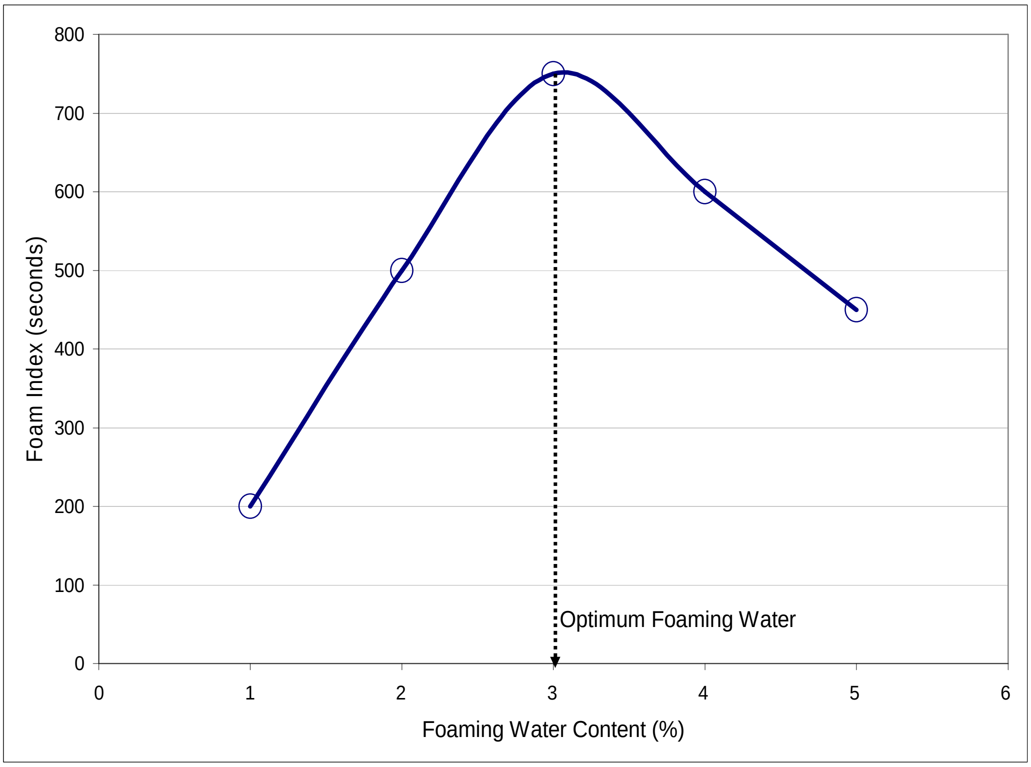


Figure 2.26 - Optimisation of foamed bitumen characteristics using foam index (FI) concept (Jenkins, 2000).

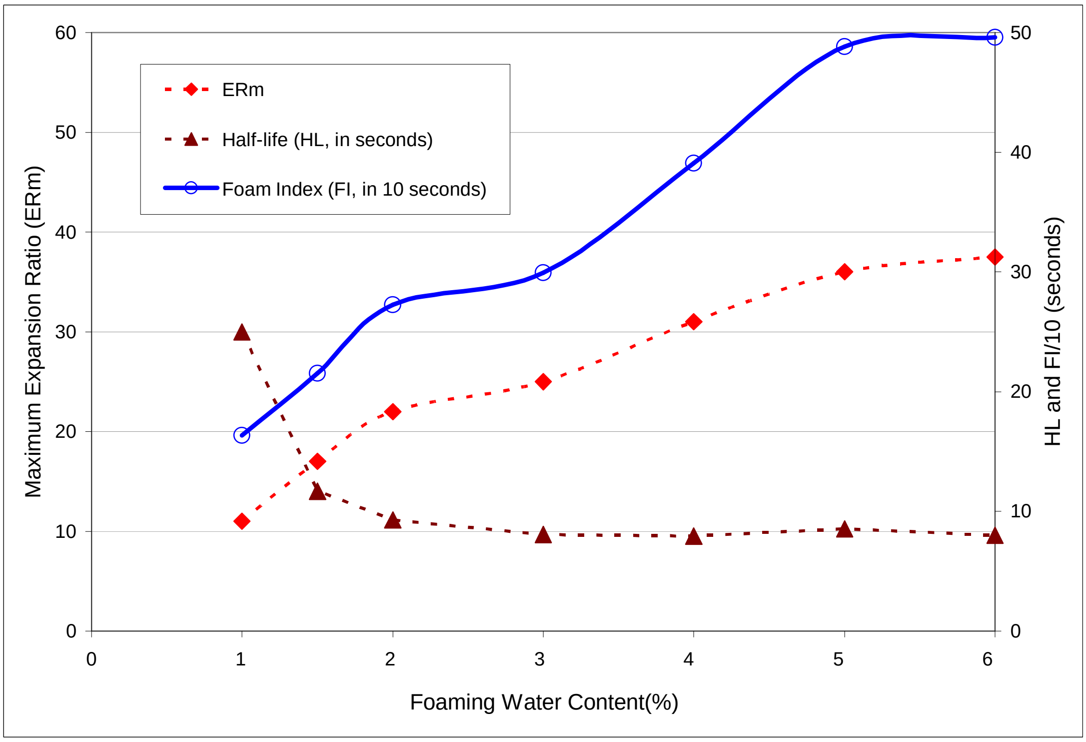


Figure 2.27 - A case of foamed bitumen characteristics with no optimum FI value.

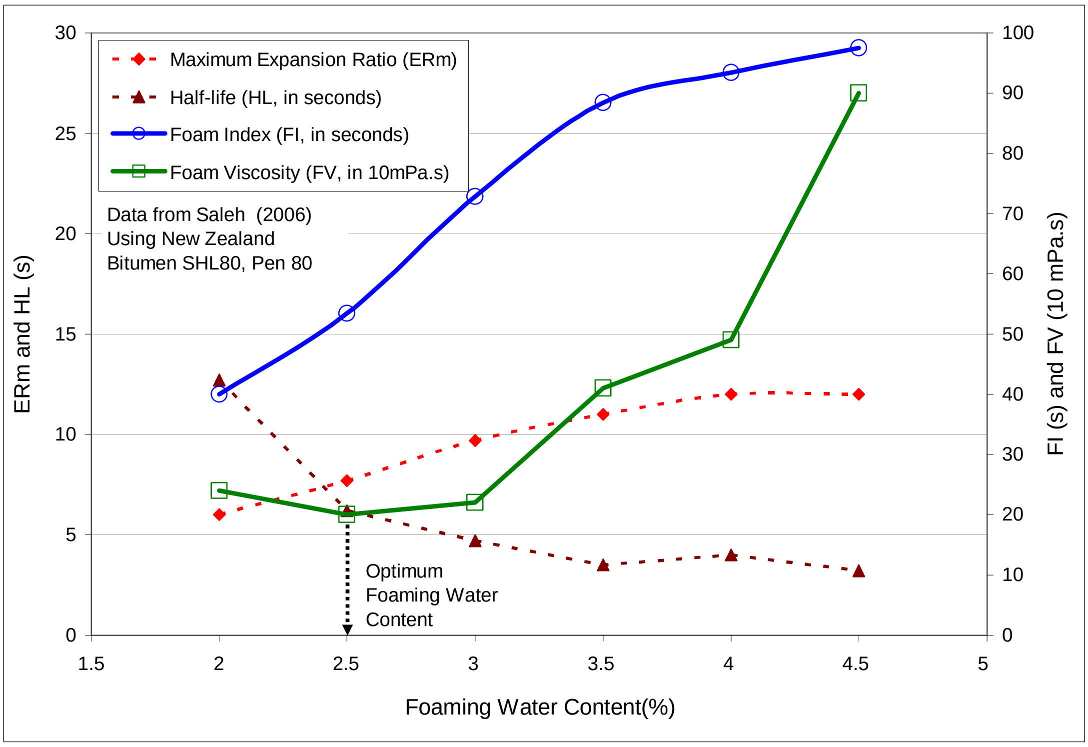


Figure 2.28 - Optimisation of foamed bitumen characteristics based on viscosity value (data from Saleh, 2006)

As noted already, Saleh (2006a) proposed an alternative method to measure foam viscosity, and he proposed that it could be used to select the optimum foam properties. The relationship between FWC and foam viscosity was developed and the minimum viscosity value was selected as the best foam composition (Figure 2.28). Saleh found that foams that were categorised as poor performing, according to their FI values, could still be effectively mixed with aggregates. It was thus proposed that the ability of a foam to form a well coated asphalt mix was directly related to its viscosity.

2.4.2 Foamed asphalt mixture properties

2.4.2.1 Classification of foamed asphalt materials

In the TRL Report TRL386, Milton and Earland (1999) defined two families of in-situ stabilized materials: (1) mixtures where the primary binder was Portland cement and (2) mixtures where the primary binder was foamed bitumen. A design method and specification for each family was therefore provided separately.

Merill et al (2004) defined material families based upon the characteristics of the stabilizing agents. Materials bound with Portland cement are expected to cure more quickly than materials with other types of hydraulic binder, and viscoelastic materials are likely to be less prone to shrinkage cracking than hydraulically bound materials. Therefore a new classification for cold recycled materials was introduced and it was published in the TRL Report TRL611 (2004) as shown in Figure 2.29. The apexes of this diagram correspond to fully hydraulically bound, fully visco-elastically bound and unbound material. Materials using combinations of binder and curing behaviour can be characterized by areas within this chart. The four material types that fall into three material families are as follows:

- Quick hydraulic (QH) with hydraulic only binder(s) including cement;
- Slow hydraulic (SH) with hydraulic only binder(s) excluding cement;
- Quick viscoelastic (QVE) with bituminous and hydraulic binder(s) including cement;
- Slow viscoelastic (SVE) with bituminous only or, bituminous and hydraulic binder(s) excluding cement.

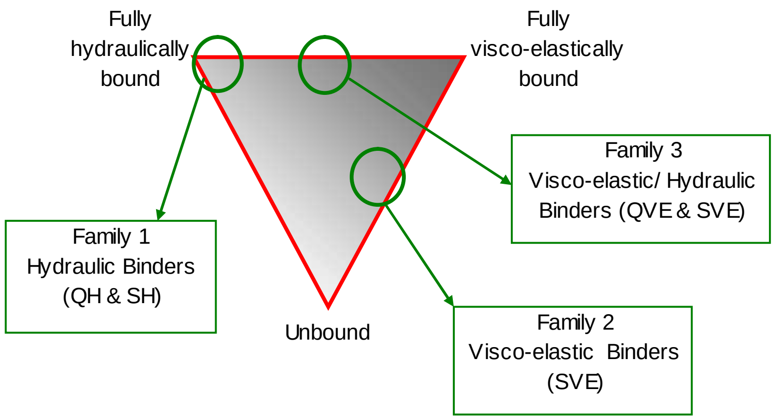


Figure 2.29 - Cold recycled materials (after Merill et al, 2004)

Based upon this new classification, foamed bitumen bound materials are expected to have a degree of viscoelastic behaviour and consequently may fall within one of the viscoelastic families (SVE or QVE).

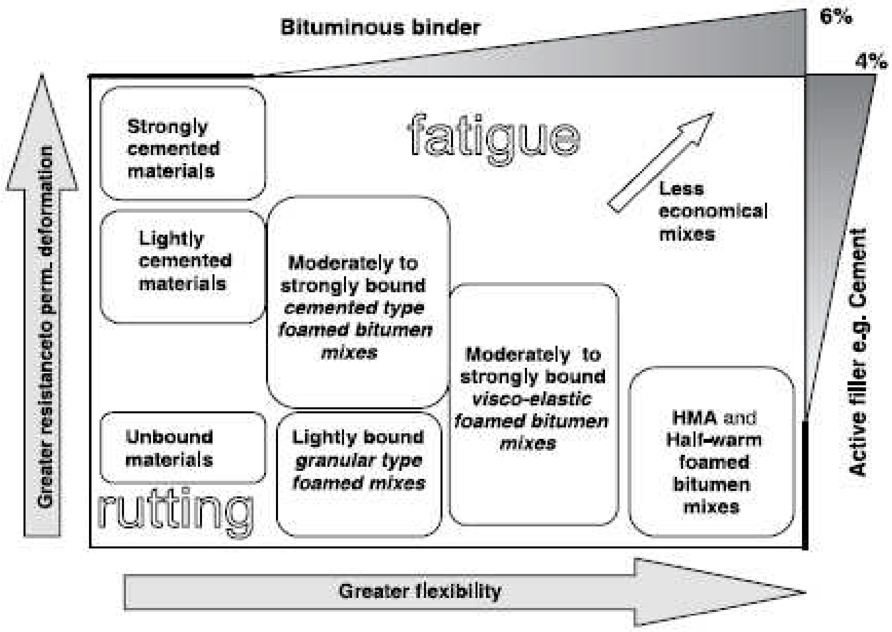


Figure 2.30 - Type of foamed bitumen mixtures (after Asphalt Academy, 2002)

In line with TRL Report TRL611, the Asphalt Academy (2002) has also published a schematic including foamed bitumen mixture types as shown in Figure 2.30. As shown in the scheme, foamed asphalt materials are put relatively in the middle of unbound/ cement bound (left side) and HMA (Hot Mix Asphalt, right side). Foamed asphalt properties seem to vary from weak to moderately strong materials, depending on the granular type and active filler content. At the same time, these properties also vary from low to high resistance to permanent deformation. It is supposed that a mixture using big stone (nominal size more than 25mm) and high coarse particle content will potentially result in a lightly bound material and hence exhibit low strength and low resistance to permanent deformation. When active filler e.g. cement is added to this mixture, the strength will increase moderately. In addition, when binder content is increased, the mixture exhibits more flexibility and hence greater resistance to fatigue. The HMA and Half-warm mixtures are put at the bottom right; this does not mean that these mixtures have low resistance to permanent deformation, but that they normally use high binder content without active filler.

2.4.2.2 Mechanical properties of foamed asphalt

As is common for cold-mix asphalts, the strength of foamed asphalt mixtures (FAM) at early life develops with loss of moisture (Bowering, 1970). In a pilot scale project (Nunn and Thom, 2002), foamed bitumen bound materials at very early life exhibited stiffness typical of unbound material when their moduli were investigated using a Dynamic Plate tool. Based on Falling Weight Deflectometer (FWD) data, the stiffness at 20^{o}C of the foamed asphalt layer was found to increase from <1000MPa (at early life) to 3500MPa (at one year). The mixture developed to gain satisfactorily high stiffness levels within 6 months.

For a particular mixture, there is an optimum foamed bitumen content (OFBC) at which the ‘strength’ of the mixture is a maximum. The associated strength has been evaluated in terms of unconfined compressive strength or UCS (Bowering, 1970), resilient modulus under the repeated load triaxial test (Shackel et al, 1974), Marshall stability and ITS (Kim and Lee, 2006) and indirect tensile stiffness modulus (Nataatmadja, 2002). However, Jenkins et al (2004) found that the OFBC can not be clearly identified in the range of bitumen content from 1.5% to 3.8% for test carried out under both dry and wet condition. The investigators gave an explanation that variation of moisture content and foam characteristics used can significantly affect the test results. The observed foaming properties (generated using 80/100 bitumen) fluctuated between 6 and 12 seconds for HL and between 15 and 24 for ERm.

Unfortunately, the investigators did not report the temperature of the bitumen during the foaming process.

Temperature sensitivity of FAM has been investigated by Fu and Harvey (2007) in terms of triaxial resilient modulus (Mr) and Nataatmadja (2002) in terms of indirect tensile stiffness. The temperature sensitivity of FAM and HMA might be similar in that they are dependent upon the binder rheology, but their micro-structures and coating details are different. The results of both investigators are relatively similar; increasing the temperature by 10^{o}C resulted in the modulus reducing by 12-15% (tensile stiffness) or 9-15% (Mr) at binder contents around 1.5 – 4.5%, with the highest stiffness having a greater sensitivity. In addition, Fu and Harvey also investigated the effect of stress state on Mr values. It was found that increasing Mr due to the confining stress was more significant than due to the deviator stress. This stress sensitivity reduced with increasing temperature. Jenkins et al (2004) also found that the Mr increased with increasing sum of principal stress, whilst, Nataatmadja found that the indirect tensile stiffness decreased with increasing strain level. The strain sensitivity was greatest at OFBC (or at highest stiffness). Moreover, Acott (1979) reported that resilient modulus of foam treated sand mixtures (determined using the repeated load indirect tensile test) was affected not only by stress and temperature but also by loading rate. The moduli were found to increase with loading rate and decrease with stress and temperature.

The use of RAP reduces the strength of a mixture according to unconfined compressive strength and indirect tensile strength data (Loizos et al, 2004), and indirect tensile resilient modulus test data (Ruenkrairergsa et al, 2004). It may be the presence of old binder reduces the interlocking between aggregate particles and hence reduces the mixture strength. It has also been found that RAP reduces the resistance to fatigue and permanent deformation of a mixture (Ruenkrairergsa et al, 2004). Different features were reported by He and Wong (2008), in that the RAP content did not significantly affect the permanent deformation susceptibility of FAM. The RAP proportion varied from 0% to 60% and the stabilized mixtures were tested under the repeated load axial test with an applied stress of 100kPa at a temperature of 30^{o}C.

Shackel et al (1974) investigated the influence of bitumen grade. The use of a low bitumen grade (90pen) was found to give a material which responded better to repeated loading than using high bitumen grade (200pen), but the effect under repeated load was less than under monotonic load. In addition, He and Wong (2007) reported that mixtures (using RAP) stabilized by 100pen bitumen have better strength (ITS) and resistance to permanent deformation (RLAT) than with 60pen under both dry and soaked conditions. The effect of bitumen grade on foaming characteristics could explain the whole role of bonding mechanism in the mixture. Merill et al (2004) suggested that the choice of bitumen grade is a compromise between foaming ability and stiffness; higher grade bitumen foams easily but has lower viscosity.

Lee (1981) and Bowering & Martin (1976a,b) have reported the effect of foamed bitumen characteristics on mixture properties. Lee (1981) has investigated the effect of ERm and HL on the characteristic of Marshall Stability and flow of foam mixture. Fine sand aggregate (nominal size 1.18mm) was mixed with 4% foamed bitumen Pen 200/300. The foams covered an ERm range of 5 to 20 and a HL range of 11 to 136 (some using anti-foam counter agent). Unfortunately, no information was given on what temperature, foaming water content and mixing protocol have been used. Figure 2.31 shows the data resulting from Lee's investigation. Lee stated that the data revealed no significant trends and the highest stability (after 24 hr immersion at 60^{o}C) was obtained at an ERm of 15 or HL of 18 seconds. However, it is noted that two samples with ERm of 15 (see points circled in figure) resulted lower stability. When these points are ignored, it seems possible to state that the optimum value of ERm can be obtained at ERm= 15; more data is however required to confirm this conclusion. The effect of ERm on mixture properties is likely to be more dominant than HL. In addition, Bowering & Martin (1976) reported that foam at ERm=15 gave better properties of mixture than at ERm=3, in which the properties were evaluated using Marshall Stability, UCS, resistance value, cohesion and swelling. These findings indicate that increasing ERm results in better mixture properties, but they did not test with ERm higher than 15.

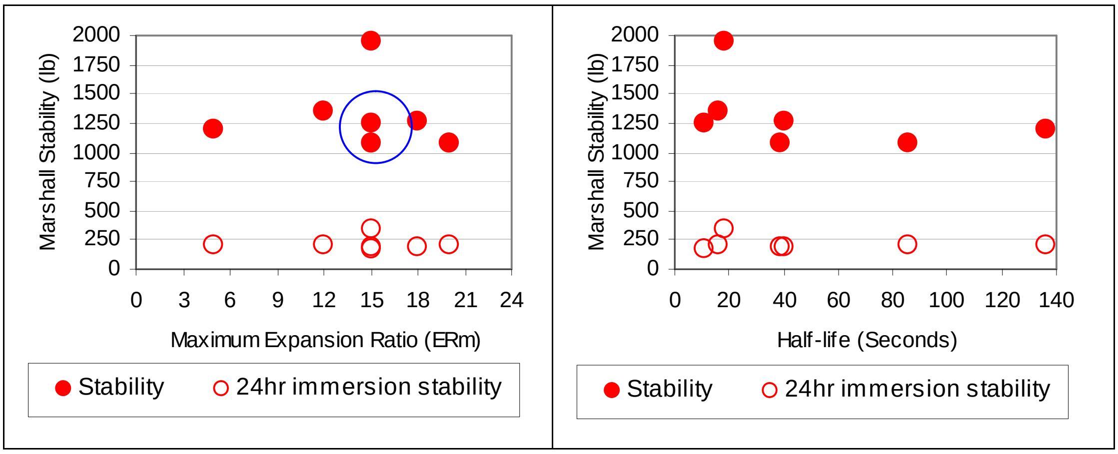


Figure 2.31 - Effect of foamed bitumen characteristics on Marshall Stability (data from Lee 1981)

Table 2.6 - Effect of bitumen/ foam properties on Marshall stability of foamed asphalt mixture (data is adapted from Bissada, 1987)

Property	Type of bitumen		
	AC 20	AC 25	VAR
Penetration at 25°C (0.1mm)	67	135	310
Softening point (°C)	51	45	36
Viscosity at 165°C (mPa.s)	120	80	50
Specific gravity	1.030	1.020	1.005
Foam generated at 165°C, FWC 2%, air pressure 200kPa			
ERm	9	11	13
HL (seconds)	8	18	22
Marshall stability (kN) after curing at 40°C for 3 days	Standard value (Immersion value)		
At FBC= 4.5%	11.0 (9.4)	11.4 (9.0)	14.0 (11.3)
At FBC= 5.5%	8.4 (7.4)	9.3 (6.7)	10.4 (7.8)
At FBC= 6.5%	6.1 (5.8)	7.9 (6.1)	9.5 (7.0)
Air voids (%)			
At FBC= 4.5%	16.4	15.9	15.4
At FBC= 5.5%	15.7	15.1	15.4
At FBC= 6.5%	15.1	14.6	15.1

Note: VAR= Vacuum asphalt residue FWC= Foaming water content
AC= Asphalt cement FBC= Foamed bitumen content
Standard value= 0.5 hour in water at 60°C
Immersion value= 24 hours in water at 60°C

Table 2.6 presents the relationship between bitumen viscosity (before foaming), bitumen grade, foam characteristics and foamed asphalt mixture. The data is adopted from Bissada (1987). The aggregate used is sand with maximum size 5mm. The MDD and OMC of the sand are 2062 kg/m^3 and 9.4% respectively. Specimens were compacted using a Marshall Hammer with 2x50 blows applied. It can be seen that the softer bitumen or the lower viscosity at foaming result in better foam characteristics. Actually, the difference in their expansion ratio (between 9, 11 and 13) is not too significant; however the HL with bitumen AC25 and VAR were significantly higher than with bitumen AC20. The results show that the mixtures made with AC 20 give the lowest stability values. The investigator reported that problems such as stickiness and lumping were encountered with the AC 20 mixture but were not observed in other mixtures. Visual examination of the foamed mixtures revealed that the VAR binder with the lowest viscosity, exhibited the best aggregate particle coating and the most uniform dark colour. Mixtures that contained FBC 4.5% were light in colour. Increasing the FBC to 6.5% resulted in the appearance of several balls of uncombined mixture. So, it is clear that the poor stability of mixtures with lower penetration is due to deficiencies in their foam characteristics and hence their mixing properties. It is noted that the lack of mixing properties was not accompanied by any lack of compaction since their air voids were not too different.

Table 2.7 - Minimum acceptable criteria for foamed asphalt materials.

Minimum acceptable value	Property	Reference
2500 MPa	ITSM (rise time 124ms, at 20^{o}C)	Nunn and Thom (2002)
6000 MPa (dry condition) 1500 MPa (wet condition)	ITSM (rise time 50ms, at 25^{o}C)	Lancaster et al (1994)
0.5 MPa (4 days soaked) 0.7 MPa (3 days cured at 60^{o}C)	UCS	Bowering (1970)
200 kPa (dry condition)	ITS	Bowering and Martin (1976b)
100 kPa (wet condition)	ITS	Maccarone et al (1995)
3.5 kN 1.5 kN/mm	Marshal stability Marshal Quotient	Akeroyd (1989)

The end product performance of foamed asphalt mixtures has been evaluated by many investigators. Nunn and Thom (2002) suggested that a long-term equilibrium stiffness of 2500 MPa (at 20^{o}C) is achievable after an intensively observed pilot scale trial in which many aggregates types such as basalt, limestone, asphalt planings/ RAP, blast furnace slag and crushed concrete, including additives (e.g. cement, fly ash), were stabilized using foaming bitumen. It was found that the mixtures having high moisture content and high air voids content (low density) performed poorly. Table 2.7 provides minimum acceptable values for foamed asphalt materials from various researchers.

2.5 Laboratory Mixture Design for Foamed Asphalt

The principal objective of mix design for pavement materials is to obtain the material proportions that fulfill the structural and functional requirements of the in-service mixture. The gradation of aggregate and amount of bitumen should be combined economically to yield a mixture property. For this purpose, the Asphalt Institute (1988) suggested consideration of the following factors:

- The amount of bitumen needed to ensure adequate fatigue cracking resistance and durability.
- The mixture stability and stiffness to resist deformation due to traffic loading.
- The void percentage in the mix to allow slight compaction under traffic loading without flushing, bleeding, or loss of stability.
- Workability during mixing, placement and compaction.

2.5.1 Mixture design considerations

2.5.1.1 Foamed bitumen characteristics

Muthen (1999) stated that foamed bitumen characteristics play an important role during the mixing stage of foamed asphalt production. High expansion foamed bitumen is reported to have resulted in improved aggregate coating (Maccarone et.al., 1994) and high cohesion and compressive strength of mixture (Bowering and Martin, 1976). Understanding of foamed bitumen properties is aimed at enabling selection of the best foam quality – from the many variants - in which 3 important matters are addressed:

- good distribution in the mix - to ensure mixture homogeneity and flexibility,

- good coating of the aggregate – to ensure mixture stability and durability,
- long life time – to ensure workability during mixing and compaction.

2.5.1.2 Bitumen grade

Bitumen grade should be an important parameter for foamed asphalt mixture performance. However, understanding of the effect of bitumen grade on the mixture properties is still unclear due to lack of understanding of bitumen grade effect on foam characteristics. Foamed asphalt mixtures are definitely loading rate and temperature-dependent behaviour mixtures, indicative of visco-elastic binder activity (Muthen, 1999). The evidence that Lee (1981) did not find any difference between the measured properties of foamed asphalt mixtures produced with different grades of bitumen is probably related to the fact that much of the shear strength of foamed asphalt mixes is due to aggregate interaction rather than binder cohesion.

2.5.1.3 Foamed bitumen content (FBC)

Foamed bitumen content can be evaluated by optimizing mixture properties over a chosen range of FBC. However, in foamed-asphalt mixes the optimum FBC often cannot be as clearly determined as it can in the case of hot-mix asphalt. The optimum FBC is normally selected after considering both dry and wet conditions of specimens.

Table 2.8 - Foamed bitumen content (Ruchel et al, 1982)

% passing 0.075mm sieve	FBC (%)	
	Gravel (<50% passing 4.75mm sieve)	Sand (>50% passing 4.75mm sieve)
3-5	3	4
5-7.5	3.5	4.5
7.5-10	4	5
> 10	4.5	5.5

The ratio of binder content to fines content plays a significant role in foamed mixture stability. Table 2.8 is intended as a guide to select the appropriate binder content based on the fines content of the mix. Akeroyd and Hicks (1988) also proposed the use of a proportional binder-fines relationship to select the binder content, ranging from a binder content of 3.5% for 5% fines content to a binder content of 5% for 20% fines content. However this approach may not be applicable for all types of material, because of the varying binder absorption characteristics of fines which, in turn, depend on the source (parent) material. In general, a foamed asphalt mixture has lower bitumen content and higher voids content than a standard asphalt mixture (Abel, 1978).

2.5.1.4 Aggregate properties

Foamed bitumen can be used with a wide range of aggregate types from conventional high quality graded materials, reclaimed asphalt and granular materials to marginal materials such as those having a high plasticity index (Muthen, 1999). However, Little et al (1983) reported that foamed bitumen mixtures with marginal aggregates have low stabilities and poor fatigue performance in comparison with conventional hot mix paving material, although they may still be acceptable in base and sub-base layers. Table 2.9 shows the many types of aggregates used for foamed asphalt and range of binder content adopted, from Bowering and Martin (1976b).

Table 2.9 - Type of Aggregates used for Foamed Asphalt

Material type	Binder content range (%)	Additional requirement
Well graded clean gravel	2 – 2.5	-
Well graded marginally clayey/ silty gravel	2 – 4.5	-
Poorly graded marginally clayey gravel	2 - 3	-
Clayey gravel	4 – 6	Lime modification
Well graded clean sand	4 - 5	Filler
Well graded marginally silty sand	2.5 – 4	-
Poorly graded marginally silty sand	3 – 4.5	Low pen bitumen; filler
Poorly graded clean sand	2.5 - 5	filler
Silty sand	2.5 – 4.5	-
Silty clayey sand	4	Possible lime
Clayey sand	3 – 4	Lime modification

With regard to aggregate grading, Akeroyd and Hicks (1988) introduced the Mobil foam stabilization grading chart (Figure 2.32) as a guidance to determine a suitable aggregate grading. The chart suggests that zones A and B are suitable for foam treatment for heavily trafficked roads and lightly trafficked roads respectively. Material that conforms to zone B will be more suitable if coarse materials are added.

However, materials within zone C are very coarse and hence unsuitable for foamed asphalt materials.

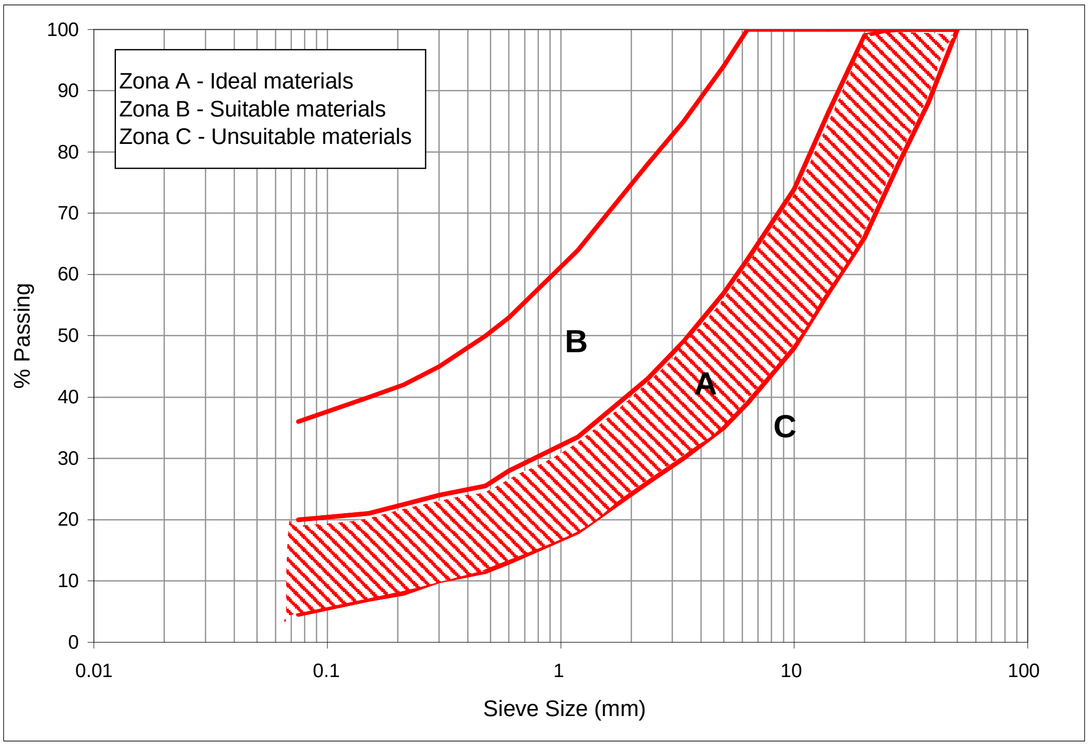


Figure 2.32 - Aggregate grading zones for foamed asphalt (Akeroyd and Hicks ,1988)

A relatively large fines fraction (less than 0.075mm) with low plasticity will produce good mix characteristics because foamed bitumen tends to coat the fines and only partly coat the coarse particles (Abel, 1978). The fines content makes a dominant contribution in determining the tensile strength (Bissada, 1987), stability (Sakr and Manke, 1985) and workability of foamed asphalt mixtures (Semmelink, 2001). Ruckel et al (1982) recommended using at least 5 percent of fine aggregate (< 0.075 mm). The mixture of foamed bitumen and fine aggregate will act as a mortar between coarse aggregate particles and hence cause increased strength. The ratio of foamed bitumen to fine aggregate should be controlled because excessive foamed bitumen will tend to act as a lubricant and to cause decreasing strength and stability.

Aggregate interlock has been found to be more important than binder viscosity in building mixture stability (Sakr and Manke, 1985). This may imply that foamed asphalt mixes are not as temperature susceptible as hot mix asphalts. This can be achieved using low bitumen content in which the binder coats the fine aggregate only. This therefore suggests that coarse aggregate interlock, mortar properties, and their mixture together will mainly drive overall mixture properties.

2.5.1.5 Moisture content

Moisture content of foamed asphalt is the most important mixture design criterion due to its significant effect during mixing, compaction and during in-service life. Moisture is required to aid foamed bitumen dispersal and to coat the aggregate during mixing. Moisture also acts as a lubricant during compaction to achieve its maximum density. On the other hand, moisture is the main factor affecting lack of stiffness at early age.

Low moisture content causes inadequate foam dispersal in the mix, whereas too much water will significantly reduce the strength and extend the curing time. Therefore, moisture content should be close to the optimum moisture content for compaction. The moisture needed is dependent on compaction level used - the higher the level the lower the moisture, and filler content (Maccarone et al, 1994). The aggregate moisture content may influence the bitumen content used in the mixture. Brennen et al (1983) found that if the amount of aggregate moisture decreases the bitumen content should increase.

Several versions of moisture content determination method have been proposed as following.

- **Fluff point method**. The Fluff point, developed by Mobil Oil, is the moisture content at which the aggregate has a maximum loose bulk volume. Brennen et al (1983) found that this point is close to 70-80% modified Proctor optimum moisture content (OMC).
- Water application at 65-85 % of the modified Proctor OMC (Lee, 1981 and confirmed by Bissada, 1987).
- **Total fluid content concept**. This concept is borrowed from emulsion mixture design in which the sum of the water and bitumen content should be close to OMC. Castedo-Franco and Wood (1983) agreed with this concept.

- **Sakr and Manke (1985) concept**. This concept uses mixing moisture content (MMC), depending on the modified Proctor OMC, percentage of fines (PF) in the aggregate and the bitumen content (BC) as described in Eq. 2.5.

$$MMC = 8.92 + 1.48\ OMC + 0.4\ PF - 0.39\ BC \qquad \text{…Eq. 2.5}$$

- **High moisture content method**. Jenkins et al (2002) applied this method. Because low moisture content produces a poor quality mix, therefore the moisture content was set at a high level (around 1-2 % higher than OMC). After the mixing process the material was air dried for 30 minutes to achieve the optimum fluid content (OFC) necessary for compaction.
- **Wirtgen concept**. This method uses optimum compaction and workability content (OCC) which is equal OMC reduced by a reduction water content (WC_{reduc}) that depends upon that OMC value (Eq.2.6).

$$OCC = OMC - WC_{reduc}. \qquad \text{Eq. 2.6}$$

Where, WC_{reduc}. = 0 (for OMC less than 2%)

WC_{reduc}. = 0.3*OMC – 0.6 (for modified Proctor OMC)

WC_{reduc}. = 0.4*OMC – 0.8 (for standard Proctor OMC)

2.5.1.6 Mixing

Foamed bitumen should be mixed with aggregates immediately since foam will collapse in a few seconds after spraying. Therefore the mixing method is very important to produce mixes which are both homogeneous and closely representative of those produced in the field. According to Lee and Kim (2003)'s study of the comparison between foam blending mixed in the field and the laboratory, field mixing results in better foamed asphalt properties than laboratory mixing. Therefore, Long et al (2004) recommended the use of a high-speed twin shaft pugmill mixer (developed by CSIR Transportek) to produce comparable materials with the field.

Mixing time was considered to be the approximate time needed for the foam to completely collapse after spraying. In the laboratory, this normally means about one minute (Bissada, 1987).

2.5.1.7 Compaction

Two kinds of laboratory compactor are commonly used for foamed asphalt, namely:

- Marshall hammer
- Gyratory compactor

The Marshall hammer can be used to prepare compacted specimens for testing, not only for stability and flow tests in the Marshall apparatus, but also for indirect tensile strength, unconfined compressive strength and indirect tensile modulus tests. About 1200 grams of foam-aggregate blend is poured into the Marshall mould and then compacted for 2x75 blows.

In recent years the gyratory compactor has gained popularity for the preparation of samples. The mass of loose mixture is calculated based on its density to obtain a particular height appropriate to the height requirement of the test. The loose material is poured into the gyratory mould and set in the gyratory compactor machine. The standard Superpave protocol can be used in which compaction is set to 30-gyrations, ram pressure 600 kPa and compaction angle 1.25 degrees. Jenkins (2002) found that applying 150 gyrations using the Superpave protocol will produce a specimen density approaching 100% modified Proctor compaction. It is recommended to observe the number of gyrations before compaction because this is affected by material type.

In spite of the density and indirect tensile strength (ITS) of gyratory compacted specimens being slightly higher than those compacted in the Marshall compactor (Lee and Kim, 2005), the gyratory compacted specimens exhibit lower stiffness due to changes in particle rearrangement (Nataatmaja, 2001).

2.5.1.8 Curing and temperature condition

One of the main differences between foamed asphalt mixture (FAM) and hot mix asphalt (HMA) is how they achieve their long-term characteristics after placement in the field. For HMA, the pavements will exhibit most of their long-term characteristics immediately after placing in the field. However, FAM needs a time period, namely the curing time, to gain its final characteristics due to the presence of water, both as aggregate moisture and foaming water.

Actually, the best way of curing in the laboratory is to cure the specimens at ambient temperature, comparable to field conditions. However, this will take a long time and is often not practical. Thus elevated temperature is usually used to accelerate the laboratory curing time, for example temperatures of 40^{o}C or 60^{o}C.

In the foamed asphalt process, some researchers have recommended to cure specimens at 60^{o}C for 3 days, being representative of a year's service after construction (Bowering, 1970; Maccarone, et al, 1995; Muthen, 1999). However, heating the specimen to 60^{o}C has an effect on binder ageing and changes its properties, so other researchers recommended applying 40^{o}C for 3 days (Jenkins, 2000) or 40^{o}C for 28 days (TRL 611, Merill et al 2004). This study selected a curing regime of 40^{o}C for 3 days in order to eliminate the effect of binder ageing and changes of specimen properties.

In determining optimum foamed bitumen content (OFBC) using ITS or UCS testing, applying curing temperatures of either 40^{o}C or 60^{o}C gives similar results (Lee and Kim, 2005). Lee and Kim also found that the loss of moisture during curing at 40^{o}C for 3 days, 60^{o}C for 2 days or 25^{o}C for 28 days is equal. It is therefore recommended to apply 40^{o}C for 3 days or 60^{o}C for 2 days in determining OFBC.

2.5.2 Mix design process

The following are 3 of the most useful design procedures currently proposed:

- Foamed asphalt mixes, mix design procedure – proposed by Muthen, K.M. (1999) and supported by SABITA Ltd and CSIR with the background of many South African projects.
- Developing of a mix design process for cold-in-place rehabilitation using foamed asphalt – proposed by Lee and Kim (2003).
- Foamed Bitumen Mix Design Procedure Using The Wirtgen WLB 10 – proposed by Wirtgen (2005).

Table 2.10 has been devised based on these 3 mixture design procedures and will be considered in this project.

Table 2.10 - Mixture design procedure for foamed asphalt

STEP 1: Determine the optimum foamed bitumen properties
- Investigate ERm and HL values for foams at various temperatures and foaming water contents for several chosen bitumen grades. - Select the best bitumen grade and temperature using the curve of ERm vs HL. - Select the best foaming water content using the curve of ER-HL vs foaming water content and selected foam specification (e.g. Wirtgen, 2005).
STEP 2: Prepare the aggregate
- Check the Plasticity Index (BS 1377-2: 1990), add lime 1% for high PI aggregate. - Check the gradation (BS EN 933-1:1997), ensure that gradation is within the specification envelope for foamed asphalt (e.g. Akeroyd and Hicks, 1988). Minimum filler content should be 5%. - Determine maximum dry density (MDD) and optimum moisture content (OMC) using a standard or a modified Proctor procedure (BS 13286-2: 2004). - Prepare 7.5 kg mass samples for each batch (need 3 to 5 batches), check initial moisture content (MCintial) using duplicate sample. - When using cement or lime, they should replace the equivalent percentage of mineral filler
STEP 3: Mixing process
- Calculate the amount of aggregate water required (% of total aggregate mass) (see the provided equations or concepts in section 2.5.1.5). - Select 3 to 5 values of FBC. Calculate the amount of foamed bitumen by % of total aggregate mass. Add 25% to calculated foam mass to cater for amount of foam lost during the mixing stage (depends on mixer agitator type). Set timer of foaming machine appropriate to foam mass required. - Add water to the aggregates first and mix for about 30-60 seconds. Introduce the foam and continue mixing for a further 1 minute. - Complete mixing the samples with selected FBCs.
STEP 4: Compaction process
- Compact the foamed blends using Marshall hammer or Gyratory compactor. - When using Marshall hammer, compact specimens 2x50 or 2x75 blows. - When using the Gyratory compactor, compact specimens with the Superpave standard protocol. Investigate the number of gyrations to obtain MDD. - Produce a minimum of 6x 1.2 kg specimens for each FBC.
STEP 5: Curing process
- Leave specimens in the compaction mould for 1 day at ambient temperature. Do not expose the top of the specimens when using cement. - Oven dry specimens (recommended at 40^{o}C for 3 days).

- Soak half the number of specimens at 25^{o}C for 24 hours.
STEP 6: Property testing
- Store the specimens in a temperature cabinet at 20^{o}C for at least 2 hours before testing. - Test the conditioned and unconditioned specimens using ITS test, ITSM test, Marshall test or UCS test.
STEP 7: Select the Optimum Foamed Bitumen Content (OFBC)
- Plot the data on a curve of FBC vs mechanical property. - Select OFBC according to the maximum values. The minimum acceptable criteria described in Table 2.7 can be used as guidance.

2.6 Industry Experiences in the Use of Cold Recycled Materials in the UK

2.6.1 UK strategy for sustainable development related to highways

By 1999 the UK produced 240 million tonnes of primary mineral aggregates for use principally in the construction industry, from which approximately 50 million tonnes of aggregate demand was met from secondary or recycled sources. By 2012 an extra 20 million tonnes of aggregates will be needed each year if UK demand for aggregates increases by the expected 1% per annum (Nageim and Robinson, 2006). This additional demand can be satisfied by either extracting further primary aggregates or increasing the use of recycled and secondary aggregates (WRAP, 2002). In order to meet this target, Nageim and Robinson (2006) suggested upgrading low-quality natural and waste aggregates for use in bituminous mixtures.

It is likely that ‘traditional’ aggregate sources will become increasingly constrained and alternative sources must therefore be considered and developed, including the greater use of secondary aggregates. It is Government policy to encourage conservation and facilitate the use of reclaimed and marginal materials, wherever possible, to obtain environmental benefits and reduce the pressures on sources of natural aggregates (William, 1996). The UK Government objective for transport systems is that they should provide the choice, or freedom to travel, but minimize damage to the environment (TRL 611, Merrill et al., 2004). A comprehensive sustainability strategy requires considering the use of all resources (including aggregates, binders, and fuel) alongside engineering requirements in the selection of construction techniques.

Cold recycling should now become an increasingly important construction activity in the UK. In-situ and ex-situ techniques are now all feasible and many large and specialist contractors can offer these services. A wide range of alternative materials can be used for cold recycling constructions. Specification clauses for the use of alternative materials have been developed, e.g. by the TRL (see TRL 386 in Milton and Earland, 1999; and TRL 611 in Merrill et al, 2004), the Highway Agency (see HAUC, 2002 and 2005) and WRAP (see WRAP, 2004). TRL 386 gave design guidance and specifications for in-situ recycling using either foamed bitumen or cement for traffic levels up to 20 million standard axles (msa) whereas the TRL 611 gave design guidelines and specifications applicable to both in-situ and ex-situ recycling for higher design traffic (up to 80 msa).

As discussed in Chapter 1 Section 1.1.3, both Potter (1996) and Nunn and Thom (2002) reported that the quality of Foamix (ex-situ production) was found to be better than Foamstab (in-situ production). A reduction of up to 20mm in thickness of a pavement with a design traffic of 10 msa can be achieved if ex-situ mixed material replaces in-situ mixed material. Variability of material properties and moisture condition in the existing pavement is the greatest problem met in the in-situ process, whereas it can be controlled in the ex-situ process. The ex-situ technique is now therefore mainly used in the UK to conform to the policy of sustainability being followed by the Highways Agency (WRAP, 2006). The ex-situ process allows greater control of materials and additives than is possible with in-situ recycling, and offers the ability to prepare material in advance of a contract so that there are no delays waiting for the excavated materials to be processed (WRAP, 2000).

2.6.2 Quality control of ex-situ cold recycling

In the UK, a quality plan is prepared by the contractor and agreed with the client and it covers the entire life cycle for the production of the cold recycled materials from the mix design stage through to the end-product testing stage. The specification does not prescribe the entire content of the quality plan although there are some mandatory minimum requirements. The contractor is provided with significant freedom to produce a material quality plan that satisfies the client whilst ensuring economic efficiency (Merrill et al, 2004).

Assessment of the suitability of materials from an existing pavement

- Assessment is required to be carried out at the same time as the assessment of pavement support using an invasive procedure. If this is not possible, an alternative method of obtaining material for the assessment should be investigated e.g. a limited coring survey (TRL 611, Merrill et al, 2004).
- Samples of aggregate obtained should be fully representative of the aggregate to be used in the recycled pavement. Furthermore, test specimens should ideally be representative of the aggregate obtained by pulverization or planing, for both grading and particle shape (TRL 611, Merrill et al, 2004).

Mix design

- The contractor can provide a mix design for cold recycled materials in accordance with TRL 611 (Merrill et al, 2004). The aim of the mix design process is to provide assurance that a cold recycled mixture will have the appropriate properties one year after construction. The selection of one year properties is to encompass slow curing materials.
- Aggregate for cold recycled material can come from either the pulverized material from existing roads or other approved aggregate types from other sources. A fine grained aggregate may be more suitable to hydraulic binders whilst certain types of aggregate may prove incompatible with certain bituminous binders (TRL 611, Merrill et al, 2004).
- Gradation of the aggregate for cold recycled material should be designed according to TRL 386 (Milton and Earland, 1999). For bitumen bound materials, it is recommended that the amount of fine material passing the 75 micron sieve should be restricted to between 5 and 20% (TRL 611, Merrill et al, 2004).

- The target of aggregate moisture content should be designed properly since it has a large influence on the workability of the material and hence can control the degree of compaction that may be achieved (TRL 611, Merrill et al, 2004).
- Portland cement can be used to provide a material that gains strength quickly at reasonable cost. The risk of thermal cracking should be considered if using a high proportion of this binder (TRL 611, Merrill et al, 2004).
- Foamed bitumen can be used with a variety of combinations of other binders (e.g. Portland cement, lime and Pulverised fuel ash/ PFA) to produce a fully-flexible pavement structure. Materials bound with foamed bitumen, on its own or with lime and PFA, are highly workable, and can be stock-piled or reworked if necessary up to 48 hours after production. Foamed bitumen can be combined with Portland cement in order to generate high early-life stiffness or when more demanding traffic conditions are encountered (TRL 611, Merrill et al, 2004).
- In the laboratory mix design, a low temperature regime for sample conditioning (curing) is recommended wherever possible and it is preferable to use a temperature that is as close as possible to the temperature that would be encountered in the pavement. TRL 611 (Merrill et al, 2004) provides guidance for laboratory conditioning regimes which are dependent upon the family of cold recycled material.

Production

- The contractor should describe in detail the following aspects (TRL 611, Merrill et al, 2004): (a) the storage method of the component materials, (b) the plant used for mixing, (c) the mixing method, (d) the method of addition of the components, and (e) the methods for controlling the addition of the components.
- The proportions of the binding fractions are also monitored by the batching checks; more strict compliance targets are placed on bituminous fractions than other hydraulic fractions (TRL 611, Merrill et al, 2004).

Transportation to site

The contractor should describe in detail the following aspects (TRL 611, Merrill et al, 2004): (a) the location of the mixing plant should be declared, (b) a preferred route and an alternative route for the transportation of the material to the site should be declared with a statement on the time of the day when transportation will occur and the anticipated duration between the mixing process and compaction, and (c) a risk assessment should be performed for the travelling time to site on the preferred and the alternative route including likely delays due to congestion or accidents.

Laying and compaction

- The procedure for laying the material should also cover early-life trafficking issues. In early-life there may be a risk of over-stressing the material and forming cracks, and excessively damaging the recycled material. Regular visual monitoring of the condition of the recycled material is advised to detect any degradation at an early stage and to enable the progression of damage to be stopped if visually occurring (TRL 611, Merrill et al, 2004).
- Moisture content is critical for compaction. The material must be properly controlled to ensure that this does not vary after the material is mixed; problems can arise with the material drying out in hot weather as much as becoming saturated in wet weather (WRAP, 2000). The moisture content should be monitored so that the material immediately prior to compaction is within 2% of the optimum moisture content for compaction (TRL 611, Merrill et al, 2004).
- The contractor should be aware of the workability of the material and manage the construction processes so that delays are minimized; some slow curing cold recycled materials are more workable and tolerant of delays than quick curing materials (TRL 611, Merrill et al, 2004).
- The contractor should provide suitable compaction equipment in order to avoid a substantial loss of serviceability of the finished pavement due to reduced levels of compaction and consequential loss of durability in the material. The plant used for placement and compaction should be described in the method statement (TRL 611, Merrill et al, 2004).
- In order to compact thick layers, the effort needs to be considerably greater, using either heavy vibratory compaction or a tamping roller, than to compact thinner layers, although the process control for compaction of both layers is the same (TRL 611, Merrill et al, 2004).

- The contractor should consider the time required for the material to gain sufficient mechanical stability or strength especially for sites on heavily trafficked routes for which there is an urgency to re-open the pavement to traffic (TRL 611, Merrill et al, 2004).

The end-product test

- All the specified process control criteria are minimum permissible values. There is opportunity for the contractor to demonstrate adherence to superior process control procedures using the appropriate sections of the quality plan (TRL 611, Merrill et al, 2004).
- The degree of compaction should be monitored using relative in situ density as measured using a Nuclear Density Gauge with an average limit of 95% of refusal density being specified (TRL 611, Merrill et al, 2004).
- Material prior to compaction is subjected to an identical sample preparation, laboratory curing and testing regime as declared in the mix design. The values of stiffness and strength obtained from the process need to satisfy the same criteria as defined in the mix design stage (TRL 611, Merrill et al, 2004).
- The contractor should provide compliance testing by means of the Indirect Tensile Stiffness Modulus (ITSM) according to BS DD 213: 1993 with a minimum long-term stiffness as specified in TRL 611 (Merrill et al, 2004). Tests are required with a frequency of 3 per 1000 tonnes of material with a minimum of three per working day (WRAP, 2006).
- The thickness of the recycled layer should be monitored so that its performance is not compromised by variation of thickness (it should be within the level tolerance). Layer thickness is an important determinant of durability of the structure (TRL 611, Merrill et al, 2004).

2.6.3 Case study: Ex-situ recycling of a trunk road in South Devon (A38)

WRAP (2006) has reported a cold recycled bitumen bound base material project using foamix technology on the A38 road in the south-western UK. The A38 is one of the most heavily trafficked roads in Devon. The Annual Average Daily Traffic (AADT) volume is around 37,000 (2-way) with around 10% of this volume being heavy goods vehicles. The road was identified as needing major structural maintenance in 2004 when the increasing amount of patching was brought to the attention of the Highways Agency. Recycling was subsequently carried out in three phases over 6 months on 4km of the northbound carriageway and 8km of the southbound carriageway. The works commenced in September 2005 and were completed in March 2006.

Ex-situ recycling in the form of foamix was selected in this project in preference to conventional reconstruction to provide the required pavement performance criteria for financial reasons and environmental benefits such as reduced pollution and congestion. The structural layers of the existing pavement were recycled to produce the foamix material.

The recycling option chosen was to use foamix with existing asphalt base materials to a thickness of about 230-280mm. An additional 100mm of conventional asphalt surfacing was applied to provide a high quality interface with traffic loads. The detail for pavement structural layers can be seen in Figure 2.33 and Figure 2.34.

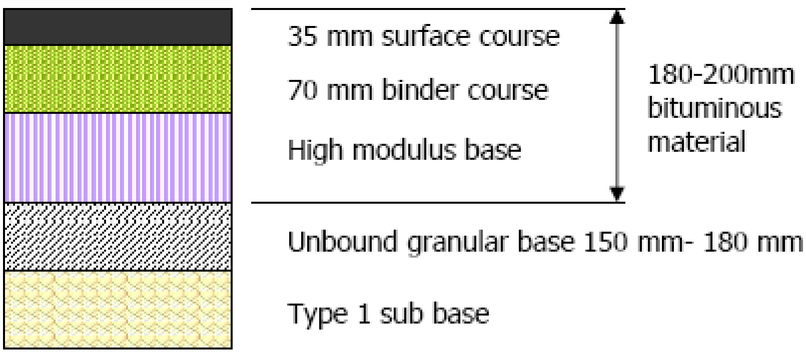


Figure 2.33 - The Existing A38 road pavement.

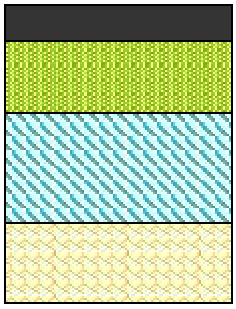


Figure 2.34 - 'New' recycled pavement for A38 road

The ex-situ process involves breaking down planed materials and grading them into different sizes. The materials used in the foamix on the A38 were as shown in Table 2.11. The mixture had been developed following investigation and sampling of materials along the site using 26 trial pits and more than 30 cores and Dynamic Cone Penetration (DCP) tests. This was a key factor in risk management and developing confidence in the mix design.

The recycled pavement was designed for a traffic loading of 35msa, which exceeded the current UK limit for recycling (30msa). However, due to the development of recycling technology and the reducing level of risk associated with recycled materials the permitted design traffic was increased in this case.

Some of the most important issues identified during the works are as follows: (1) a high quality detailed assessment of the volume, type and condition of the existing pavement is required, and (2) during the works, close attention to material quality through quality control procedures must be given; this is especially important in providing a consistent final product if the existing materials are variable.

Table 2.11 - Constituents of the foamix mixture for the A38 road project.

Material	Percentage by mass	Compliance criteria
Recycled aggregate from the A38	88%	Zone A (TRL 611)
Pulverised Fuel Ash (PFA)	5%	Incorporated in the end performance requirements
Bitumen (Pen 100/150)	3%	3% ± 0.5%
Portland cement	1.5%	1.5% ± 0.3%
Foaming water content	Varies; typically 2%	3.4% - 7.4%

2.7 Summary

To support the traffic loads, road pavement materials are divided into 4 essential layers, i.e. surfacing, binder course, base course and pavement foundation. Each layer has a different purpose to service either structural or functional requirements. Therefore, road materials should be designed according to their function in the pavement.

For structural purposes, three fundamental properties are required i.e. stiffness modulus, resistance to permanent deformation and resistance to fatigue, in order to characterize road materials and hence to ensure whether they can perform their function or not. The failure mechanisms of pavement materials have been well understood and the way to achieve the optimum material properties in order to fulfill the requirements both in mixture design and in the pavement design process has been well delineated. Therefore, it is important to provide a clearer estimate of the material quality with the intention of material properties optimization.

As an alternative road material, the properties of foamed asphalt mixture, either under field condition or at laboratory scale, are gradually becoming understood. Understanding of the key factors affecting foamed asphalt mixture properties is also progressively improving. The effects of aggregate properties, moisture content, curing method, binder content, temperature etc have been clearly defined. Some guidance and specifications related to material components and end product performance have been introduced. All these contribute to the improvement of foamed asphalt application. However, foamed bitumen characteristics are not yet fully understood in correlation with mixture properties. This leads to inconsistent production of foamed asphalt materials and hence to be not achieving their optimum performance.

Understanding of the influence of foamed bitumen characteristics on the corresponding mixture is absolutely essential. This will aid in selecting binder type and generating the best foam quality. These two aspects are marked as an important step in order to produce optimum mixing properties. It has been observed by many investigators (e.g. Bissada, 1987 and Jenkins et al, 2004) that poor mixture performance is often due to poor mixing properties. It has been remarked that foamed bitumen characteristics are one significant factor affecting the mixing performance. The evidence from Lee's investigation (1981) is that the effect of foam characteristics on mixture stability can be obscured by other factors such as aggregate properties, moisture content and mixer type. On the other hand, Bowering and Martin (1976b) reported that foamed bitumen characteristics have a significant effect on mixture performance. However, this is based on two different foam types only, i.e. at ERm= 3 and ERm= 15. Therefore, more experiments are required to complete the picture of correlation between foam characteristics and mixture properties.

Extensive industry experience in the UK in implementing foamed bitumen technology (Potter, 1996; Nunn and Thom, 2002; and WRAP, 2006) has found the quality of Foamix (ex-situ production) to be better than Foamstab (in-situ production). The ex-situ technique is now therefore mainly used in the UK to conform to the policy of sustainability being followed by the Highways Agency (WRAP, 2006). The ex-situ process allows greater control of materials and additives than is possible with in-situ recycling, and offers the ability to prepare material in advance of a contract so that there are no delays waiting for the excavated materials to be processed (WRAP, 2000).

To assess the influence of foam characteristics on the mixture performance, it is best to measure fundamental properties which represent response under traffic loads in pavement layers. These fundamental assessments can easily be conducted using NAT facilities. Although foamed asphalt mixtures are not fully visco-elastically bound materials, they can still be evaluated using NAT facilities. The results should be proportional to actual field performance and hence any differences in mixture performance affected by foam characteristics can be evaluated directly. It should be noted that the values resulting from NAT tests will, of course, be different from when confining stress is applied on them, since foamed asphalt materials have been observed to be stress-dependent by many investigators.

3 INITIAL STUDY

3.1 Introduction

This chapter reports the results of an initial study to investigate the properties of the materials used in this research, including laboratory trial works and pilot scale experiments in the Pavement Testing Facility. The main aims of the initial study are: (1) understanding the whole process of foamed asphalt manufacture including material preparation, mixture design, laboratory work and construction, (2) understanding the appearance of foamed asphalt material and its performance, and (3) identifying any real problems in the application process.

3.2 Investigating Properties of the Materials Used

3.2.1 Virgin crushed limestone (VCL) aggregate

3.2.1.1 Initial condition of VCL aggregate

VCL aggregate was collected and stored separately in six stockpiles according to the following size fractions: 20 mm, 14 mm, 10 mm, 6 mm, fines and filler. The initial colour is light brown in the dry condition (Figure 3.1).

3.2.1.2 Gradation of VCL aggregate

Gradation or particle size distribution of aggregate is the range of particle sizes from maximum size (D) down to minimum size (d) and it can be determined by sieve analysis. The maximum size is selected according to mixture type, layer position and layer thickness.

The individual and combined gradations are shown graphically in Figure 3.2. The combined gradation was designed to be close to the Fuller packing equation with a maximum aggregate size of 25mm and within the ideal grading envelope for foamed asphalt as recommended by Akeroyd and Hicks (1988) (see Table 3.1).

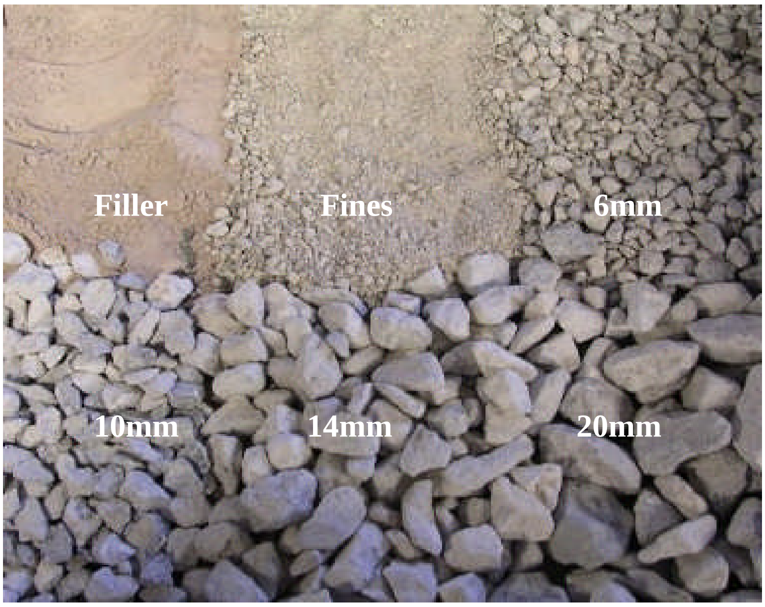


Figure 3.1 - Appearance of virgin crushed limestone aggregate used in this study

Table 3.1 - Gradation design of virgin crushed limestone aggregate (BS 812-103.1: 1985)

Sieve size (mm)	Percent passing		
	Ideal envelope	Fuller D25(*)	Design
50	100 – 100	-	-
37.5	100 – 88	-	-
25	100 – 78	100	100
20	99 – 66	89.44	96.71
14	86.5 – 57	74.83	74.60
10	74 – 48	63.25	62.86
6.3	62.5 – 39	50.20	51.20
2.36	43 – 26	30.72	31.65
1.18	33.5 – 18	21.73	22.70
0.6	28 – 13	15.49	17.20
0.3	24 – 10	10.95	13.55
0.15	21 – 7	7.75	10.85
0.075	20 – 4.5	5.48	8.60

(*) Fuller equation: $p = 100*(d/D)^{0.5}$

p= total percentage passing a given size (theoretical ideal passing)
d= size of sieving opening
D= largest size (sieve opening) in the gradation
Exponent 0.5= degree of an ‘ideal’ curve equation

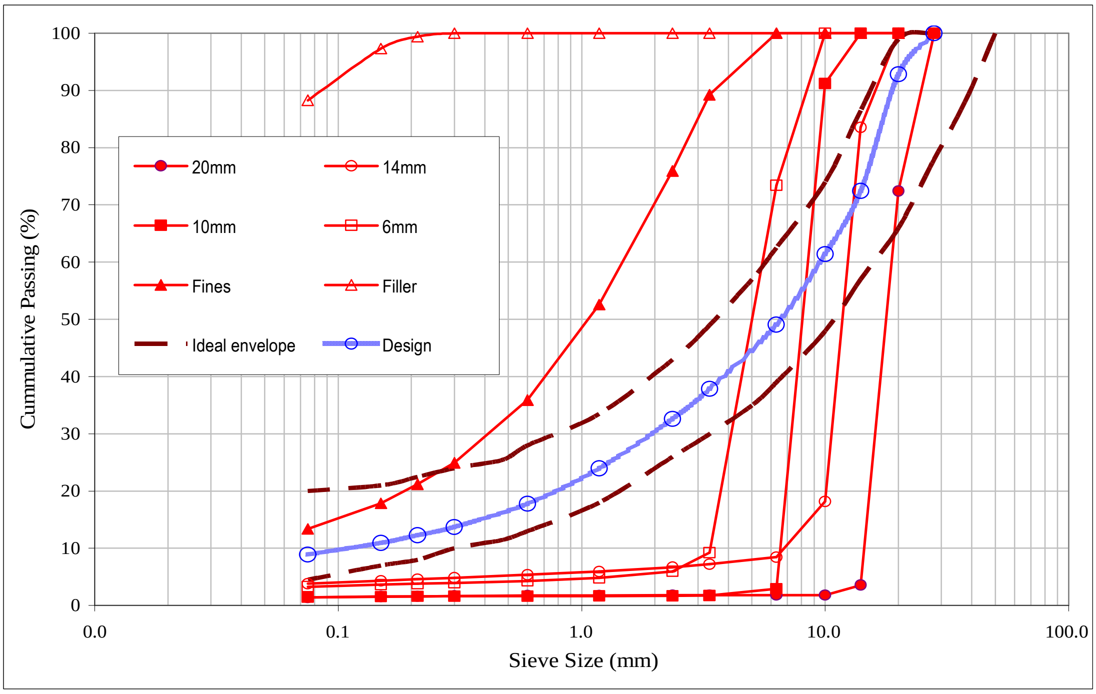


Figure 3.2 - Gradation of virgin crushed limestone (VCL) aggregate

3.2.1.3 Liquid limit and plastic limit of VCL aggregates

The consistency of material passing 425 microns was expressed by Liquid Limit (LL) and Plastic Limit (PL) in accordance with BS 1377 part 2: 1990. Liquid Limit was determined using the cone penetrometer**,** giving 17.9% whereas the Plastic Limit obtained was 15.2%. It can be calculated that the Plasticity Index (PI), LL minus PL, was 2.7%.

3.2.1.4 Particle density and absorption of VCL aggregate

Table 3.2 shows the results of particle density and water absorption investigation for all 6 fractions of VCL aggregate. Testing has been performed according to BS EN 12697-28:2001 (sample preparation), BS 812-2: Clause 5.4: 1995 (for fines to 20mm) and BS EN 1097-7: 1999 (for filler).

Table 3.2 - Particle density of virgin crushed limestone aggregate

Property	Size (mm)					
Particle density (Mg/m^3)	20	14	10	6	fines	filler
Oven dried	2.633	2.607	2.608	2.427	2.668	2.623
Surface saturated dry	2.653	2.634	2.640	2.526	2.674	-
Apparent	2.685	2.679	2.693	2.696	2.686	-
Water absorption (%)	0.74	1.03	1.2	4.1	0.26	-

3.2.1.5 Compaction characteristic of VCL aggregate

The compaction characteristic of VCL aggregate has been investigated using modified Proctor in accordance with BS EN 13286-2: 2004 in order to determine the maximum dry density (MDD) and optimum moisture content (OMC). As shown in Figure 3.3 the value of MDD is 2.242 Mg/m3 and OMC is 6.4%.

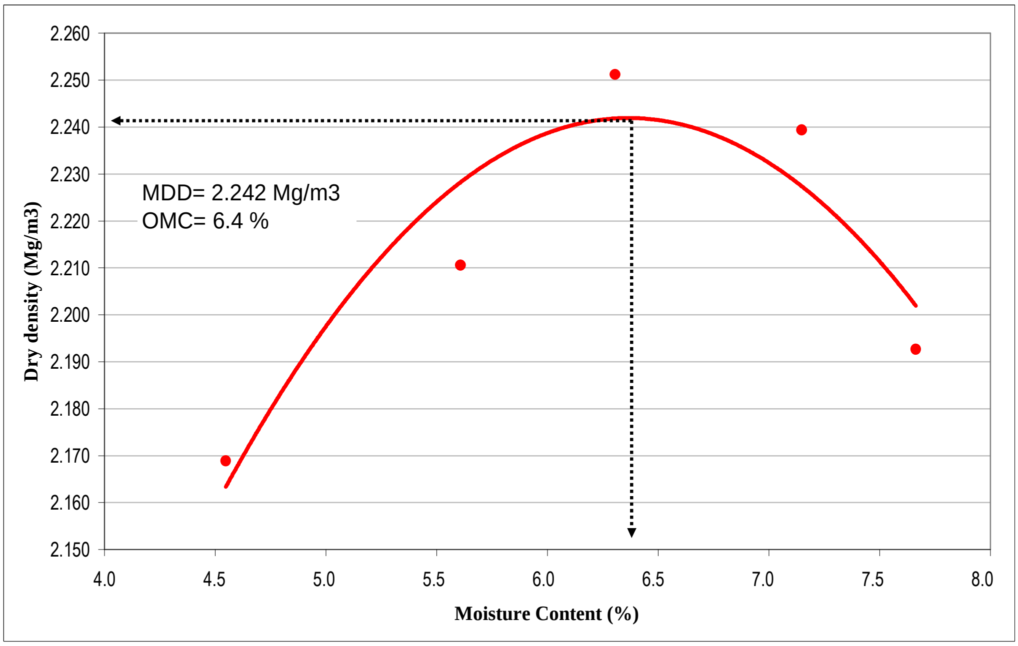


Figure 3.3 - Compaction characteristic of virgin crushed limestone aggregate

3.2.2 Reclaimed asphalt pavement (RAP)

3.2.2.1 Initial condition of RAP

RAP material was collected from an asphalt producer with an in-plant asphalt facility. This material was originally milled from various asphalt roads and brought together into one stockpile. Initial water content of RAP varies between 3 and 4%. It was subsequently dried to achieve homogeneous dry condition prior to the mixing process. Figure 3.4 shows the dried RAP aggregates.

3.2.2.2 Gradation of RAP material

As shown in Figure 3.5, the RAP gradation exhibits deficiency in fines according to the recommended ideal grading envelope for foamed asphalt (Akeroyd and Hicks, 1988). The filler content was only found to be 0.3% (dry method) or 2.1% (washing method) whereas the minimum required for foamed asphalt mixture is about 4.5%.

3.2.2.3 Particle density and water absorption

Particle density of RAP material can be established by a pycnometer and gas jar technique in accordance with BS 812-2: 1995 or BS EN 1097-6: 2000, and at the same time water absorption can be obtained (Table 3.3).

3.2.2.4 RAP components and recovered bitumen properties

The presence of RAP when it is combined with virgin aggregate can reduce the required binder to obtain the optimum mix properties (Ruenkrairergsa et al., 2004 and Loizos et al., 2004). It may be therefore useful to investigate the composition of RAP and the properties of its components.

The old bitumen was separated from the aggregates using a fractionating column test in accordance with BS EN 12697-4: 2005. The recovered bitumen was then tested for penetration, softening point and viscosity accordance with BS EN 1426: 2000, BS EN 1427: 2000 and BS EN 13302: 2003 respectively. The old bitumen content was determined by the difference method in accordance with BS EN 12697-1: 2000. Table 3.4 and Table 3.5 show the test results.

Figure 3.4 - Appearance of RAP materials used in this study

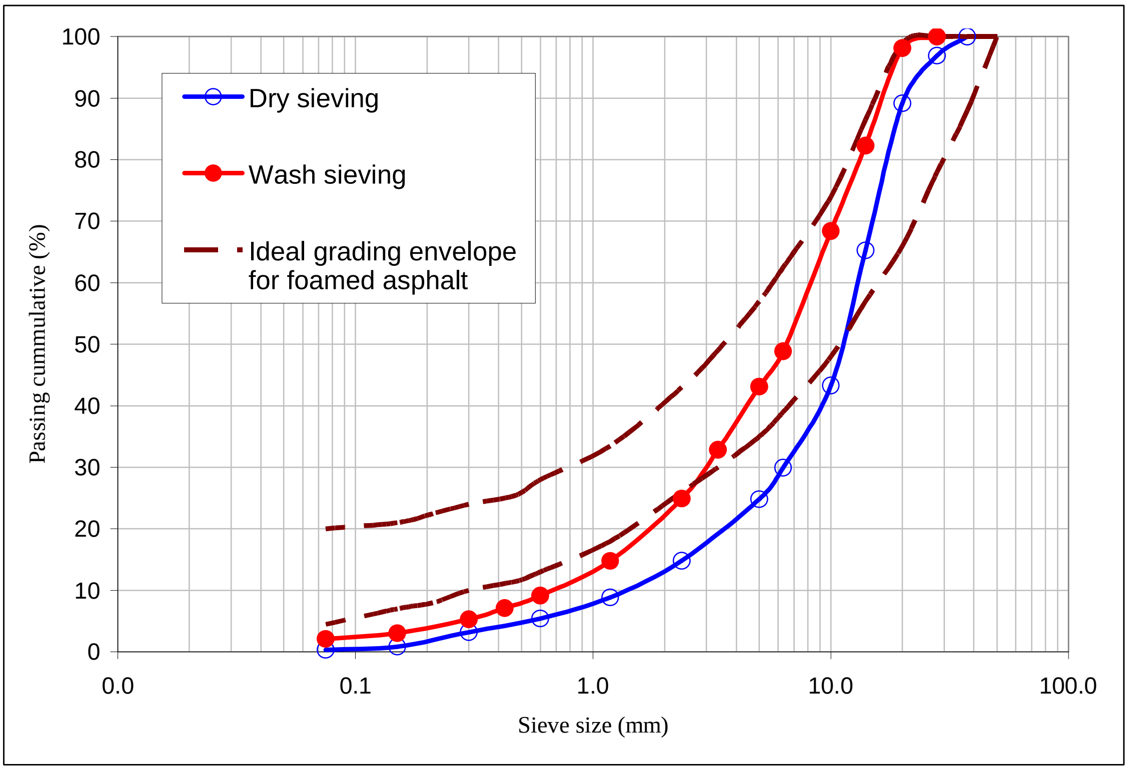


Figure 3.5 - Gradation of RAP material

Table 3.3 - Particle density and water absorption of RAP material (BS 812-2:1995)

Property	Size (mm)		
Particle density (Mg/m3)	< 5	5-10	10-20
Oven dried Saturated surface dry Apparent	2.39 2.45 2.54	2.49 2.52 2.56	2.54 2.56 2.60
Water absorption (%)	2.55	1.01	0.93

Table 3.4 - Recovered bitumen properties after fractionating column test

Property		Test samples		
		1	2	3
Penetration (0.1 mm) (at 25°C, 5s, 100g)		21	25	32
Softening Point (°C) (Ring and Ball)		64.6	63.2	62.2
Viscosity (mPa.s)	at 120°C	-	2530	2920
(Brookfield Viscometer)	at 150°C	550	420	500
	at 180°C	-	110	150

Table 3.5 - Composition of RAP material

Component	Test samples			Average
	1	2	3	
Recovered bitumen (%)	3.69	4.1	3.9	3.9
Filler (%)	10.67	9.74	8.95	9.8

3.2.3 Bitumen

Three bitumen grades were used in this study i.e. Pen 50/70, Pen 70/100 and Pen 160/220. The properties of these bitumens are shown in Table 3.6 and Figure 3.6.

The bitumen viscosities shown in Figure 3.6 were measured using a Dynamic Shear Rheometer (DSR) at a frequency of 0.1 Hz for temperatures of 5, 20 and 40°C and a Brookfield rotary viscometer for temperatures of 140°C to 180°C. It can be seen that the differences in viscosity values of the 3 bitumen grades increase with decreasing temperature.

Table 3.6 - Properties of bitumen pen 50/70, pen 70/100 and pen 160/220

Property	Bitumen		
	Pen 50/70	Pen 70/100	Pen 160/220
Specific gravity	1.024	1.03	1.021
Penetration (25 °C, 100g, 5s) (0.1 mm)	54 - 56	85 - 93	180 - 198
Softening Point (ring and ball) (°C)	52 - 53	45 - 49	37 - 38
Viscosity at 5°C (DSR 0.1 Hz, kPa.s)	8795	4322	1315
Viscosity at 20°C (DSR 0.1 Hz, kPa.s)	899	257	68
Viscosity at 40°C (DSR 0.1 Hz, kPa.s)	21.90	5.54	1.63
Viscosity at 140°C (mPa.s)	362	262	164
Viscosity at 150°C (mPa.s)	235	172	111
Viscosity at 160°C (mPa.s)	153	114	74
Viscosity at 170°C (mPa.s)	108	80	53
Viscosity at 180°C (mPa.s)	77	57	40

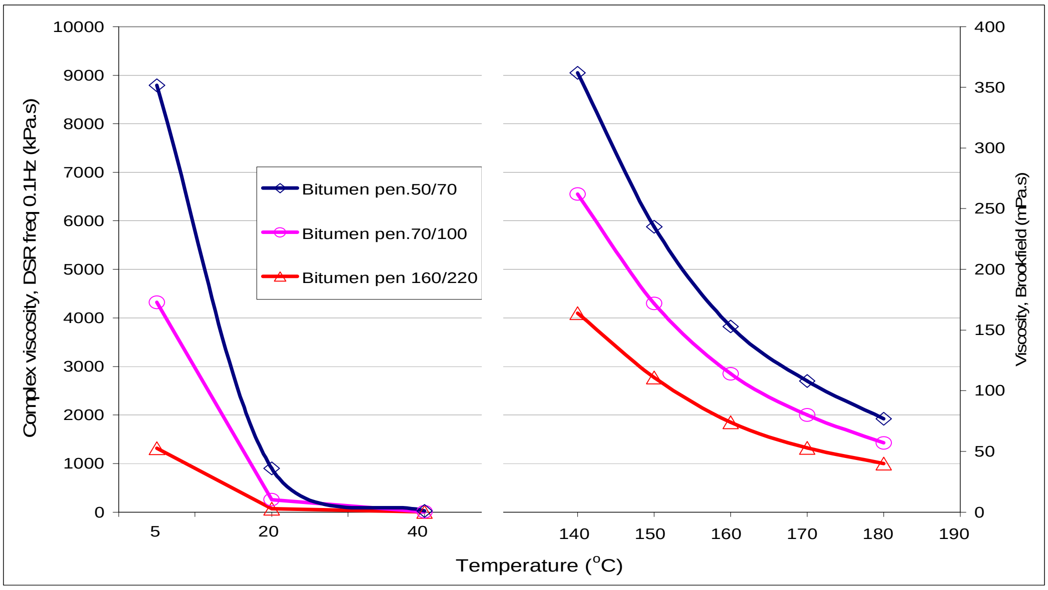


Figure 3.6 - Viscosity of bitumen pen 50/70, 70/100 and 160/220 at various temperatures

3.3 Laboratory Scale Trials

3.3.1 Setting up the Wirtgen WLB 10 foaming plant

The foaming plant used in this study was a Wirtgen WLB 10, developed in 1996 (Figure 3.7). This mobile plant was specially designed to produce foamed bitumen under laboratory conditions, to investigate foamed bitumen characteristics and to generate foamed asphalt mixes as well by attaching a mechanical mixer to the foaming unit. The foamed bitumen produced by this unit is similar to that produced by the foamed bitumen systems mounted on large recycling machines (such as the Wirtgen WR 2500). The bitumen temperature (available up to 200°C) and foaming water content (available up to 10 %) can be controlled by this machine in order to investigate the optimal application rate for asphalt mixes.

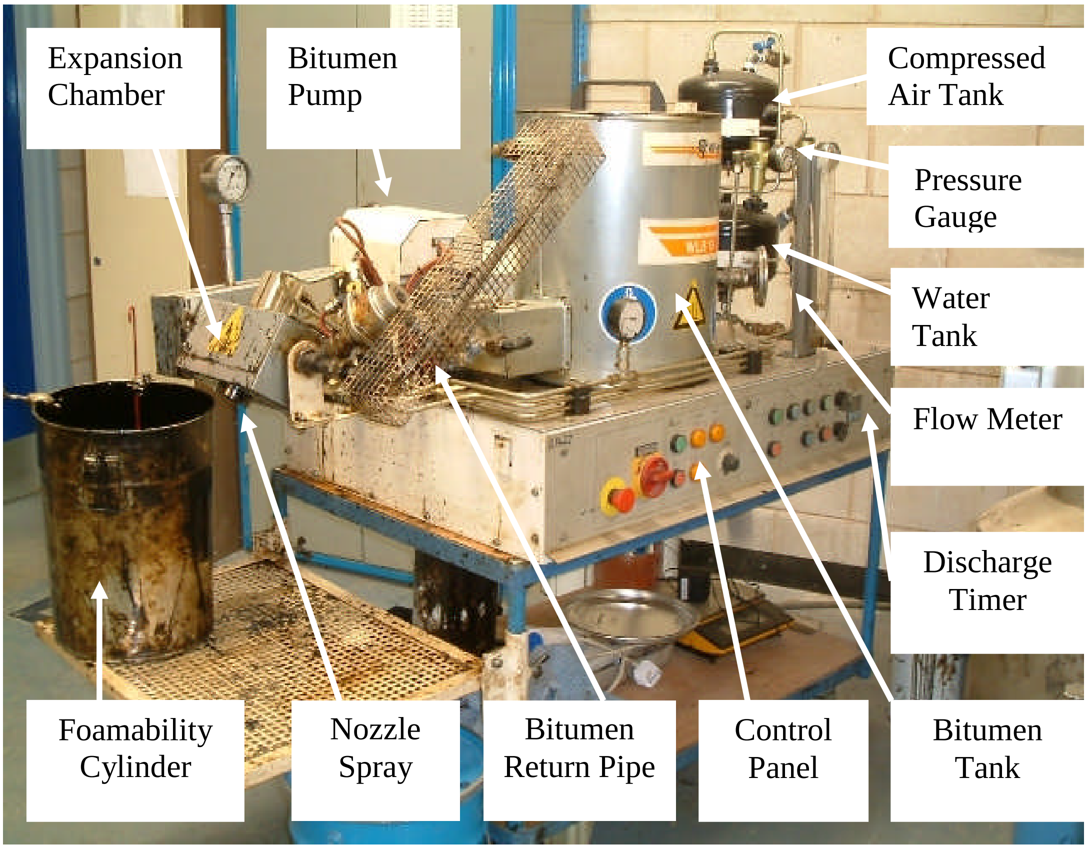


Figure 3.7 - Laboratory Foaming Plant type Wirtgen WLB 10

The operation system of this plant can be described as follows (see also Figure 3.8):

- The fluid hot bitumen is circulated in a pipe by a bitumen pump from a bitumen tank, through the expansion chamber and returned to the bitumen tank via a return pipe. This circulation will operate whenever the machine is switched on.
- The amount of water that will be added into hot bitumen is controlled by a water flow meter, whereas the amount of foamed bitumen desired to be produced is controlled by a discharge timer. For this machine, one second will produce 100 grams of foamed bitumen.
- The air pressure and water pressure are controlled separately by pressure gauges.
- When the foam start button is pushed, the 2.5mm valve will open and hot bitumen flow inside the pipe will be pushed out by 1.5 bars pressure to the expansion chamber in which at the same time water and air are injected together. The created foam in the expansion chamber is then sprayed out via a spray nozzle at 100g per second.

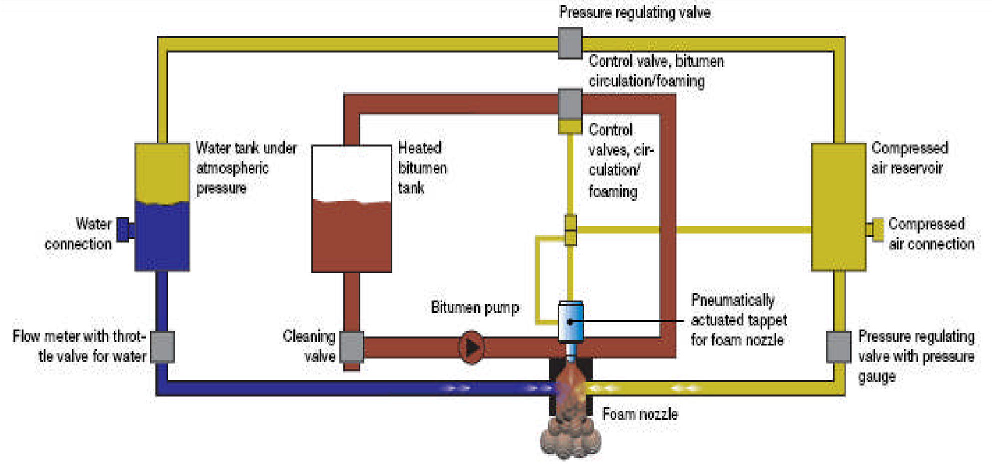


Figure 3.8 - Schematic illustration of the laboratory-scale foamed bitumen plant WLB 10

The foaming machine has been set to produce the best quality of foam. The nozzle was set to enable spraying of water and air under a certain pressure at the chamber. The pressure was set at 6 and 5 bars for water and air pressure respectively, which was found to produce the best foam over the full range of pressures investigated. It is noted that the water pressure should be higher than the air pressure to enable water spraying. It is known from literature that the pressure influences the quality of foam but that it has only a small effect, except at a pressure of 1 bar, which is disadvantageous to the foamed bitumen (He and Wong, 2006)

During setting of the machine, it was found that the temperature of the bitumen tank could not be set properly at more than 180^{o}C, and that when temperature is desired at a certain level, it fluctuates up and down around that level over about 2.5^{o}C.

3.3.2 Setting the rate of bitumen spraying

Bitumen spraying rate is the mass of foam production per second. This machine was designed to produce foam at 100 grams per second. The foam production is controlled by a discharge timer. It means that when the timer is set at certain number of seconds e.g. 1.5s, the foam produced contains about 150g of bitumen.

It is recommended to check the bitumen spraying rate regularly. When checking the bitumen spraying rate, the water valve should be closed to allow production of bitumen only. Table 3.7 shows the result of a bitumen spraying rate investigation which gave values in a reasonable range i.e. 104 g/second on average.

Table 3.7 - Bitumen spray rate (using bitumen grade Pen 160/220)

Setting time (seconds)	Mass of bitumen (gram)	Bitumen flow rate (gram/ second)
1	104	104
2	209	104.5
3	315	105
4	412	103
5	517	103
Average		104

3.3.3 Foam production

3.3.3.1 Setting foaming water content (FWC)

FWC is controlled by a water flow meter in which water flow rate can be set related to bitumen spraying rate. Wirtgen provided a table of relationship between FWC (% of bitumen mass) and water flow rate setting (litre/ hour) as shown in Table 3.8, for a bitumen flow rate of 100 grams/second. Using this table, the water needed to be added to the bitumen phase can be set easily using the provided water flow meter.

Table 3.8 - Water flow rate setting

Foaming water content (%)	1	2	3	4	5	6	7	8	9	10
Water flow rate setting (litre/hour)	3.6	7.2	10.8	14.4	18	21.6	25.2	28.8	32.4	36

3.3.3.2 Measuring maximum expansion ratio (ERm) and half-life (HL)

In the first trial, foam was produced using bitumen grade 160/220 and a bitumen temperature at 180^{o}C. It was confirmed that the ERm and HL were significantly affected by foaming water content (FWC). When FWC was increased the ERm increased and the HL decreased.

3.3.4 Foamed asphalt mixture (FAM) production

3.3.4.1 Material used for trial

Materials and foam production used are shown in Table 3.9. Properties of the limestone aggregates and bitumen used in this trial are as described in section 3.2.

Table 3.9 - Materials used for laboratory trial

Bitumen grade	Pen. 160/220		
Aggregate	Crushed Limestone OMC = 6.4% MDD = 2242 kg/m3	Composition: 20mm = 25% 14mm = 12% 10mm = 13%	 6mm = 8% Fines = 39% Filler = 3%
Selected foamed bitumen production	Temperature 180^{o}C Foaming water content 2%		

3.3.4.2 Mixing process

Mixing was carried out using a 20 quarts Hobart mixer (Figure 3.9) which was mounted on the foamed bitumen plant (see Figure 3.10a). The aggregates were added with water in the mixer bowl and then they were mixed for about one minute to achieve homogeneous condition. While aggregate mixing was in progress, foamed bitumen was sprayed from the nozzle into the bowl and directly mixed with the wet aggregates for another one minute. The mixing time of foam mix was related to the approximate time needed for the foam to collapse after spraying.

The amount of water added to the aggregate was selected at 4.6% (percentage by dry aggregate mass) which is approximately 72% of the OMC value (6.4%). At this point, the most homogeneous mixture in terms of foam dispersal in the aggregate appeared to be obtained. This point was close to the middle value of the range 65% to 85% of OMC (recommended by Lee, 1981) and slightly lower than the optimum compaction and workability recommended by Wirtgen (2005), which was found to be 5.1%. However, this point was far lower than the fluff point (7%) and the compaction moisture (Sakr & Manke, 1985), which was found to be 9% when the OMC, filler content and binder content were taken as 6.4%, 9% and 4% respectively. When the water was added at OMC or more, the foam was not uniformly distributed and the mixture had a spotty appearance.

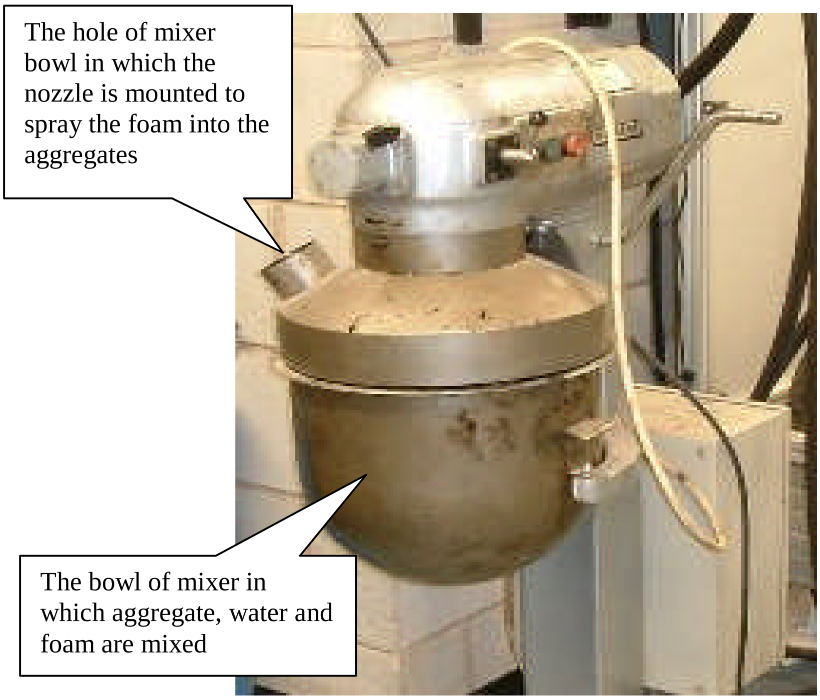


Figure 3.9 - Hobart Mixer 20 Quarts Capacity

The Hobart mixer was preferred to a pan mixer for two reasons. Firstly, the Hobart mixer was already designed to be mounted onto the foaming plant. Secondly, the mixing speed of Hobart mixer is higher than that of the pan mixer. Long et al (2004) recommended using a high-speed twin shaft pugmill mixer (developed by CSIR Transportek in South Africa) since it produced mixes that closely represented those produced in the field. Unfortunately, this mixer is not available in the NTEC laboratory.

The Hobart mixer has several types of agitator. In this study, two types have been tried namely a spiral dough hook and a flat type (Figure 3.11). The latter type gives better mixing speed but has degradation problems. Its wide frame gives less than 15mm space between the frame and the mixer bowl wall so that any stone larger than this will cause the agitator not to work properly and may potentially be broken. On the other hand, the spiral dough hook has a single spiral rod so that it has no space problem but gives segregation problems. The fine aggregates will sink during mixing and the agitator can not reach them properly. It was therefore suggested to use aggregates with a maximum size of 10mm when using the flat agitator to avoid degradation problems, and to do hand-mixing before and after spraying foam to avoid segregation.

When foam is sprayed from the nozzle into the mixer bowl, a part of the foam becomes attached to the agitator (see Figure 3.10b). This means that not all sprayed foam will mix with aggregate. From much evidence, it was found that the amount of bitumen attached to the agitator was around 20 to 30 percent of the sprayed foam. Therefore, agreeing with a Wirtgen suggestion (Wirtgen, 2005), this study applied 20% and 25% additional foam for the spiral dough hook and flat type agitator respectively.

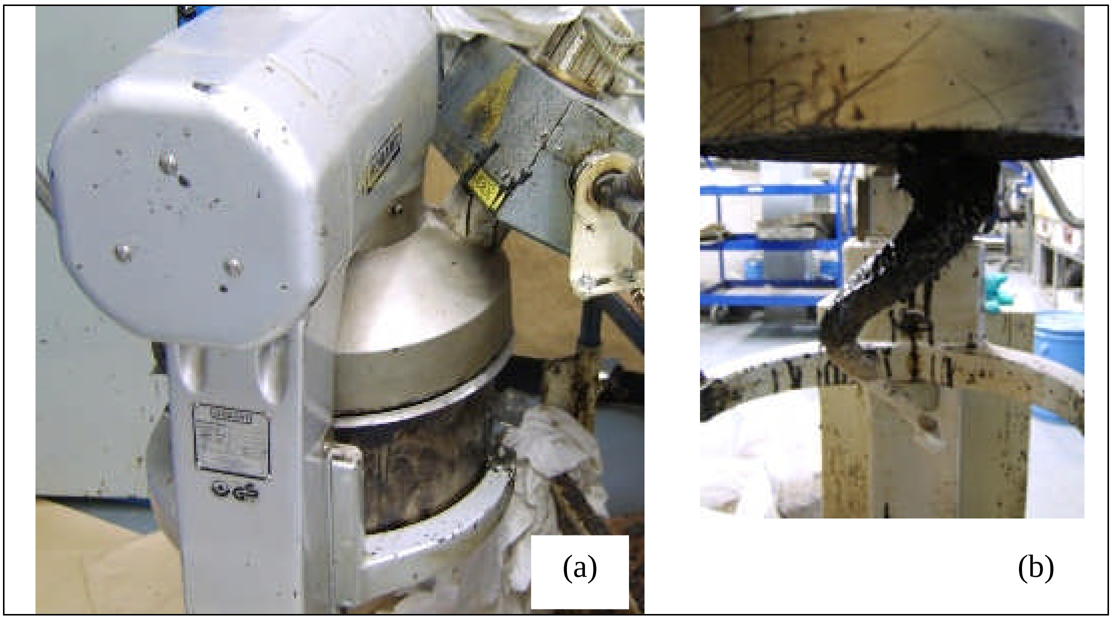


Figure 3.10 - (a) The Hobart mixer was mounted onto the foaming plant, (b) An amount of foam becomes attached to the agitator

Figure 3.11 - Two types of agitator used in this study, Left: Spiral dough hook type; Right: Flat type

3.3.4.3 Loose material of foamed asphalt

A 20mm well graded limestone aggregate was mixed with 4% foamed bitumen (percentage by weight of dry aggregates). Figure 3.12a shows the homogeneous wet aggregate before foaming, in which water was understandably concentrated in fines particles due to suction effects between particles. When foam was sprayed onto these wet particles, it appeared that the foam preferred to distribute on the fine particles (see Figure 3.12b). When the flat type agitator was used, the foam seemed well distributed and separated into fine particles; however, when the dough hook was used, the foam was not as well distributed and separated.

Figure 3.13 shows the dry loose material. It can be seen that fine aggregate (< 6mm) seemed to be bonded to form a mastic. When the mastic was pressed with a finger, the bitumen came out and had clearly coated the fine aggregates. It was noted that not all fine aggregate particles were coated by foam as shown in the figure. Medium aggregate (6-10mm) appeared partially coated, whereas coarse aggregate (> 10mm) was totally uncoated. Overall, the foamed-mixture looked like a moist aggregate, coloured brown rather than black.

Figure 3.12 - Appearance of wet loose materials; (a) before foaming, (b) after foaming

Figure 3.13 - Appearance of dry loose materials

When the same aggregates were mixed with 6% foam, the result was not too different from the previous mixture (using 4% foam), except in the colour. It looked darker in colour.

A 10mm single size aggregate was also tried with 4% foamed bitumen. It was clear that the foam could not coat the single size coarse aggregate; the resulting mixture was an uncombined asphalt in which the foamed bitumen appeared in several globules.

3.3.4.4 Storage period

Loose foamed mixtures (with no hydraulic binder) can be stored up to 3 months before compaction (Wirtgen, 2004). Khweir (2006) also reported that foam mix stiffness did not significantly decline even if stored up to 28 days. However, in this study it was found that the best approach was to leave the mixture to cool prior to being sealed in plastic bags. Sealing foamed mixture while still in a warm condition caused condensation. The use of the plastic bag is to avoid excessive evaporation occurring. Therefore the mixtures were placed inside closed tins and stored in a cool room (temperature 5^{o}C). The storage of foamed materials will not change the mixture structure as long as there is no significant inter-particle pressure or loss of moisture.

3.3.4.5 Compaction

The foam mixtures were compacted using a gyratory compactor to achieve the maximum dry density. Specimens were fabricated at a diameter of 100mm and a height of 50mm. In this study, the gyratory compactor was selected for preparing specimens since it was believed that this would yield material structures approaching those set up during the construction of actual pavements. This method is also practical and enables a particular specimen density to be achieved in laboratory. In this case, the machine can be set to compact specimens to obtain 100% of maximum dry density (MDD). In this research, the specimens were compacted using a ram pressure of 600 kPa and a compaction angle of 1.25 degrees, in line with the Superpave protocol. Jenkins et al. (2002) used a gyratory compactor to prepare foamed asphalt specimens and found that a pressure of 600 kPa at an angle of 1.25 degrees for 150 gyrations approach the 100% MDD modified AASHTO condition.

3.3.4.6 Compacted specimens

Figure 3.14 shows two different foamed asphalt materials, mixed using the spiral dough hook agitator and flat agitator respectively, and both are compared to hot mix asphalt specimens. All materials have equivalent aggregate grading and binder content.

Figure 3.14 - Appearance of compacted specimens; (left) foamed asphalt mixed using spiral dough hook agitator, (middle) foamed asphalt mixed using flat agitator and (right) hot mix asphalt.

Clearly, both the foamed asphalt materials are lighter in colour than the corresponding hot mix specimens due to different coating profiles. The large stones in the foamed asphalt specimens can also be seen to be obviously uncoated.

Differences between the foamed asphalt specimens mixed using different agitators were also clearly evident. Foam dispersal in flat agitator mixed specimens is far better than in dough hook mixed specimens. Bitumen in the latter specimens is not uniformly distributed and the appearance is spotty.

3.3.4.7 X – Ray scanning

Figure 3.15 and Figure 3.16 show the X- Ray scanning results of a foamed asphalt and hot mix asphalt respectively. Scanning has been performed on specimens with a diameter of 100mm and height of 65mm. The pictures shown are a middle slice scan through the specimens. Both specimens were compacted using a Gyratory compactor to approximately the same bulk density. However their void contents were slightly different due to their rice density also being slightly different.

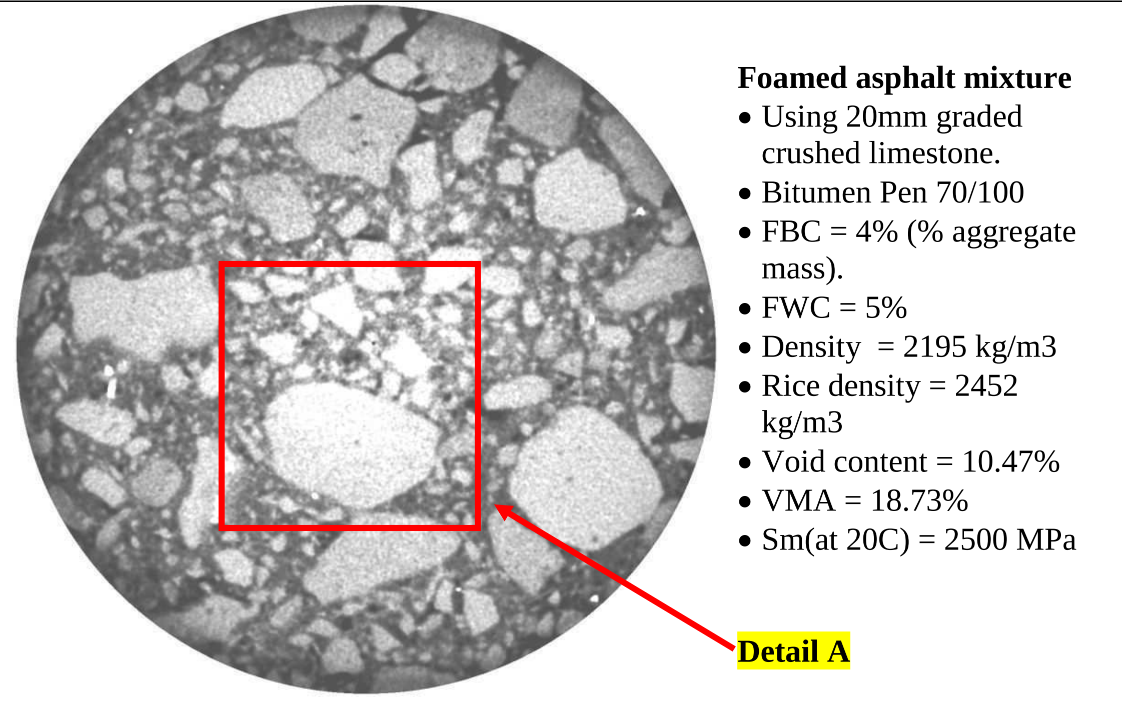


Figure 3.15a - Appearance of foamed asphalt mixture under X-Ray scanning

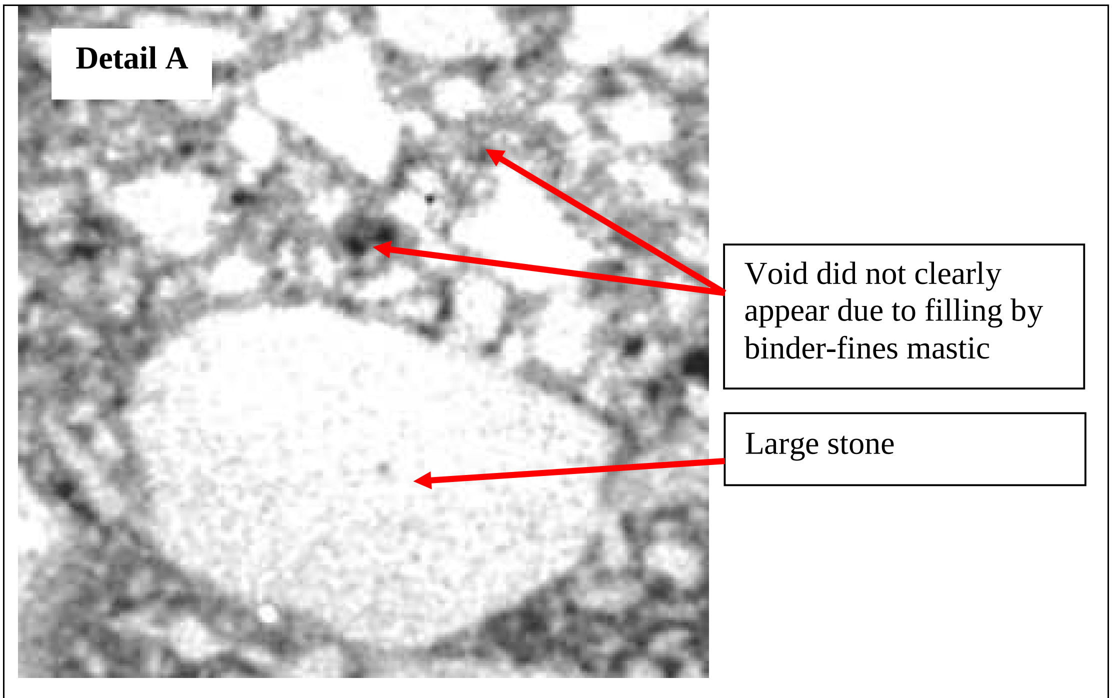


Figure 3.15b - Appearance of foamed asphalt mixture under X-Ray scanning (Detail A)

The greatest difference between the specimens is in the appearance of the void component (black colour). The void component in the hot mix specimen is clearly evident between the large particles. This is not the case in the foamed asphalt specimen. The void does not appear clearly due to continuous filling by binder-fines mastic. This is probably a function of the uniform distribution of water (including binder droplets) in the mixture during compaction; hence when water evaporates the mastic remains well distributed.

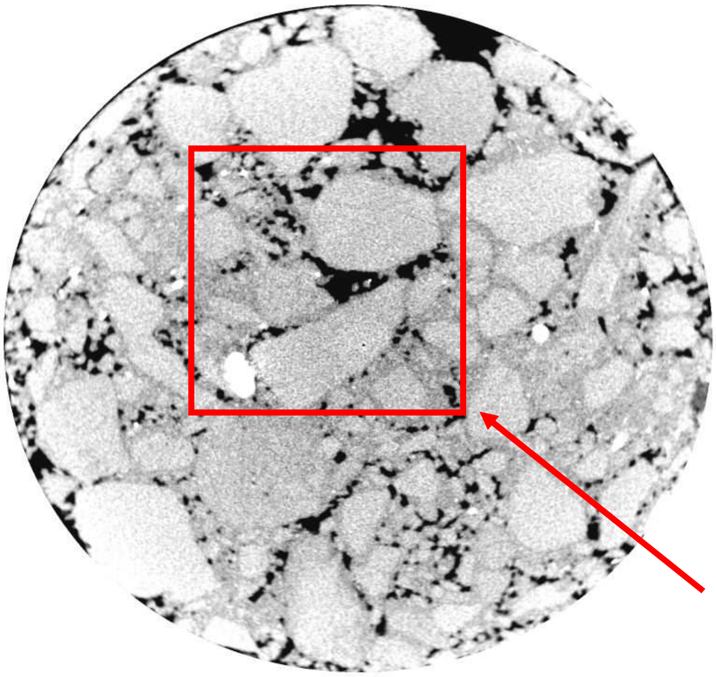


Figure 3.16a - Appearance of hot mix asphalt under X-Ray scanning

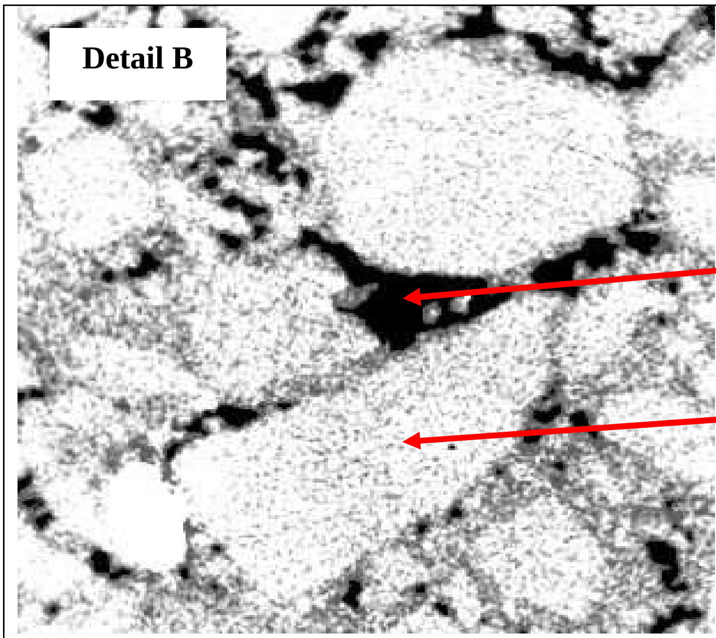


Figure 3.16b - Appearance of hot mix asphalt under X-Ray scanning (Detail B)

3.4 Pilot Scale Trial

3.4.1 General

A trial section of a foamed cold mix asphalt pavement was constructed in the Nottingham University Pavement Test Facility (NPTF). A combination of crushed limestone and reclaimed asphalt pavement (RAP) aggregates with selected binder was used to simulate recycled construction. It is common that a deteriorated asphalt layer overlying an unbound crushed stone layer is milled, remixed with additional foam and subsequently recompacted to a specified density.

The purpose of this simulated pilot scale trial was to understand the whole process of foamed asphalt construction including (1) mixture design, (2) foaming-mixing process, (3) construction and (4) layer performance.

3.4.2 Pavement Test Facility (PTF)

The PTF is housed in the laboratory of the Nottingham Transportation Engineering Centre (NTEC). This facility was developed in 1970s (Brown and Brodrick, 1981) and the hydraulics were reconditioned in 2001. The movement of the wheel is controlled by a hydraulic motor which pulls a steel rope (attached to both sides of the carriage housing the wheel) in both directions. Figure 3.17 represents the NPTF with a pit area of 2.4m x 4.8m. The maximum load and speed that can be applied are 15 kN and 8 km/hr respectively. Normally, when a high load is applied the speed is kept at a low level to maintain stable wheel motion.

3.4.3 Test program

The NPTF pit area was divided into 6 sections of which 4 sections were foamed asphalt pavement composed of various mixture proportions and binder types (Mix 1 to Mix 4); the other 2 sections did not form part of this research. Figure 3.18 shows the pavement layout including the embedment strain gauge positions (see also Figure 3.19). The 4 combinations of foamed asphalt mixture were:

- Mixture 1: using 75%RAP aggregate and bitumen Pen. 50/70,
- Mixture 2: using 75%RAP aggregate and bitumen Pen. 70/100,
- Mixture 3: using 50%RAP aggregate and bitumen Pen. 70/100,
- Mixture 4: using 50%RAP aggregate and bitumen Pen. 70/100 + 1.5% cement.

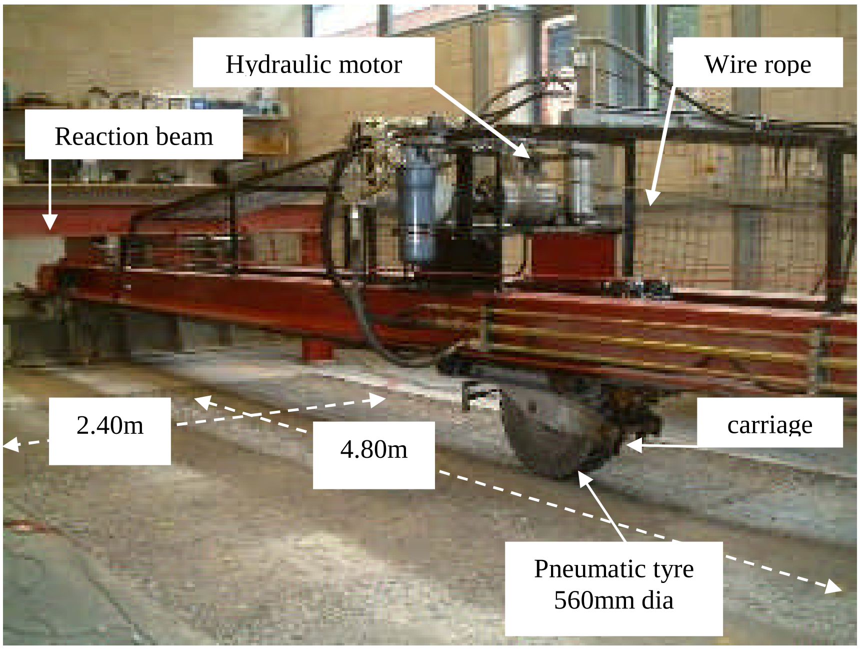


Figure 3.17 - The Nottingham University Pavement Test Facility (NPTF) housed in NTEC.

The crushed limestone and RAP aggregates used were as described in Section 3.2. Mixture design tests were carried out to optimise the properties and/or content of binder for each mixture type. The mixtures were subsequently mixed and compacted at optimum binder contents.

The thickness of the trial pavement was purposely designed to ensure that the pavement would suffer some degradation within a reasonable number of load applications. It was decided to construct the pavement as thinly as practically possible. Thus, based on the compaction criteria, the trial pavement layer thickness was selected at 80 mm which was around four times the maximum aggregate size. The trial layer was paved onto an existing NPTF foundation which consisted of a 450mm crushed limestone sub-base sitting on top of a Keuper Marl clay sub-grade (see Figure 3.19).

Once constructed and cured for a fixed duration (see section 3.4.5.4), the trial sections were trafficked using a single loaded wheel. The performance of the pavement was assessed at frequent intervals by monitoring the magnitude of accumulated permanent surface deformations (rutting) and transient strains at the bottom of the stabilised layer in the wheel path during trafficking.

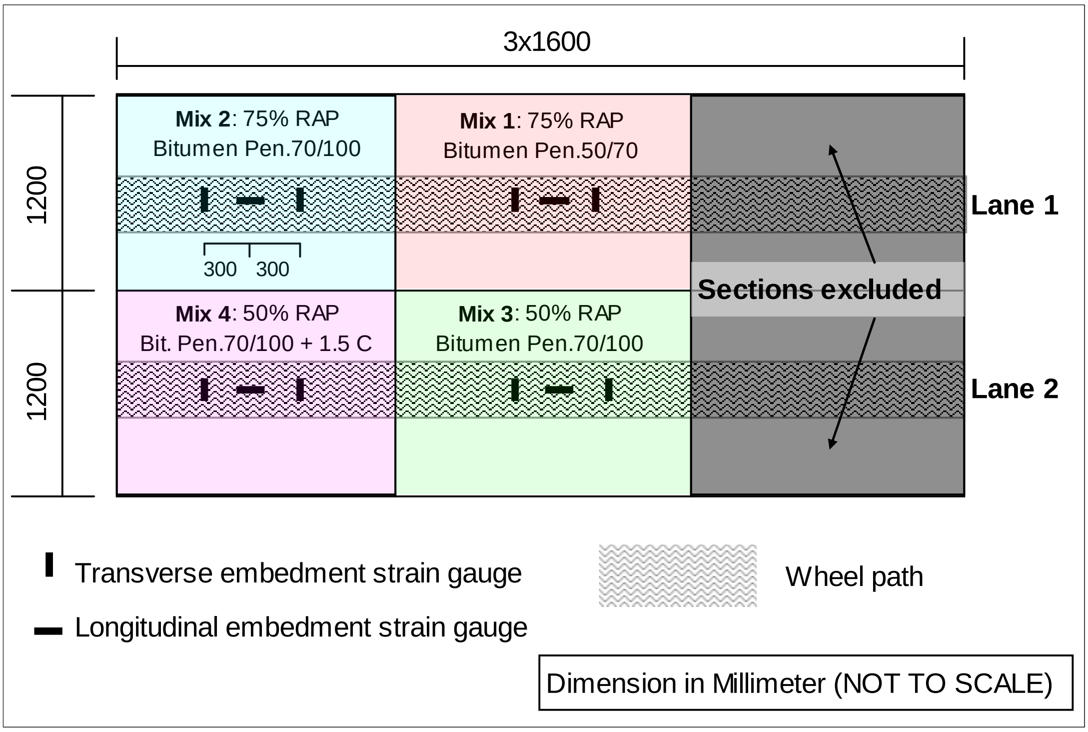


Figure 3.18 - Trial pavement layout

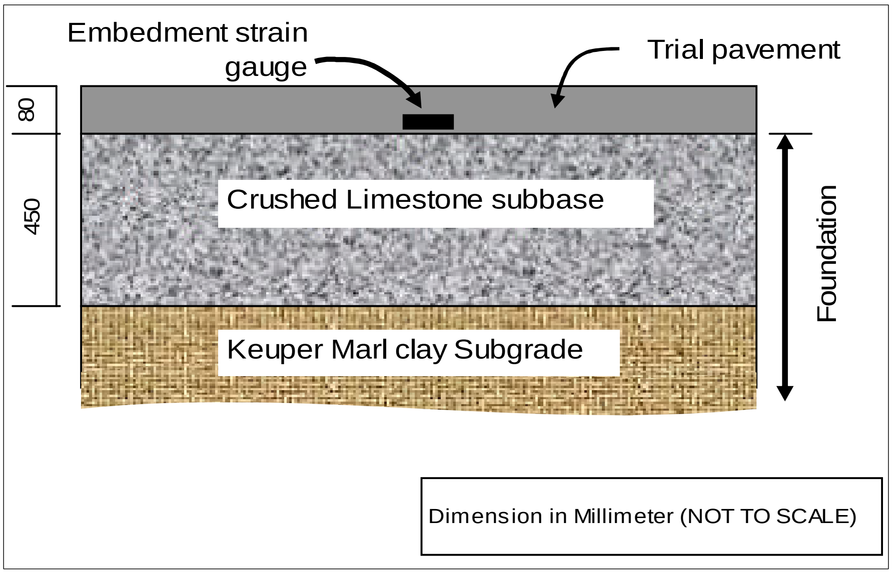


Figure 3.19 - The trial pavement layer laid on top of the existing NPTF foundation

3.4.4 Mixture design

3.4.4.1 Select foam quality

Two penetration grade bitumens, Pen. 50/70 and Pen. 70/100, were selected for the production of foamed bitumen. The basic properties of these bitumens are presented in Table 3.6.

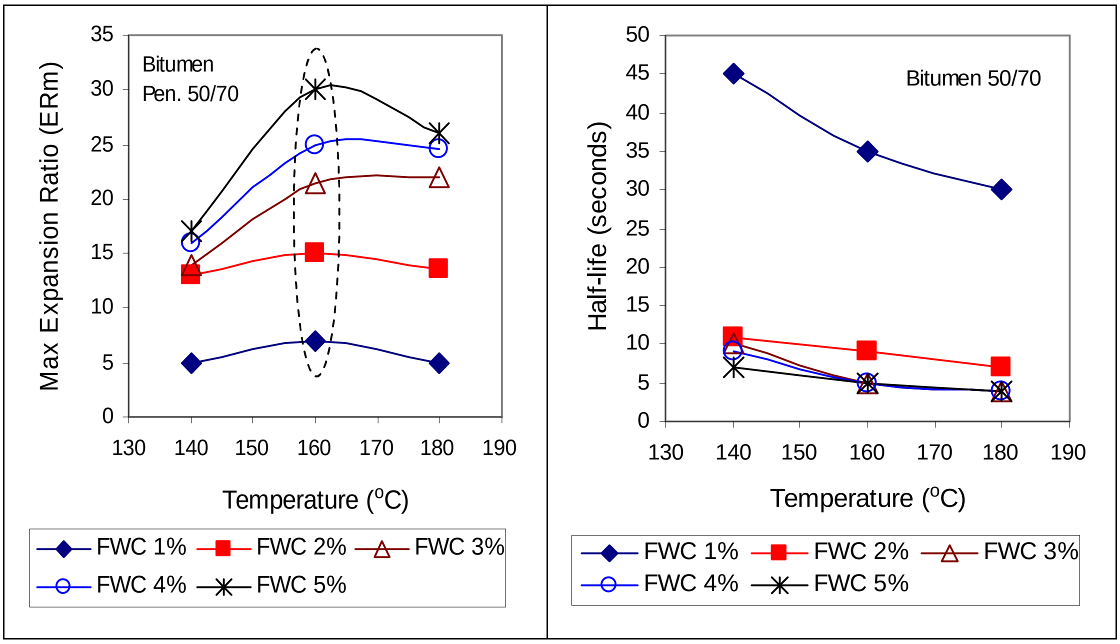


Figure 3.20 - Foaming characteristics of bitumen Pen. 50/70

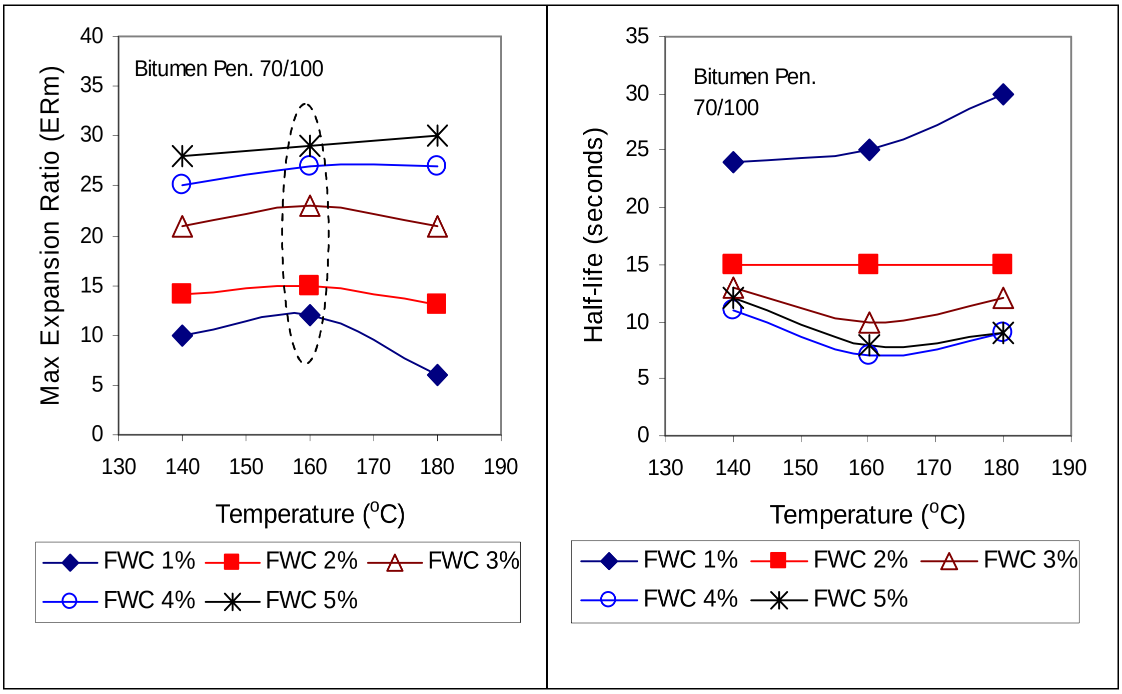


Figure 3.21 - Foaming characteristics of bitumen Pen. 70/100

In order to determine the temperature and foaming water content (FWC) required to produce suitable foaming characteristics, the two bitumens were subjected to foam production at 140˚C, 160˚C and 180˚C and at various water contents using the Wirtgen WLB-10 laboratory foaming plant. The foamed bitumen characteristics are presented in Figure 3.20 and Figure 3.21. Foams generated at a temperature of 160˚C were selected as the most stable for both bitumens since most of their ERm values were higher than others, although the HL values were not the longest.

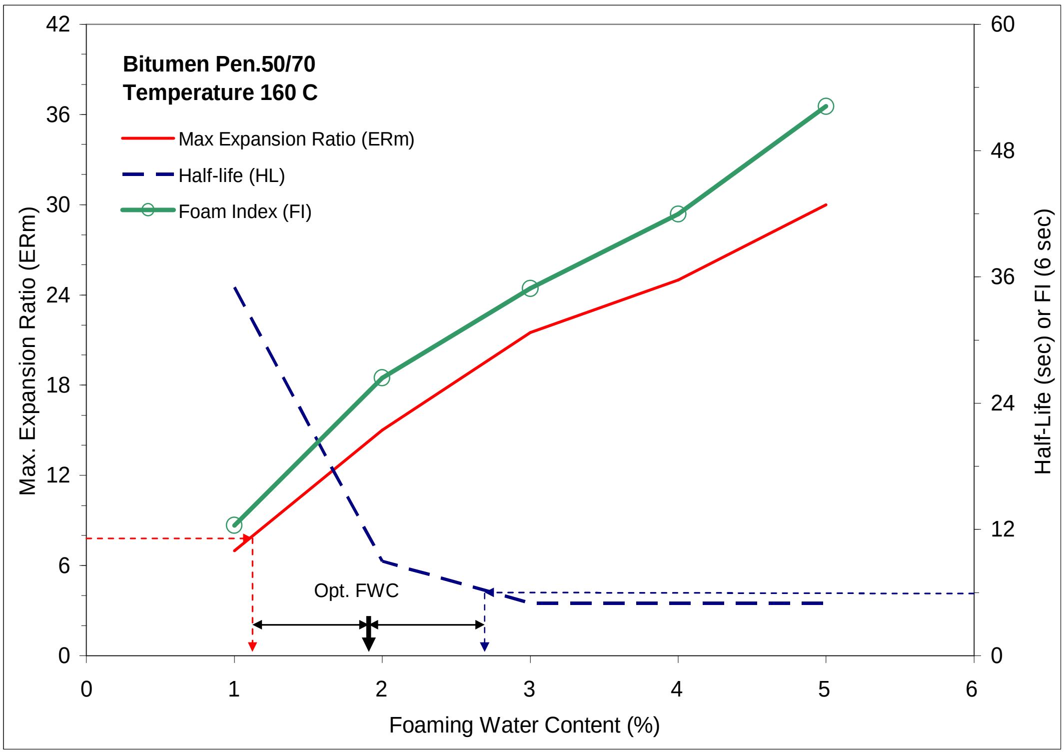


Figure 3.22 - Determine the optimum foaming water content for foam generated using bitumen Pen. 50/70 at temperature of 160^{o}C.

A further step was to select the optimum foaming water content (Opt. FWC) at temperature of 160 ˚C for both bitumens. The Foam Index (FI) concept was initially attempted. However, as shown in Figure 3.22, FI values increased continually with increasing FWC and hence an optimum FWC value was impossible to locate. A similar effect was found for bitumen Pen. 70/100. Therefore the Wirtgen method was used to approach the opt. FWC, although this method is not rigorous. The opt. FWC was found to be around 1.9% for bitumen 50/70 (Figure 3.22) and around 2.2% for bitumen 70/100. A FWC of 2% was therefore applied for both bitumens.

3.4.4.2 Compaction characteristics of mixture

The maximum dry density (MDD) and optimum moisture content (OMC) of each mixture were investigated using modified Proctor in accordance with BS EN 13286-2: 2004. The results are presented in Table 3.10. The mixture using cement was assumed to have the same characteristics as the corresponding 50% RAP mixture without cement since the cement content replaces the filler content.

Table 3.10 - Compaction characteristics of mixture proportion

Parameters	RAP proportion	
	50%	75%
Maximum dry density (Mg/m^3)	2.200	2.020
Optimum moisture content (%)	2.7	2.5

3.4.4.3 Mixing process

Water was added to all mixtures at 72% OMC (see Section 3.3.4.2) and mixed using the dough hook agitator of the Hobart mixer for one minute before and after foam spraying. Bitumen Pen. 70/100 was selected to generate the foams used for mixture design. Approximately 7.5kg of material was mixed for each batch (for 6 specimens).

3.4.4.4 Compaction process

All mixtures were compacted using the Marshall Hammer with 2x75 blows. The mass of each specimen was 1200g, giving 100mm diameter and 63.5mm height.

3.4.4.5 Curing process

All specimens were oven cured at 40^{o}C for 3 days.

3.4.4.6 Property testing

The Indirect Tensile Strength (ITS) and Indirect Tensile Stiffness Modulus (ITSM) were used to determine the optimum foamed bitumen content for 50% and 75% RAP respectively. Figure 3.23 shows the results. It can be seen that the soaked specimens exhibit an opposite trend compared to the dry specimens in the ITSM test. This evidently causes slight difficulties in determining the optimum foamed bitumen content (opt. FBC). Finally, it was decided to select foamed bitumen contents of 2.6% and 2.4% for 50% and 75% RAP respectively.

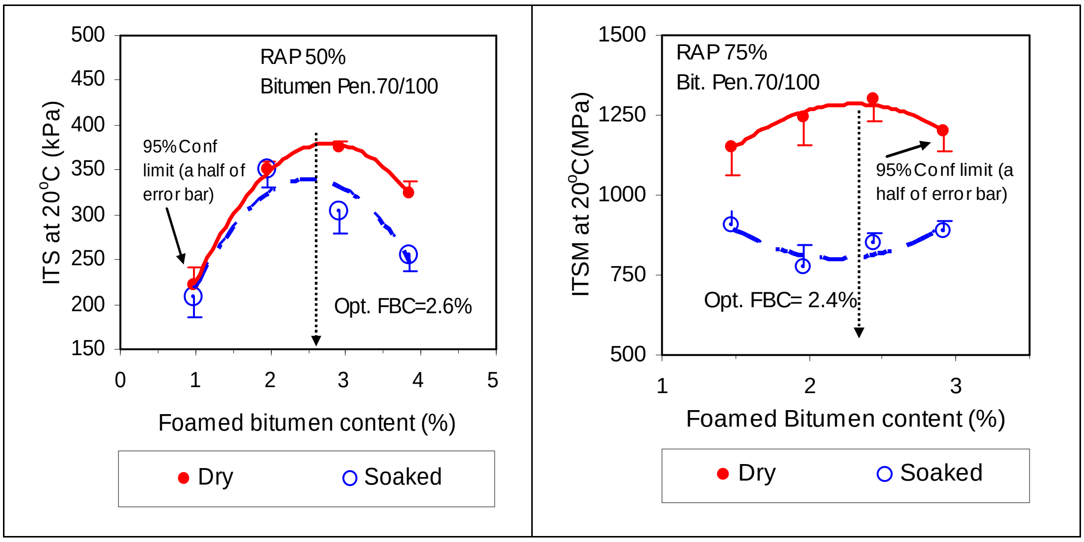


Figure 3.23 - Determine the optimum foamed bitumen content (Opt. FBC) for mixture proportion of RAP 50% and RAP 75%

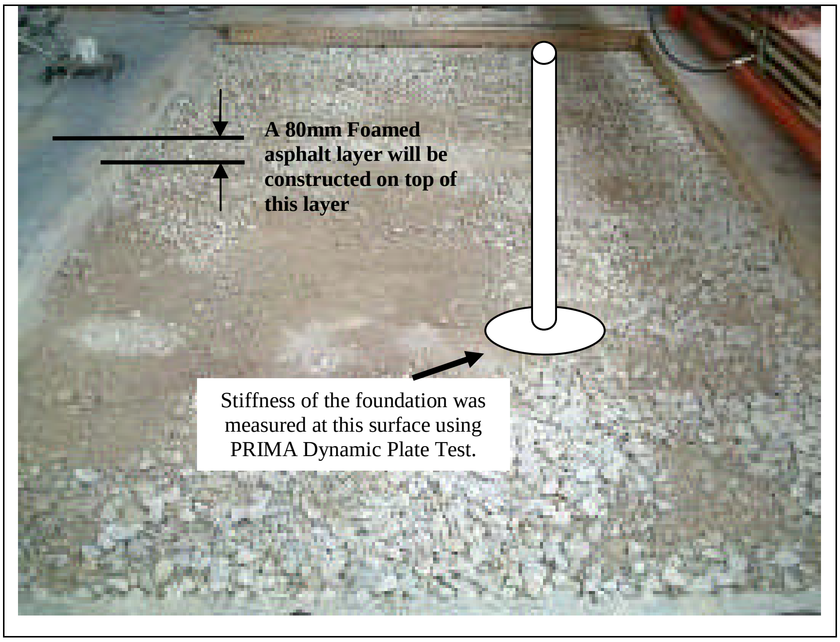


Figure 3.24 - Appearance of crushed limestone surface on which the 80mm foamed asphalt layer will be constructed. The stiffness of foundation was measured at this surface.

3.4.5 Construction procedure

3.4.5.1 Investigate the NPTF foundation

The existing NPTF foundation consisted of a 450 mm crushed limestone subbase and Keuper Marl clay subgrade. Figure 3.24 shows the surface of the crushed limestone subbase layer. The strength of the subgrade was investigated using a dynamic cone penetrometer (DCP). It was found that the CBR values of all sections were less than 1%. This value is less than the minimum CBR requirement for a foundation platform for UK roads up to 5 msa, i.e. 2% (TRL 611, Merrill et al 2004) and may therefore cause excessive rutting under heavy traffic load. A Light falling weight deflectometer (PRIMA dynamic plate test) was also utilised in order to estimate a foundation stiffness value. Measurement was conducted at the surface level of crushed limestone subbase as demonstrated in Figure 3.24. This means the result will represent a stiffness value of the combined subbase and subgrade layer. The results varied from 37 MPa to 81 MPa with an average of approximately 60 MPa.

3.4.5.2 Instrumentation

Embedment strain gauges, two in the transverse direction and one in the longitudinal direction, were installed at the bottom of the recycled pavement layer in each section and all gauges were installed directly underneath the wheel path. The instrumentation processes are shown in Figure 3.25.

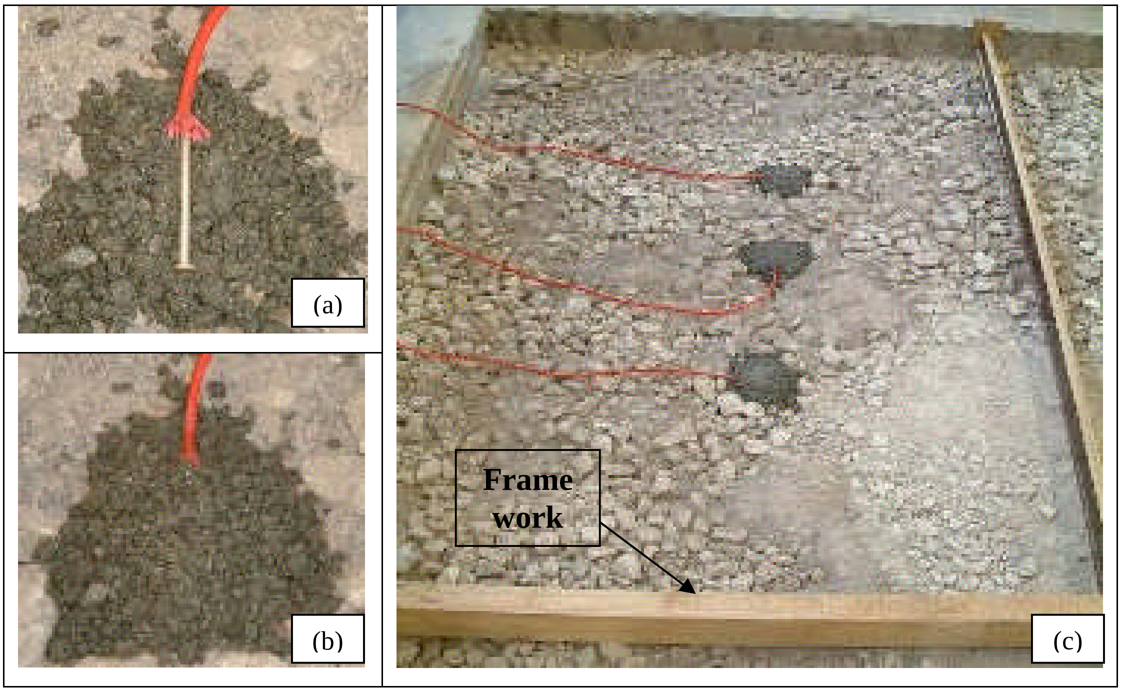


Figure 3.25 - Process of strain gauges instalment at foundation surface; (a) strain gauge placed on a thin foamed asphalt layer, (b) strain gauge covered using foamed asphalt material, (c) three strain gauges were installed in each section.

3.4.5.3 Producing the trial pavement foamed asphalt materials

The materials were weighed and then mixed in a Hobart mixer at the optimum moisture and binder contents based on the mix design results. Foamed bitumen was produced at 160°C with 2% water content. However, due to the limitation of the mixer's capacity, the materials could only be mixed in 6kg batches. Each section needs material weighing approximately 338kg (for 50% RAP) and 311kg (for 75% RAP). Thus the mixtures had to be stored in sealed containers at room temperature (20±5°C) and it took approximately 11 to 15 days to manufacture enough quantity for construction of all the stabilised layer sections in the PTF pit.

For the foamed bitumen plus cement mixture, the RAP and aggregate were treated with foamed bitumen first and the product was stored in sealed containers for up to about 14 days. On the day of compaction, cement and an additional quantity of water were then mixed with the foamed bitumen treated material using a concrete mixer. Figure 3.26 shows the mixing process using the Hobart mixer and concrete mixer.

Figure 3.26 - Mixing process using Hobart mixer for foamed asphalt materials (left) and using concrete mixer for foamed asphalt plus cement (right).

3.4.5.4 Compaction and curing process

The materials were then placed into the NPTF pit, spread and compacted. Segregation was found to be a potential problem during the spreading process in which the uncoated coarse aggregates tended to separate from the mastic (Figure 3.27a).

A Wacker VP1340A plate compactor was used to compact the materials in a single layer (see Figure 3.27b). Before compaction, the moisture content of the materials was measured. The materials were weighed so that following compaction to the required thickness, their dry densities would not be less than 95% of their corresponding maximum dry density values as achieved in the laboratory compactability tests. It was found that the pavement surface could not be levelled perfectly during the compaction process.

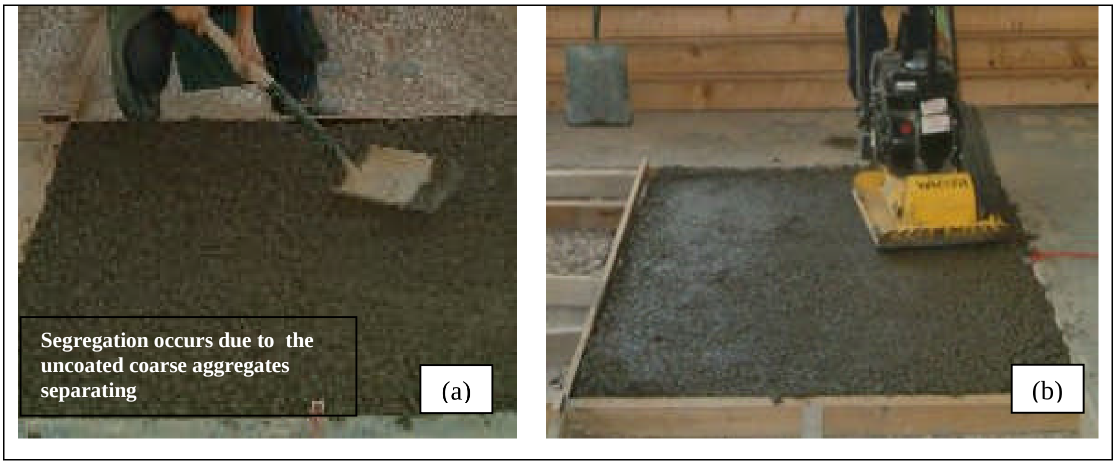


Figure 3.27 - Spreading (a) and compaction (b) process

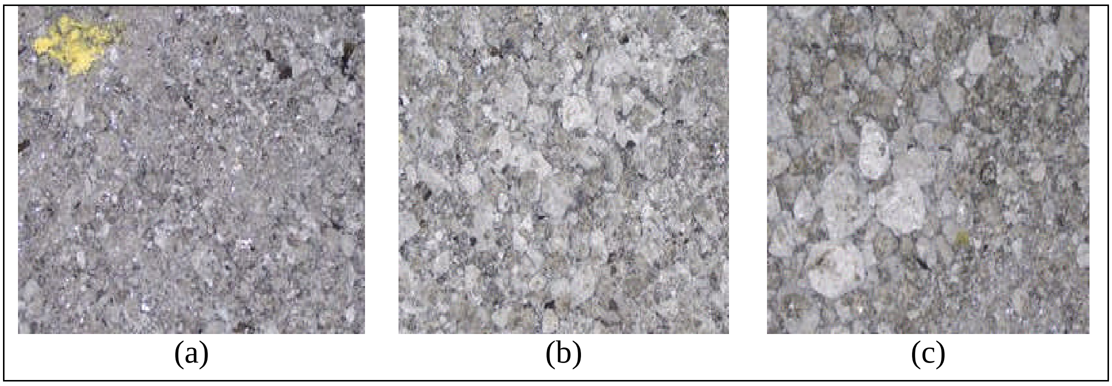


Figure 3.28 - Appearance of foam pavement surface; (a) the cured wheel path surface before trafficking, (b and c) segregation at the section edges.

The time required to lay and compact all the sections in the trial pavement was such that most sections would have been left to cure in the compacted state for at least 13 days before trafficking commenced, and this was adopted as a reasonable target age for start of trafficking. The exception was the section that was composed of foamed bitumen plus cement as the binder, which was unfortunately cured for only 8 days before trafficking due to circumstances beyond this author's control. Figure 3.28

shows the cured pavement surface before trafficking started (a) and the segregation appearance at the section edges (b and c).

3.4.5.5 Summary

All information about construction of the 4 mixture types is summarised in Table 3.11.

Table 3.11 - Resume of construction work

Section	**Mix 2**	**Mix 1**
Mixture type	Foamed asphalt	Foamed asphalt
Composition	75% RAP+25% VCL	75% RAP+25% VCL
Binder	Pen 70/100, FBC 2.5%	Pen 50/70, FBC 2.5%
Storage time	15 days	11 days
Water content at compaction	3.90%	4.20%
Target density	2020 kg/m3 (total mass= 311 kg)	2020 kg/m3 (total mass= 311 kg)
Age at start trafficking	12 days	13 days
Section	**Mix 4**	**Mix 3**
Mixture type	Foamed asphalt + 1.5% cement	Foamed asphalt
Composition	50% RAP+50% VCL	50% RAP+50% VCL
Binder	Pen 70/100, FBC 2.7%	Pen 70/100, FBC 2.7%
Storage time	14 days	11 days
Water content at compaction	4.30%	4.30%
Target density	2200 kg/m3 (total mass= 338 kg)	2200 kg/m3 (total mass= 338 kg)
Age at start trafficking	8 days	13 days

3.4.6 Trafficking

Performance of the trial pavement under traffic was evaluated by repeatedly applying wheel loads onto the pavement. The trial pavement had two lanes. Both lanes were trafficked with an equal number of load applications on each day of testing. This ensured that the performance of the different mixtures could be compared directly without having to consider the effects of differential curing between the various test sections.

Trafficking was carried out at an approximate velocity of 3 km/hr. The number and magnitude of the loads applied to each section are presented in Figure 3.29. The early life strengths of the foamed asphalt were relatively low, and to avoid premature damage, the magnitude of the first applied wheel load was selected at the lowest practical level (3 kN) that can be comfortably applied using the PTF.

The sections were trafficked and the accumulation of permanent deformation was monitored at this load level. When the rate of increase of surface deformations reduced to an insignificant level, the load applied was subsequently doubled in magnitude. In this experiment, the first 5000 passes were at a wheel load of 3 kN, the next 10,000 passes were at a wheel load of 6 kN, and the remaining passes were at a wheel load of 12 kN. Trafficking was terminated when the cumulative number of passes was equal to 45,000 passes per lane. The tyre pressure was 600 kPa for all applications, equivalent to that in typical heavy goods vehicle tyres.

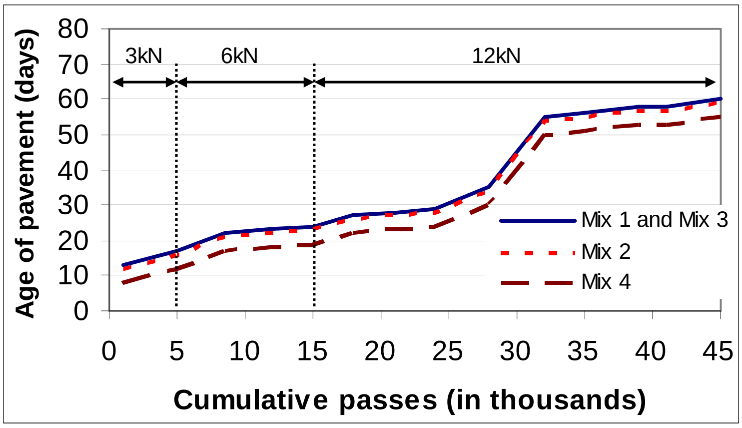


Figure 3.29 - Trafficking schedule

3.4.7 Visual inspection

The strains at the bottom of the trial pavement layer were recorded at intervals of approximately 1000 load applications. The pavement profile was measured using a straight edge (Figure 3.30) and the pavement surface was visually inspected at every 3000 to 4000 wheel passes. After completion of all trafficking, a number of cores were extracted from each section using the dry coring technique to provide samples for laboratory testing.

The sections with no cement additive started to rut as soon as the first load was applied (i.e. 3 kN). Rutting occurred only in the wheel path. Elsewhere, other than in the wheel path, the transverse profile of the pavement remained unchanged. It was also observed that, on every occasion that the magnitude of wheel load was increased, there was a significant immediate rise in rut magnitude. At each load level, the rutting rate was found to gradually decrease with increasing number of wheel passes. After the wheel load was increased to 12 kN, i.e. the maximum load selected in this investigation, and when no more significant increase in rut depth was noticed, it was decided to terminate the test. Trafficking was thus terminated after 45,000 wheel passes per lane.

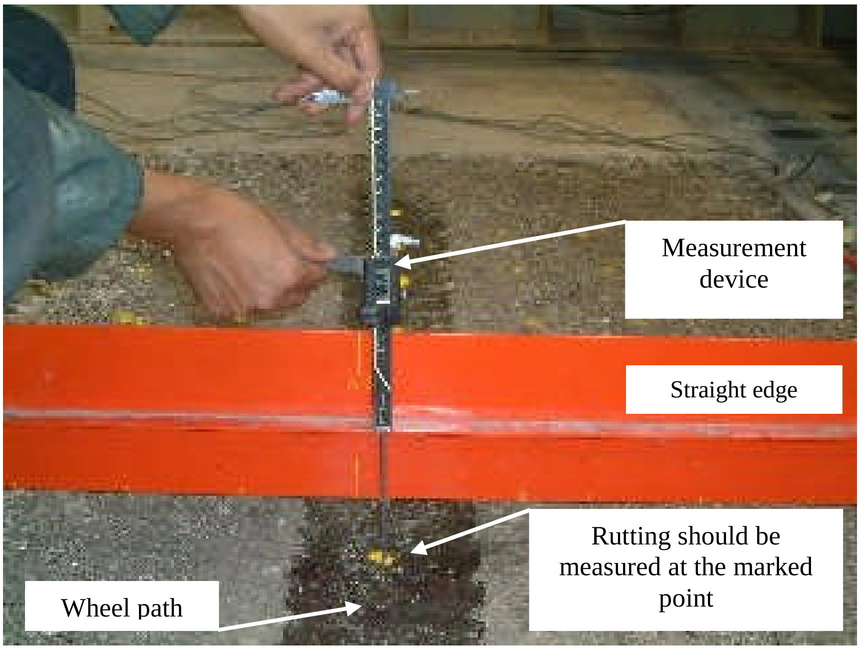


Figure 3.30 - Measurement of rutting using straight edge.

Between 15,000 and 20,000 passes, i.e. soon after the wheel load was increased to 12 kN, longitudinal cracks were observed at both sides of the wheel paths on the sections with no cement (see Figure 3.31). Cracks were not observed in the foamed section with added cement. These cracks were the result of excessive rutting and are not believed to have been caused by fatigue cracking. Due to rutting, tensile stresses and strains developed in the top layer of the shoulder of the rutting. These tensile strains, which are a function of the depth of the rutting or the height of the shoulder of the rutting, generate longitudinal cracking.

The excessive rutting at the wheel path was accompanied by bleeding, i.e. excess binder on the surface, as shown in Figure 3.32. At high load pressure, the uncoated coarse aggregates probably separate from the foam mastic, hence the coarse aggregates go down while the binder goes up, as seen on some the cored samples.

Figure 3.31 - Appearance of longitudinal cracks observed at both sides of the wheel path (coloured black).

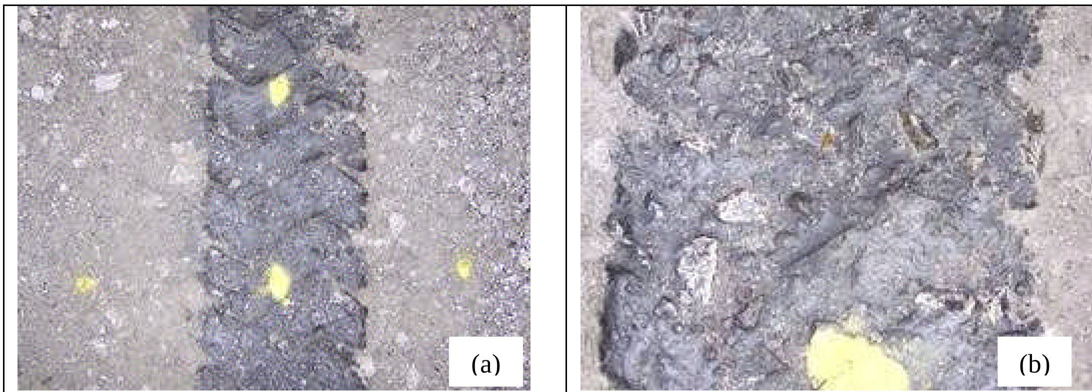


Figure 3.32 - Appearance of rutting in the wheel path; (a) Wheel texture was clearly evident along the wheel path, (b) formation of bleeding.

3.4.8 Rutting measurement

The surface rutting of each section was measured at 7 points at equal intervals along the wheel path as shown in Figure 3.33. It can be seen that the initial surface profile along the wheel path was not flat. However, based upon the target level (106mm depth below reference straight edge), the calculated average pavement thickness along the wheel path was close to 80mm, as shown in Table 3.12.

Table 3.12 - Average pavement thickness along the wheel path

Mixture type	Average thickness (mm)
Mix 1: 75%RAP, Pen50/70	79
Mix 2: 75%RAP, Pen70/100	76
Mix 3: 50%RAP, Pen70/100	81
Mix 4: 50%RAP, Pen70/100 + cement	77

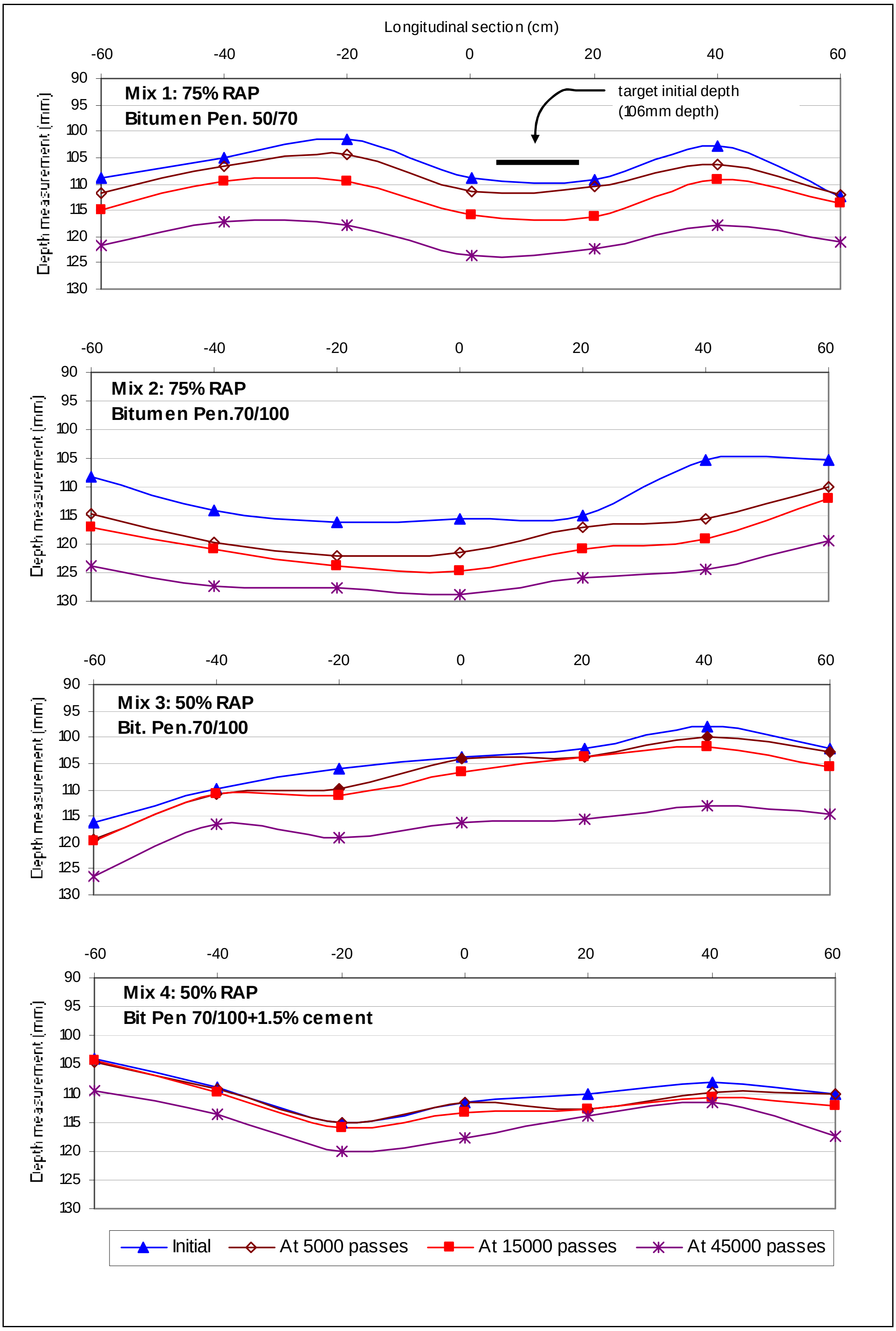


Figure 3.33 - Longitudinal profiles for each section.

The average surface rutting of the 7 points along the wheel path of the trial sections is shown as a function of load applications in Figure 3.34. Rutting was developed to varying degrees in all foamed bitumen bound sections. In terms of the amount of rutting, the mixtures can be ranked as presented in Table 3.13.

Table 3.13 - Ranking of rutting

Mixture type	Ranking
Mix 4: 50%RAP, Pen70/100 + cement	1
Mix 3: 50%RAP, Pen70/100	2
Mix 1: 75%RAP, Pen50/70	3
Mix 2: 75%RAP, Pen70/100	4

The differences in the rut depth values and profiles between different mixtures indicate that the rutting potential of stabilised mixtures depends on the binder type, mixture proportion and the presence of cement. In general, foamed bitumen bound mixtures that contained a higher proportion of RAP and a softer binder exhibited greater deformation. A mixture using a higher RAP proportion implies a higher total binder (old and new) content due to the presence of old binder in the RAP. This will give less resistance to deformation since the binder film around the aggregate particles becomes thicker (see Chapter 2 section 2.3.2).

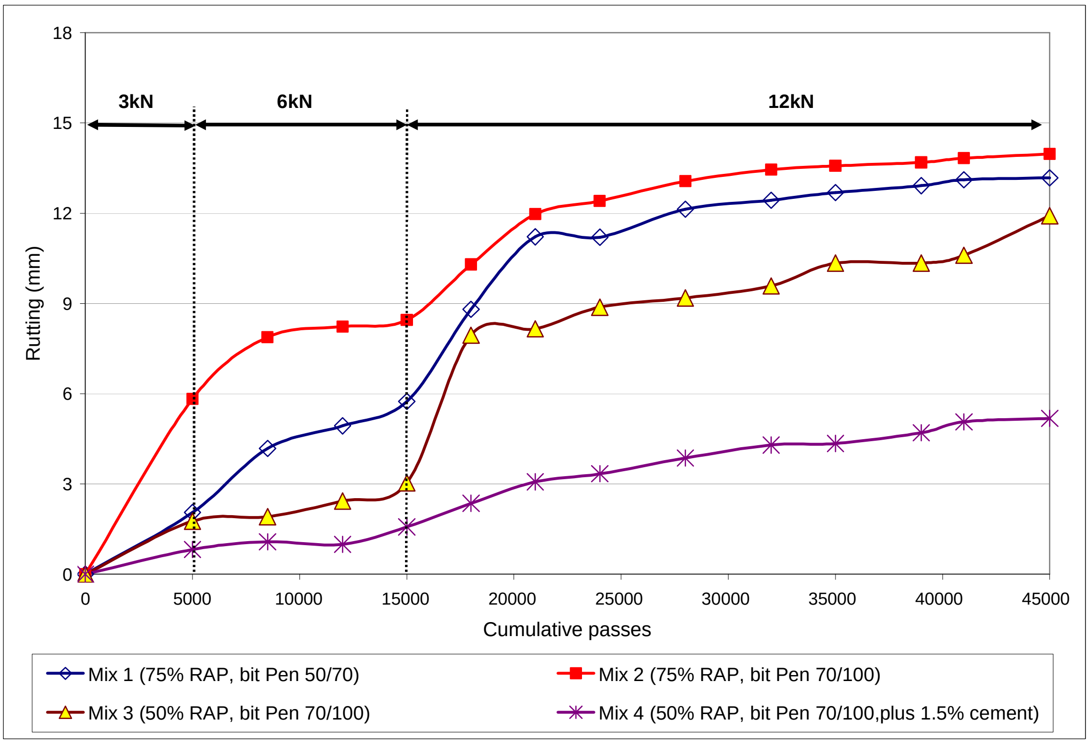


Figure 3.34 - Average surface rutting for each mixture type

The effect of mixture proportions can be seen by comparing Mix 2 and Mix 3. Up to the first 5000 load applications, just before the load level was increased to 6 kN, the average rut depth in Mix 2 was approximately 6 mm and that of Mix 3, which contained less RAP, was 2 mm. At 15,000 load applications, the average deformation of Mix 3 was still about one third that of Mix 2. However, when the load was increased to 12 kN, the difference in rut depth between these two mixtures decreased. The final deformation at 45,000 load applications of Mix 2 was about 14mm while the deformation of Mix 3 was about 12mm, i.e. a difference of only about 2mm. The effect of mix proportions on permanent deformation was more pronounced during early life and at low loads, but when the mixture was subjected to higher loads, this effect was less significant. The mixture using a higher RAP proportion had lower compactability (at the same aggregate gradation it gave a lower maximum dry density, see Table 3.10) and hence probably resulted in higher void content which allowed greater deformation in the form of secondary compaction.

The penetration grade of the binder also affected the permanent deformation during the early life and at low load. This effect can be shown by comparing the performance of Mix 1 and Mix 2, which contained the same amount of RAP but different binder grades. After the pavement had been subjected to 5000 passes at 3 kN load, the rut depth of Mix 1, containing the harder grade binder, was about 36% that of Mix 2. When the pavement was loaded with another 10,000 passes at 6 kN, the rut depth of Mix 1 increased to about 67% that of Mix 2. During the application of the 12 kN wheel loads, although Mix 2 deformed on average by a slightly greater amount, the permanent deformations of both materials were approximately equal with a difference of 1mm or less. The harder penetration grade bitumen produces a foam that improves the mixture resistance to permanent deformation. However, when the mixture is subjected to higher loads, a harder penetration grade bitumen makes little difference compared to a softer grade bitumen. This implies that at a higher load, the mixture resistance to permanent deformation is mainly affected by aggregate properties rather than bitumen properties.

The RAP and virgin aggregate particles in the foamed bitumen bound mixtures, though bound together, were not necessarily fully coated by the bitumen as would be the case in a typical hot asphalt mix. Also noticeable was the visibly voided nature of the mixtures. The evidence of rutting in foamed bitumen bound material is mainly due to densification (see Eisenmann and Hilmer, 1987) and the weakness of bonds during early life (see Zoorob and Thanaya, 2002 and Merrill et al, 2004). Mix deformation was therefore likely to depend on both particle interlock as well as binder stiffness. At low stress levels, the binder contribution to the mix response was relatively high resulting in improved response with the use of a harder grade binder. On the other hand, at high loads, the binder contribution to the mix behaviour was less evident and the deformation behaviour of the mixtures was primarily governed by the aggregate interlock regardless of the binder grade.

The effect of adding a small amount of cement is clearly observed from the performance of Mix 4. The magnitude and rate of deformation of Mix 4 was clearly smaller than that of all the other foamed bitumen mixtures. When the test was terminated at 45,000 load applications, all three foamed bitumen bound sections with no added cement developed significant surface deformation with an average rut depth greater than 12mm. On the other hand, the foamed bitumen bound section containing 1.5% cement deformed on average by only 5mm. This 60% decrease in rutting indicates a significant increase in permanent deformation resistance caused by the inclusion of cement as an additive. A small amount of cement can effectively work to accelerate the moisture loss and hence the mixture strength gains faster at early age, reducing the measured rutting.

3.4.9 Strain at bottom of the trial pavement layer

3.4.9.1 Characteristics of strain response

Figure 3.35 shows the strain responses for all sections during trafficking. There were 12 gauges in total and 10 of those gauges survived the compaction process undamaged. Each section had two transverse and one longitudinal strain gauge, and this allowed the elimination of data from gauges generating unreliable data.

As expected, and except for the response of Mix 2, the deflections and hence tensile strain values of all sections increased as the wheel load was increased in magnitude. This can be seen as sudden jumps in the strain profiles in Figure 3.35. The unexpected readings from the gauges in Mix 2 may imply lack of cohesion between the surface of the gauge and the surrounding mixture.

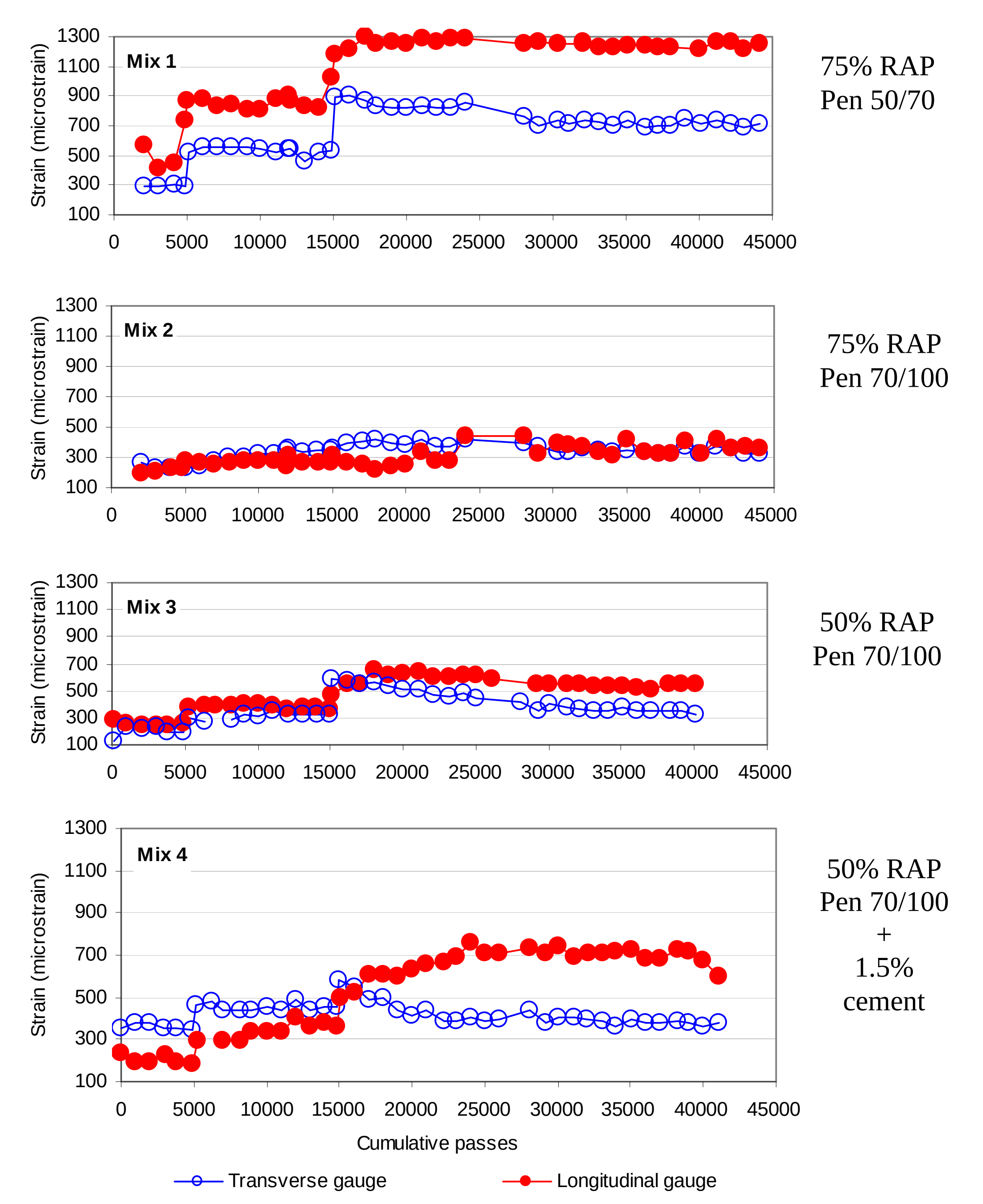


Figure 3.35 - Measured transient tensile strain results

The recorded tensile strains appeared to be relatively stable during trafficking under wheel loads of 3 and 6 kN. This was followed by a gradual reduction in strain response with time during the 12 kN wheel load trafficking. This is quite reasonable as the strength of the foamed bitumen bound materials increased with time due to the curing effect. These trends were not observed clearly from the strain gauges of Mix 2 for which the strain response increased continually during wheel loads of 3 and 6 kN, nor from the longitudinal gauge of Mix 4, for which the strain response tended to increase under the wheel load of 12 kN.

3.4.10 Back calculated modulus

In this study, an attempt was made to estimate the modulus of the stabilised layers. This problem was approached with the aid of a Multi-layer linear elastic analysis software "BISAR" (Shell 1998). This software is normally used as a forward calculation tool in pavement analysis. Using BISAR, the stresses and strains can be calculated at any point within the structure for any multi layered construction, assuming each layer is infinite in the horizontal direction, knowing the thickness, modulus and Poisson's ratio of each of the layers and the magnitude and locations of the applied loads.

To calculate the modulus of a stabilised layer based on the measured strains and using BISAR software, the following procedure was devised:

1 Simplify the layer system from an actual 3-layered construction to a 2-layered system, since the individual moduli of the granular sub-base and soil sub-grade layers were unknown (see Figure 3.36). Using the dynamic plate test over the sub-base surface, the combined modulus of the sub-base + sub-grade (= foundation) was measured at about 60 MPa. Therefore the pavement was analysed as a 2 layered system, in which the stabilised material is referred to as the first layer with a thickness of 80 mm and the second layer has a modulus 60 MPa with an infinite thickness.

2 Generate a small data-base relating predicted tensile strain values underneath the wheel to a range of stabilised layer moduli values at each of the 3 wheel loads applied. These data can be fitted to develop a strain-modulus equation at each applied wheel load. Figure 3.37 shows the relationships resulting from this step.

3 Calculate the modulus of the stabilised layer using the strain-modulus equation, at each wheel load, in which the experimentally measured strain value becomes the input data.

Figure 3.38 presents the results of the modulus calculations. The results from gauges in Mix 2 are included as an indication only, not to be compared with others due the unreliability in strain gauge response. The modulus results appear to span a relatively wide range. This was primarily caused by the scatter of output from the various strain gauges within each test section. It is also easy to appreciate that the responses were greatly affected by fluctuations in load, plus variability in layer thickness and foundation stiffness across the test pit. The calculated stiffness was then not very rigorous, e.g. Mix 4 was not found to be highest stiffness as expected.

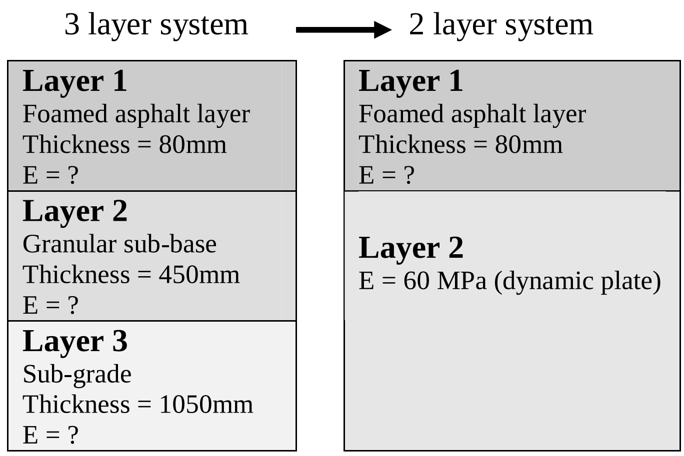


Figure 3.36 - Simplify the layer system

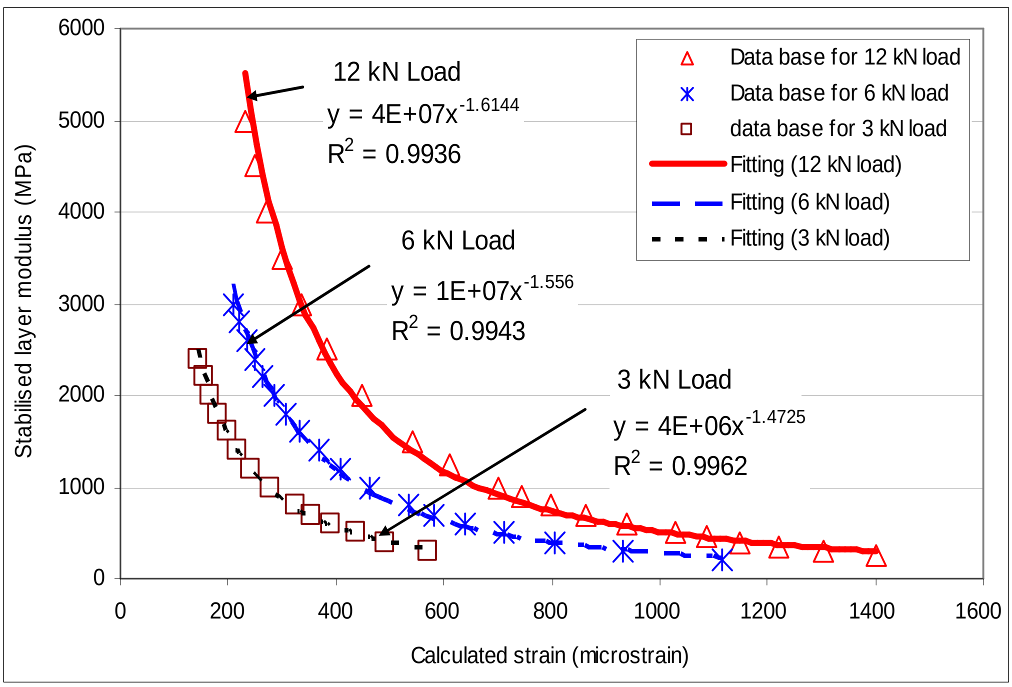


Figure 3.37 - Calculated strain-modulus relationships for a stabilised layer based on a 2 layered BISAR model.

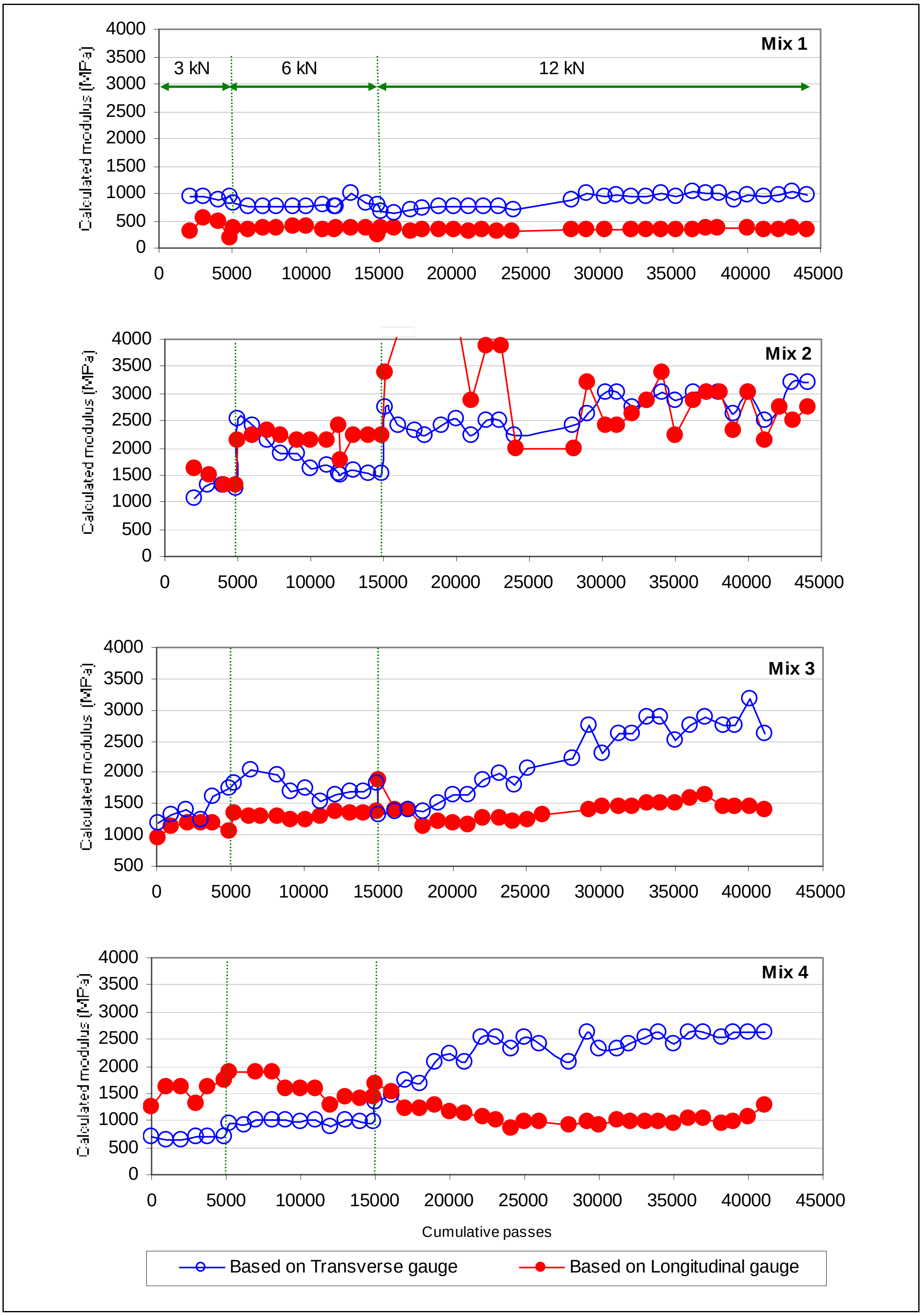


Figure 3.38 - The calculated modulus values of the stabilised layer

Taking an overview of all the results, the modulus of an 80 mm foamed asphalt layer on a 60 MPa foundation at early life is a function of mixture composition and extent of curing. Foamed bitumen bound recycled asphalt mixes had stiffness values ranging from around 300 MPa at the lower end to about 2500 MPa at the upper end (the results of Mix 2 are not included).

3.4.11 Cored specimens

At the conclusion of the trafficking phase, the trial sections were cored. Unfortunately, it was not possible to conduct coring with any ease because the aggregates were only weakly bound by foamed bitumen. Wet coring was impossible because the water would damage the bonding of the material, and therefore it was decided to use a dry coring technique. Initially, a coring barrel diameter of 100mm was tried but this did not produce any intact cores. Using 150mm diameter, coring was only slightly more successful. Coring was carried out in the wheel path (trafficked) and away from the wheel path (un-trafficked). It was possible to core Mixes 3 and 4 at both positions; however all the un-trafficked cores obtained from Mixes 1 and 2 were damaged during the coring process. Figure 3.39 shows the appearance of cored specimens. The stiffness modulus of the cored specimens was measured using the ITSM test at a temperature of 20^{o}C and the results are shown in Figure 3.40. The calculated modulus values at the end of trafficking for each section are also superimposed on this figure.

The ITSM stiffnesses of the un-trafficked specimens in Mix 4 are about three times higher than those of the trafficked specimens. The lower end of the calculated modulus band for this section coincides with the average trafficked ITSM value (about 500 MPa). However, in the case of Mix 3 the ITSM stiffnesses of both trafficked and un-trafficked specimens were very similar values (about 300 MPa), and far lower than the calculated modulus values. Sample disturbance and damage during coring of low stiffness mixtures such as Mix 3 are likely to have caused a reduction in the measured ITSM values. This applies to both the trafficked and un-trafficked sections. The trafficked specimens ITSM stiffness values in Mix 1 also coincided with the lower limits of the calculated modulus for this section (about 400 MPa), whereas the average ITSM values of the trafficked specimens of Mix 2 (about 400 MPa) were far lower than the calculated modulus values. In general, the stiffness values of trafficked and un-trafficked specimens can not be compared precisely due to sample disturbance and damage during coring.

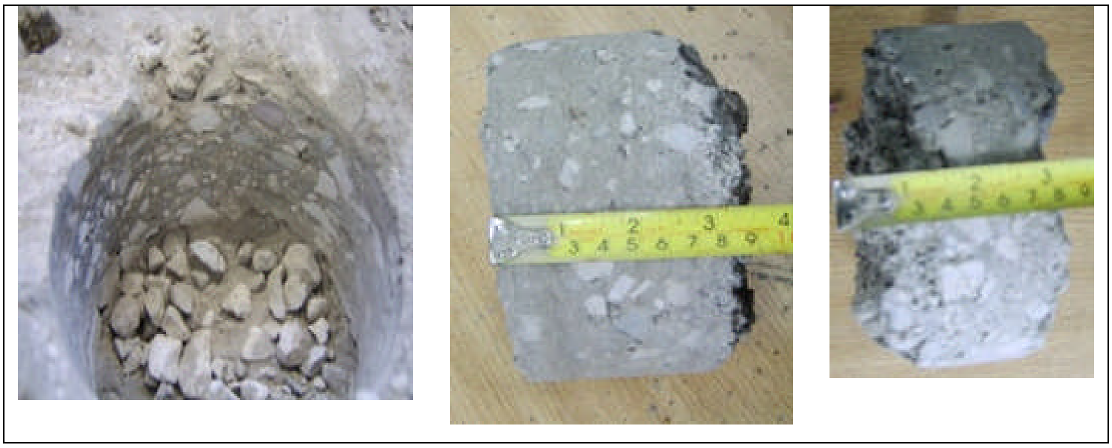


Figure 3.39 - Appearance of cored specimen; (a) coring hole, (b) thick cored specimen, (c) thin cored specimen.

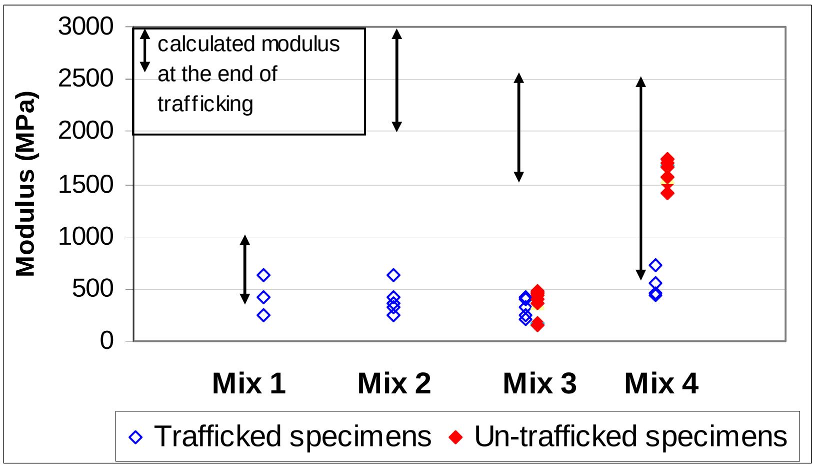


Figure 3.40 - Comparison between actual ITSM values of cored specimens from the four foamed bitumen stabilised sections and the calculated modulus limits from strain gauge readings.

3.5 Discussion and conclusions

Following the work described in this chapter, it can be deduced that foamed asphalt material has definite potential for use in road pavements. When this material is put down as a pavement layer, it exhibits good structural capability to support traffic load. The predicted stiffness modulus values indicate that foamed asphalt material has good ability to spread load and thereby reduce stress concentration on the underlying layer. The material also demonstrates excellent fatigue resistance (no fatigue cracking was observed) which indicates the flexibility of the material. The evidence of rutting in foamed bitumen bound material is mainly due to densification and the weakness of bonds during early life. Unlike hot-mix asphalt materials, cold-mix asphalt material using foamed bitumen requires a curing period to gain its final properties. Commonly, hydraulic agents such as cement are added to cold-mix to accelerate the curing process and enhance the strength of the material. This phenomenon was clearly observed in the pilot scale work, that a small amount of cement added to the foamed bitumen bound mixture significantly reduced the measured rutting. The process of foamed asphalt manufacture is also easily handled, clean and can be stored as required prior to compaction.

Foamed asphalt is a unique mixture. Not all aggregate particles are coated by binder. The sprayed foamed bitumen enables coating of wet aggregates and is seen on fine particles only. If the predetermined aggregate moisture is incorrect and the quantity of fine particles is insufficient, the resulting mixture becomes unacceptable (see Brennen et al, 1983 and Ruckel et al, 1982). Moreover, if both moisture and fines have been prepared correctly, but this is not accompanied by proper design of selected foamed bitumen characteristics (see also Muthen, 1999) and suitable mixing (see also Long et al, 2004); the resultant mixture will be inconsistent and hence its performance will be unpredictable. Thus, mixing plays an important role attaining the optimum end product performance. Therefore, the role of the mixing process in foamed asphalt manufacture must be properly understood.

It is definitely necessary to select the best foamed bitumen quality in order to provide the most homogenous mixture and the best coating quality between binder and aggregate particles. However, the current parameters used to characterise foamed bitumen are questionable and hence the method of selecting the optimum foamed bitumen characteristics remains a problematic issue. Lack of understanding of foamed bitumen characteristics in association with the manufacturing process and in-service performance of the mixture is considered to be the root problem.

4 INVESTIGATING FOAMED BITUMEN CHARACTERISTICS

4.1 Introduction

This chapter discusses foamed bitumen characteristics which are explored based on theoretical and experimental studies. The theoretical study is an investigation of foam in a general context, including foam definition, structure, modulus, rheology, life time and collapse, surface tension and drainage; whereas the experimental study covers a laboratory investigation of maximum expansion ratio (ERm), half-life (HL) and flow rate using foamed bitumen samples produced using a laboratory foaming machine. The results from both studies are combined to increase understanding of the essential properties of foamed bitumen as a binder in foamed asphalt mixtures.

4.2 Understanding of foam in a general context

Foams can be found in a wide variety of contexts. They occur in the form of soap froth, fire-fighting foam, the head on a glass of beer, the froth in a washing-up bowl, and many more. Practically, foams are important, for example, in cleaning, dampening explosions and collecting radioactive dust. In these applications, the ability of foam to spread a small amount of liquid over a wide area is important. In any case, it is important to understand the bulk properties of foam and how its constituents affect it. In the case of foamed bitumen, it is desirable to know what properties are most significant in their effect on foamed asphalt mixture performance.

4.2.1 Definition of foam

Foam is a combination of gas and liquid in which the gas bubbles are separated by thin liquid films and the volume fraction of the liquid is small. The principal distinguishing characteristic of foam is the large volume fraction of the discontinuous gas phase so that its density is relatively low. Foam viscosity is found to be relatively higher than its components and dependent on its density. Therefore, the quantity known as 'kinematic viscosity', the ratio between the viscosity and the density, is likely to be more suitable to characterise foam consistency. Foam is also known as a compressible material due to the gas constituent being compressible in nature. Because of the density difference between the gas and liquid in the foam, the liquid fraction (the denser phase) always tends to drain out of the foam body.

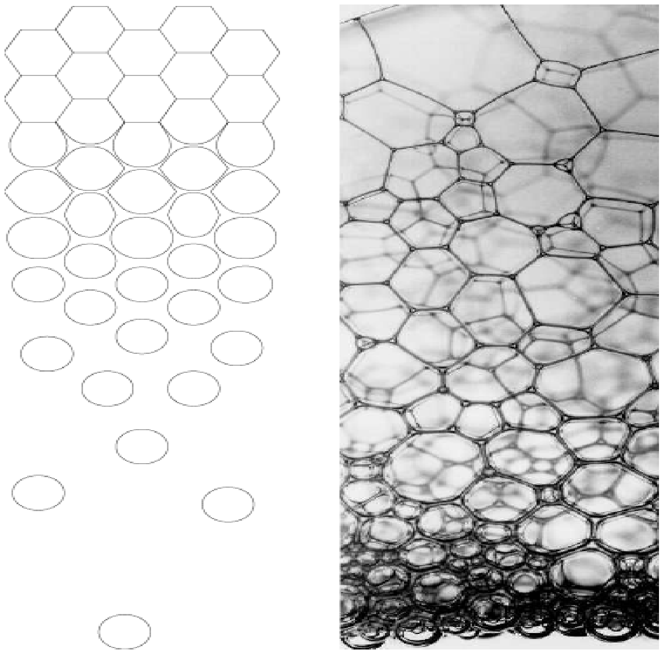


Figure 4.1 - An example of a foam in a column frame which forms a transition from wet foam in the bottom to dry foam in the top. (Left) Two dimensional and (right) three dimensional picture (Schick, 2004)

As described in Chapter 1 (see section 1.1.1), Schramm (1994) and Breward (1999) have defined the structure of foams, which are broadly divided into wet and dry foams. Figure 4.1 shows an example of a foam in a column frame. In this formation, the bubbles tend to rise to the top and the liquid fraction tends to fall due to gravitational effects. Consequently, a transition is created from wet foam (at the bottom) to dry foam (at the top). In two dimensions (Figure 4.1 left), it can be seen that bubble shapes in wet foam are approximately spherical, while in dry foam, the bubbles are more polyhedral.

Foams are generally characterized according to their quality (Fq) as defined in Eq. 4.1.

$$Fq = \frac{Vg}{Vg + Vl} * 100 \qquad \text{……… Eq. 4.1}$$

where: *Fq* is the foam quality (%), *Vg* is the gas volume, *Vl* is the liquid volume.

The onset of bubble motion (the starting point for wet foam) has been found at a foam quality of 52% (Mitchell, 1971) or 60% (Weaire et al., 1993). The transition between wet and dry foam may be at a quality of 75% (Rankin et al, 1989) or 87% (Weaire et al., 1993). This transition should be at the densest possible spherical bubble packing. Based upon a face-centred cubic system (by calculation), this limit is found to be at 74% quality, which is lower than those quoted by Rankin et al (1989) and Weaire et al. (1993). However, the experimental result may be more accurate due to bubbles being arranged randomly. Stable foam can be observed up to 96% (Rankin et al, 1989), but when the quality exceeds this point the foam becomes unstable. Thus, foam quality gradation and the limits of wet and dry foam can be summarised as shown in Figure 4.2.

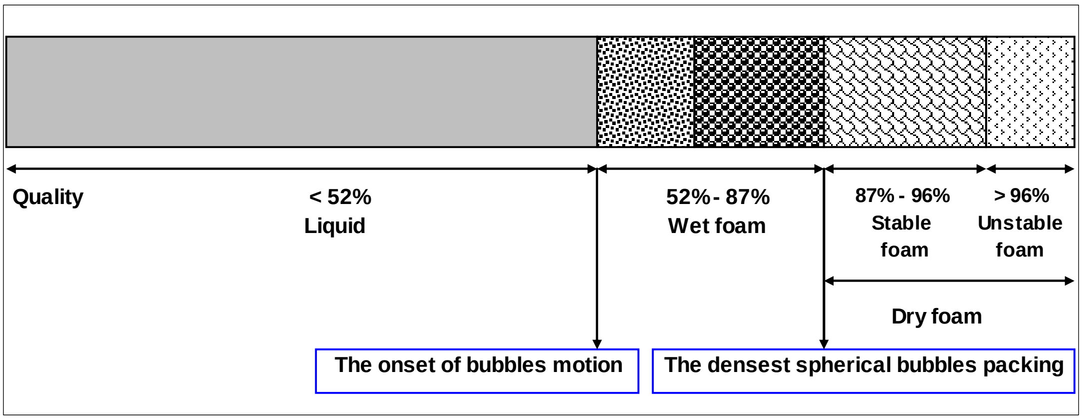


Figure 4.2 - Gradation of foam quality

4.2.2 Structure of foam

Figure 4.3 illustrates the structure of foam. In a wet foam, gas bubbles are dispersed in the liquid phase and they are separated from each other due to the liquid volume still being large. In a dry foam, the thin films forming the faces of the roughly polyhedral bubbles are called lamella and the tubes of liquid at the junctions between these films are called Plateau borders (named after Plateau). The vertices where the Plateau borders meet are called nodes. Foams may consist of bubbles with a wide distribution of sizes, randomly mixed and arranged. Figure 4.4 shows a random foam structure. Discussion of foam structure can be found in Weaire and Hutzler (1999).

Foam can only be formed if surface active materials (surfactants) are present. A foam lamella consists of a thin liquid slab stabilized by two amphiphilic adsorption layers. In a wet foam lamella, both adsorption layers are separated by a fairly thick liquid slab (Koelsch and Motschmann, 2005). In an extremely dry foam lamella, the liquid becomes so thin so that molecular forces arise due to the interaction of the two free surfaces. If these forces are repulsive, this culminates in the formation of a stable film of thickness between 10 and 100 Angstroms (1 Angstrom= 10^{-10} m) (Breward, 1999).

It is noted that a molecule of surfactant is amphiphilic by means of both a hydrophobic and a hydrophilic part. At sufficiently high bulk concentrations, the surfactant molecules form micelles in which the hydrophobic 'tails' are surrounded by hydrophilic 'head' (see Figure 4.5a). Basically, the surfactant molecules prefer to be present at an interface rather than within the body of the liquid. In this case, the tail groups are in the gaseous phase while the head groups remain within the liquid phase (see Figure 4.5b). This arrangement potentially reduces the surface tension of the interface. However, if the surfactant molecules form micelles, they will not be able to affect the surface tension and hence will not benefit the foam properties.

4.2.3 Foam modulus

Elastic modulus may be an important property for foam stability, since a stiff film limits bubble rupture (Bauget et al, 2001). Under low applied stress, foam behaves as a solid and displays elastic behaviour that depends upon bubble size and quality. The elastic (shear) modulus is small and depends on foam surface properties. The modulus comes from the surface tension present on the foam film (lamella). As the stress is increased, the foam response becomes increasingly plastic. Beyond a certain yield stress, the foam starts to flow. This foam stress-strain behaviour has been discussed clearly in Weaire and Hutzler (1999) based on many experiments of general foams (see Figure 4.6).

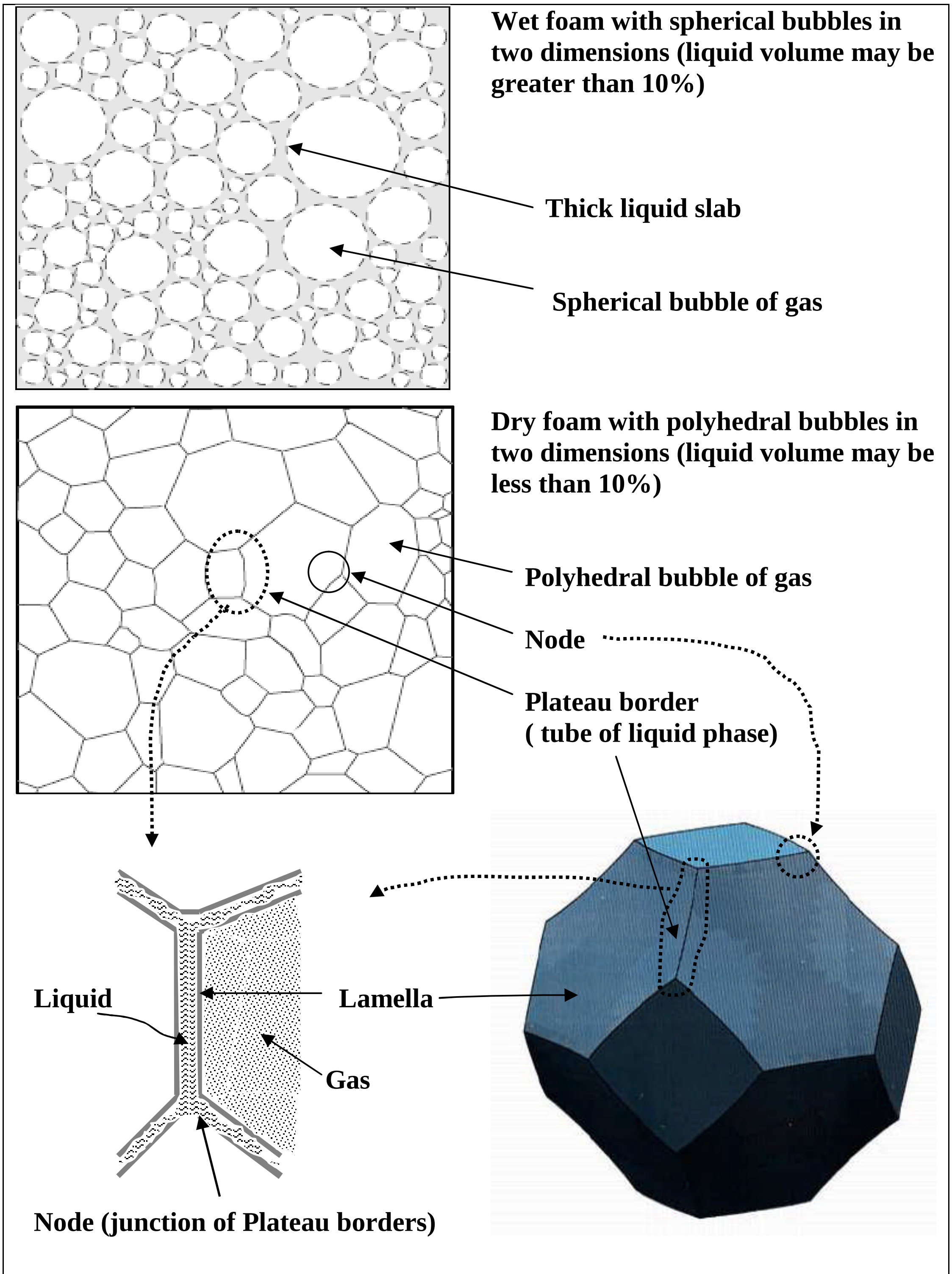


Figure 4.3 - Structure of wet and dry foam

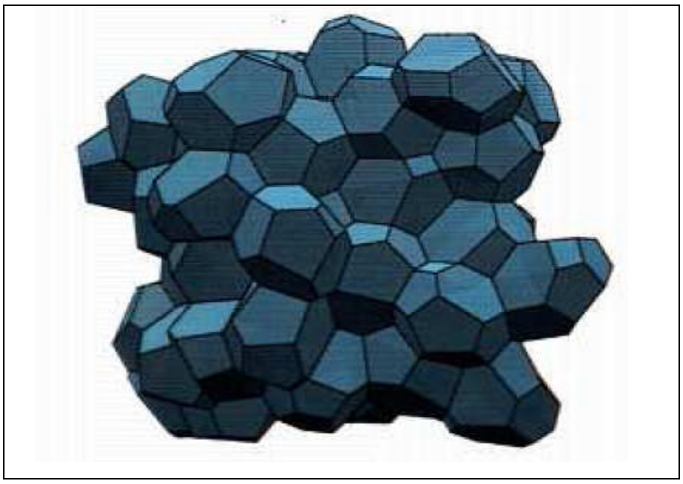


Figure 4.4 - A random foam structure (Breward, 1999)

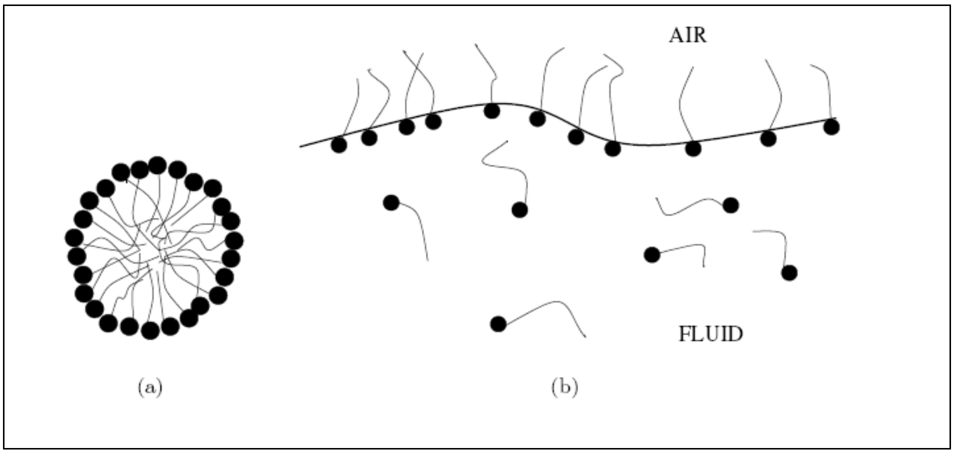


Figure 4.5 - Surfactant molecules (a) forming a micelle within the liquid and (b) at a free surface (Breward, 1999)

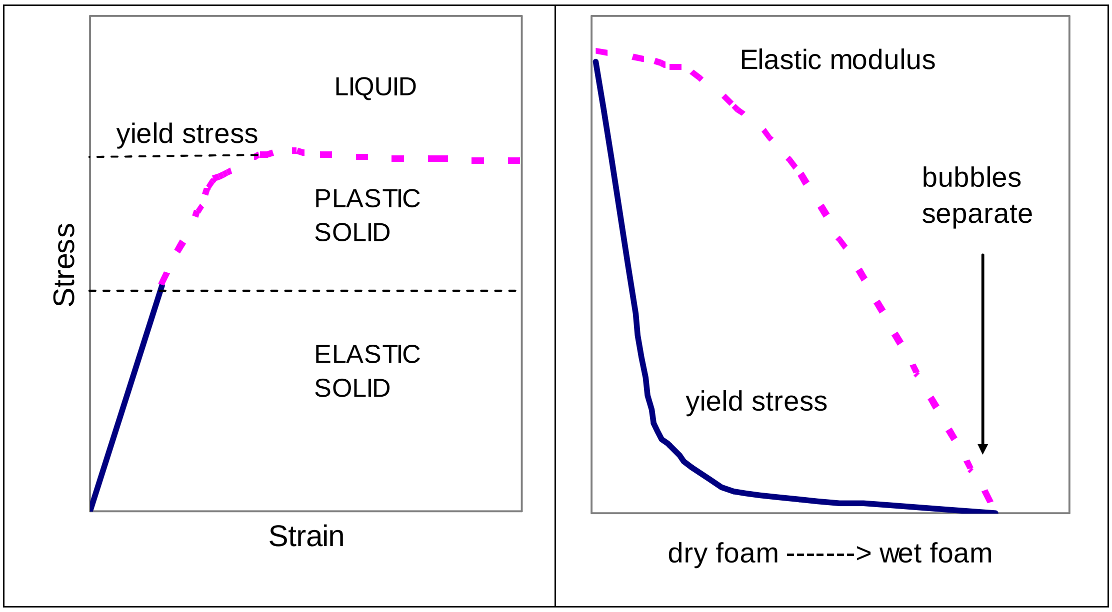


Figure 4.6 - Foam properties: (left) Stress – strain relationship and (right) The elastic modulus and yield stress depend strongly on the liquid fraction of the foam (Weaire and Hutzler, 1999).

4.2.4 Foam rheology

It is not simple to understand the rheology of foam. The flow of a foam differs from that of conventional fluids. Complications include the following three aspects (Heller and Kuntamukkula, 1987). Firstly, the flow characteristics are affected by the size and shape of the channels that confine it due to the presence of bubbles with various sizes. Secondly, compressibility behaviour affects foam flow in tube viscometric measurement. Thirdly, foam drainage during testing can have an impact on the flow characteristics.

In most work related to foam flow experiments, the results of measurements have been analysed in terms of the traditional rheology parameters of fluids. In practice, the term ‘effective or apparent viscosity’ is used to describe foam rheology in order to accommodate the differences between foam and fluid flow. The presence of the compressible bubbles with various sizes affects the flow characteristics, and the measured viscosities are then not true for the absolute values, like in the case of a fluid.

According to the ratio of mean bubble size (r_B) and flow channel size (Rc), foam flow is divided into two types, i.e. macroflow and microflow (Kraynik, 1988). Foam flow through pipes is a typical macroflow, which is characterized by $r_B << Rc$. This type represents bulk foam flow, exhibits a nonlinear correlation (between shear stress and shear rate) and has a slip problem (between foam structure and pipe wall). Investigators generally note the same trends in the variation of apparent viscosity with shear rate, quality and pipe size; however this is not generally true for the absolute values themselves (Heller and Kuntamukkula, 1987). On the other hand, foam flow in a porous medium or in a fine capillary tube is a microflow type, where $Rc \leq r_B$. In this case, the flow can not be related to foam viscosity because the dimension of bubbles is less than or comparable to that of the pore space (Kraynik, 1988).

Studies on foam rheology have been conducted in a number of viscometric devices, including the Brookfield viscometer, and also using a modified viscometric device

(Heller and Kuntamukkula, 1987). Major experimental problems in viscosity measurement are (1) the evidence of collapse or rearrangement of the network of foam bubbles on contact with the rotating solid surface of the viscometer device and (2) drainage of the foam liquid during testing.

Most studies agree that the apparent viscosity increases with the gas content and decreases with shear rate (Assar and Burley, 1986). For example, as shown in Figure 4.7, Marsden and Khan (1966) found that increasing gas content from 70% to 80% and then 90% resulted in increasing the foam viscosity from 130cp to 210 and then 280cp respectively at 100 rpm rotational speed (Heller and Kuntamukkula, 1987). It is clear that foam viscosity is dependent on its density, with viscosity increasing at lower density (increased gas content). Kinematic viscosity may therefore be a suitable property to represent resistance to foam flow. It was also found that the apparent viscosity varied between 50 and 500 cp for aqueous foam having a quality in the range 70-96%. As a comparison, a Brookfield viscosity measurement of bitumen pen 70/100 in the range 140^{o}C to 180^{o}C is typically about 260 to 55 mPa.s (note: 1 mPa.s = 1 cp).

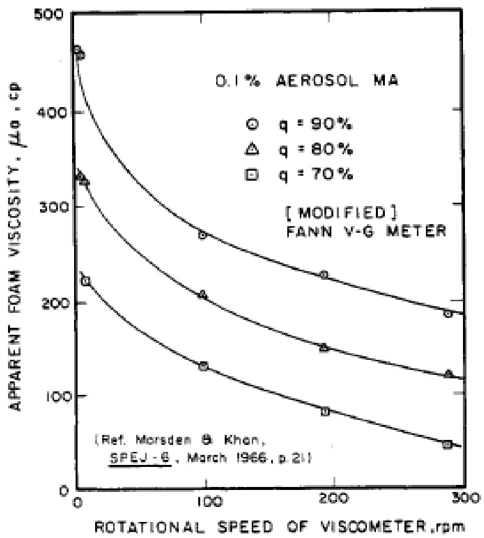


Figure 4.7 - Apparent foam viscosity at various foam qualities (Marsden and Khan, 1966 in Heller and Kuntamukkula, 1987)

4.2.5 Foam lifetime and collapse

Foam lifetime may be determined by flow drainage and various chemical components which affect the microstructure. Generally, foam starts wet and becomes drier as the liquid flows out of it. For such foams, especially those generated by shaking, this process often continues until a stable state is reached. In other cases, the foam has a finite lifetime and when it becomes sufficiently dry, the lamella become unstable and rupture, and hence the foam collapses.

If a lamella is allowed to drain it becomes very thin. In practice, films spontaneously rupture when they become sufficiently thin. The effect of external fluctuations, physical actions, or other surface properties (e.g. viscosity or surface tension) may accelerate the foam collapse process.

Low quality foams formed of spherical bubble distribution will collapse more slowly than high quality polyhedral foams. Due to the curvature of the Plateau borders (their radii are greater than those of lamella), their fluid pressures are much lower than in the lamella, and the liquid therefore drains rapidly from lamella into the Plateau borders. This process is called ‘Plateau border suction’. Skupien and Gaskell (2000) found that foam life increases with decreasing surface tension acting in the lamella, reducing Plateau border suction.

Foam lifetime may be extended by the addition of surface-active agents (surfactants), which generate surface effects by reducing surface tension and therefore balancing the liquid drainage from the lamella (Kraynik, 1983). For bitumen, surfactant is found to be primarily contained in asphaltenes (Barinov, 1990 and Sheu et al., 1994) so that it enhances foamability and life-time (Bauget et al, 2001).

Foam collapse may be also caused by contacting between large and small bubbles. When they contact each other, the higher gas pressure inside the smaller bubble will diffuse through the liquid separating the two bubbles, until the smaller bubble is fully absorbed by the larger. This also causes the larger bubble lamella to become thinner and therefore increases the danger of collapse.

Gravity is also found to be one of the reasons for foam collapse. It forces the liquid towards the bottom and the gas constituent towards the top. This will happen until the spherical walls are not capable of withstanding the surface tension. Stirring spherical foam to redistribute the bubbles can prevent this excessive thinning. Any agitation of high quality polyhedral foam will, however, promote rupture of the thinned bubble walls.

4.2.6 Effect of surface tension

Webber (1974) defined surface tension as a physical property caused by the cohesion between molecules at the surface of a liquid. This enables the surface to behave as an elastic sheet. Surface tension will always tend to minimize the surface energy by reducing the surface area. This is the reason why a free droplet of liquid naturally tends to a spherical shape due to a sphere having the minimum surface area for a given volume. In a gas bubble, surface tension of the liquid in a bubble wall therefore tends to collapse the bubble. Gas pressure inside the bubble balances this tension. It is noted that the gas pressure within a bubble is inversely proportional to the bubble size. This means a larger bubble, with reduced pressure inside, will tend to collapse due to the pressure not being able to withstand the surface tension. The surface tension, σ_s, can be expressed per unit length in mN/m or per unit area in mJ/m^2 (note: 1 mN/m = 1 inch.erg/cm^2 =1 mJ/m^2). For water, its magnitude is around 73mN/m in contact with air at room temperature.

With regard to foam stability, Breward (1999) showed that a surface tension gradient at the lamella and Plateau border results in a surface stress acting on the liquid at the surface. If this surface stress opposes the Plateau border suction (surface tension in the lamella being lower than in the Plateau border) the foam stability will tend to increase; conversely, the foam will be destabilised if the surface stress enhances the Plateau border suction.

4.2.7 Foam drainage

Foam drainage is important since it controls the life time of the foam. As described previously, foam consists of polyhedral bubbles separated by liquid films or lamellae. Films drain into the surrounding Plateau borders due to capillary suction and the liquid drains out of the foam (foam drainage) due to gravity induced flow through an interconnected network of Plateau border channels. Film breakage and liquid drainage can occur simultaneously and interact in a complicated way. Figure 4.8 shows an illustration of foam and film drainage.

Kraynik (1983) has investigated foam drainage using aqueous foams ranging from shaving cream to detergent soaps. He found that the time required for one half of the liquid to drain from the foam (termed as 'drain half life'), is inversely proportional to the square of the bubble diameter. This finding indicates that improved foam stability is related to small bubbles. This illustrates that the foam structure is very important in the drainage process. Film drainage affects foam drainage by controlling the rate of liquid delivery into the Plateau borders.

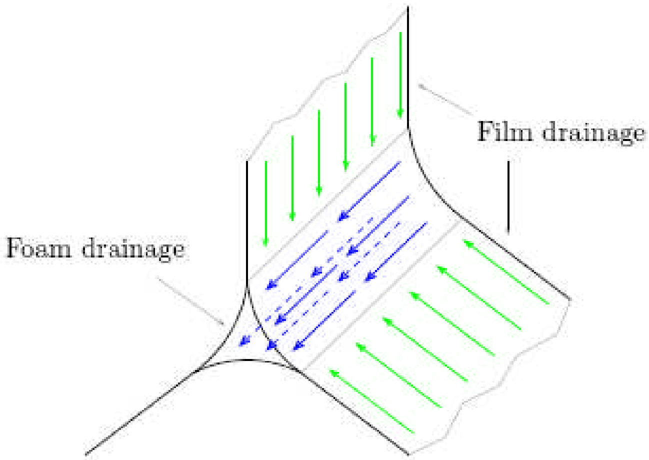


Figure 4.8 - Illustration of foam drainage and film drainage

4.3 Generating process and category of foamed bitumen

4.3.1 Foamed bitumen generating process

Since the foam generation process influences the created foam type and its properties (Weaire and Hutzler, 1999), it is therefore important to understand the dynamic process in generating foamed bitumen. The operation of a foaming machine to generate foamed bitumen has been described in Chapter 3 section 3.3.1. Subsequently, the different steps comprising the foaming process were described. This was developed with some consideration of the foaming process discussed by Jenkins (2000), He and Wong (2006) and Barinov (1990). These steps can be described as follows:

- **Step 1: Initial position**

When water and air are injected together under high pressure, the water takes the form of large number of small water droplets which directly enter the hot bitumen liquid phase. This situation aids rapid energy exchange. It is noted that the heating process of bitumen causes its nuclei to distribute uniformly in the bulk phase, and heating to temperatures above 100^{o}C is accompanied by its conversion into a true liquid, creating the conditions for diffusion of (asphaltene) surfactant molecules from its bulk to any interfaces which develop (e.g. when foamed).

- **Step 2: Heat transfer**

Heat transfer from hot bitumen to the cold small water droplet surfaces occurs rapidly when they come into contact in the expansion chamber. Subsequently, the temperature of the water droplet increases whilst temperature of the bitumen decreases.

- **Step 3: Evaporating and steaming**

As soon as the water droplet reaches a temperature of 100^{o}C, the energy transferred from the hot bitumen exceeds the latent heat of steam, resulting in evaporation of the water droplet surface. In this way, the water evaporating from the droplet generates steam and this results in explosive expansion bubbles due to the presence of internal pressure. It is noted that the amount of water becoming steam is dependent upon the amount of water added and the bitumen temperature. As the water is not sprayed in a single instant, there is probably also condensation occurring during steaming. This indicates that some water potentially still remains in the foamed bitumen (see section 4.4.3).

- **Step 4: Foam forming**

Water steam is forced into the bitumen liquid phase, causing steam bubbles to be trapped within the bitumen liquid. At this time, wet foam is formed in which small bubbles appear dispersed within the bitumen liquid phase. In this condition, the (asphaltene) surfactant forms an adsorption layer on the bitumen-vapor phase boundary. Due to the bubbles' explosive expansion behaviour, they pressurize the bitumen liquid. In seconds, the bubbles become larger and the bitumen films become thinner, and this results in a dry foam until a state of equilibrium is reached. The foam state may remain as long as the thickness and surface properties of bitumen film can counteract the bubble pressure. It was noted that during this investigation, in which foam was sprayed into a container for 5 seconds, foams grew rapidly. Some bubbles seemed to collapse before the 5 seconds of spraying had finished. However, not all foam bubbles grew and collapsed at the same rate. Sporadically, small bubbles grew up following collapse of large bubbles. When foams reached their maximum volume, they remained stable for seconds before all bubbles ruptured together, which was accompanied by loss of steam. At this time, foam volume therefore dropped dramatically. It is supposed that when foam expanded in the container, the bitumen liquid (including collapsed bubbles) tended to drain to the bottom, whereas the bubbles tended to rise up. This situation results in a foam column transition from a wet condition in the bottom to a dry condition in the top as shown in Figure 4.1.

- **Step 5: Foam collapsing**

The collapse of foamed bitumen may be explained by the following three scenarios:

Scenario 1 (Foam collapse caused by external physical contact):

Physical contacts between bitumen films (lamella) and an outside material having lower temperature such as air, aggregate surface or container cause the foam to collapse. Breward (1999) and He and Wong (2006) agreed with this scenario. If this scenario is correct, it means when foamed bitumen is directly sprayed into cold wet aggregate, it then immediately collapses and returns to the original fluid state. Interestingly, when foamed bitumen is sprayed into a cold steel container, it does not collapse; it even grows to reach a maximum volume. It is likely that the heat energy loss is more significant than physical contact with external materials in affecting foamed bitumen ruptures.

Scenario 2 (Foam collapse caused by excessive pressure of expanded bubble):

Foam may collapse if the surface tension (which is a function of viscosity) of the bitumen film is not adequate to withstand the expanded bubble pressure. In this study, when applying foaming water at 5% or 6% (using a bitumen mass of 500 g), some foams were found to burst as they reached maximum volume. This might indicate that the expanded bubble pressure exceeded the surface tension of the bitumen film. Jenkins (2000) agreed with this scenario since, when the elongation of the bitumen for the given (short) loading time is exceeded, the bubble will burst. He and Wong (2006) also supported this scenario since they found that the maximum expansion ratio (ERm) of foam produced using bitumen pen 100 was lower than with bitumen pen 60. This indicates that foam with lower viscosity and relatively low surface tension (pen 100) will be more likely to collapse prematurely before reaching its maximum volume than a higher viscosity foam (pen 60). It is interesting to note that the effect of surface tension on the foam life time may have two different aspects. First, considering the pressure in an expanded bubble, high surface tension of the bitumen film gives a positive effect on the foam life; on the other hand, secondly, considering Plateau border suction, high surface tension of the bitumen film may increase this suction and give a negative effect on the foam life.

Scenario 3 (Foam collapse caused by draining process)

In general foam literature, it is found that foam collapse is caused by the foam draining process (Weaire & Hutzler, 1999; Breward, 1999; Breward & Howell, 2001; Hilgenfeldt et al, 2001). The draining process can be caused by gravity, suction between Plateau border and lamella, and capillarity between small and large bubbles. As the liquid fraction drains out of the foam body, the thin film becomes unstable and then ruptures so that eventually the foam collapses.

4.3.2 Foamed bitumen as a member of the foam family

Considering the process of generating foamed bitumen in the expansion chamber as described in section 4.3.1, foamed bitumen may be composed of air, water vapour (steam), hot bitumen liquid and remaining water. At the beginning, the air and water vapour are trapped as small bubbles in the continuous bitumen liquid phase. The remaining water may be present in the bitumen liquid or else inside the bubbles. This formation gives a ‘wet foam’ condition. The water vapour content then increases rapidly due to the presence of internal bubble pressure, causing the bitumen liquid to form a thin film at the bubbles surfaces. This gives a ‘dry foam’ condition.

The above description indicates that the constituent of air and water vapour forming bubbles with internal pressure represents the gas in general foam literature, and that the bitumen acts as the liquid phase. The volume fraction of remaining water is small compared to that of bitumen liquid and therefore its presence will not alter the general formation of foamed bitumen. Therefore, it can be deduced that foamed bitumen can be included as a member of the foam family. This statement is utilised in this study to develop foamed bitumen characteristics, borrowing knowledge from general foam literature. In asphalt material literature, it is noted that Jenkins (2000) placed foamed bitumen into the polyhedral foam category in terms of bubble-form.

4.4 Heat transfer and foamed bitumen temperature

Since foamed bitumen is a result of a heat transfer process from hot bitumen to the cold water, it is necessary to understand this process including temperature change during foaming. Foam temperature may control foam stability and affect foam properties.

4.4.1 Steam and its heat energy

Figure 4.9 shows the paths of energy needed by 25 g of water (representing FWC of 5%) to change its heat energy from a temperature of 20^{o}C, becoming steam when it is injected into 500g of hot bitumen at a temperature of 180^{o}C. In heat energy theory, the energy used by the water to increase its temperature from a low temperature to 100^{o}C (boiling point) is called the enthalpy of the water (specific heat of water), whereas the energy used by water in evaporation is called the enthalpy of evaporation or latent heat (Spirax-Sarco, 2005). The enthalpy of water and evaporation are approximately 4.1858 J/g^{o}C and 2,258 J/g respectively, whereas the specific heat of bitumen is 2.0929 J/g^{o}C. Path A-B in Figure 4.9 is the energy needed by the water to reach a temperature of 100^{o}C (Qw1= 8,372 Joules), whereas Path B-C is the evaporation energy (Qw2= 56,446 Joules). Thus, the total energy (Qw1+Qw2) is around 64,818 Joules (Path A-C) which is still less than the energy loss of the hot bitumen to reduce its temperature to 100^{o}C (Q_{b100}= 83,716 Joules). This means the steam/water will reach the dry saturated steam line and potentially go into the superheated region (point D), in which all water has become superheated steam. However, when the amount of cold water is 40 g (FWC 8%), the total energy (Qw1+Qw2) is around 103,707 Joules which is higher than Q_{b100}. For this case, the steam/water will be unable to reach the dry saturated steam line, indicating that not all water will become steam.

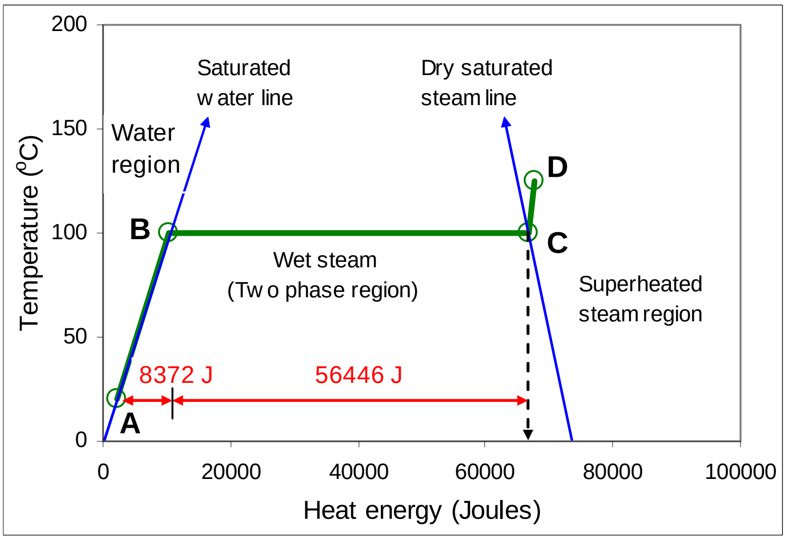


Figure 4.9 - The paths of energy needed by 25 g water at 20°C to change to the steam phase.

4.4.2 Foamed bitumen temperature

The final formation of steam, either as a superheated steam or as a wet steam, affects the foam temperature. Subsequent foam temperature was studied in the following scenarios, namely (1) Foam sprayed in the air, (2) Foam sprayed into a steel cylinder (when measuring ERm and HL), and (3) Foam sprayed onto cold moist aggregate.

Table 4.1 - Properties of components and data to calculate foam temperature

Property	Components				
	Water	Steam	Bitumen	Aggregate (Limestone)	Steel (cylinder)
Specific heat, J/g.°C	4.1858	2.100	2.0929	0.8	0.4688
Latent heat, J/g	-	2257.8	-	-	-
Temperature, °C	20	100	180	20	20
Thermal conductivity, W/m. °C	0.58	0.016	0.17	1.3	16
Other data used are as follows: Mb (bitumen mass) = 500g (for scenario 1 and 2) or = 3.85% (by mass of mixture, for scenario 3), Steel cylinder: M = 1200g, diameter = 280mm, height = 370mm, thickness = 0.35mm Aggregate water content = 4.43% (by mass of mixture, for scenario 3)					

The properties of components and data used to find the foam temperature are shown in Table 4.1, whereas the calculations apply Eq. 4.1 to Eq. 4.8. The results are represented in Figure 4.10 while the calculations can be seen in Appendix B: Table B.4.1 to Table B.4.5.

$$Qw1 = Mw*Sw*(100-Tw) \quad \text{…………………………..Eq. 4.1}$$

$$Qw2 = Ms*Ls \quad \text{…………… ……………………….Eq. 4.2}$$

$$Qw3 = Ms*Ss*(T_f-100) \quad \text{………………………..…Eq. 4.3}$$

$$Q_{b100} = Mb*Sb*\Delta T \quad \text{……………….………………Eq. 4.4}$$

$$Qb = Mb*Sb*(180-T_f) \quad \text{…. ……………………….....Eq. 4.5}$$

where:

Q = heat energy in Joules,

M = mass in grams,

S = specific heat (or enthalpy) in J/g.^{o}C,

T = temperature in ^{o}C,

L = latent heat (or enthalpy of evaporation) in J/g,

ΔT = temperature difference,

Index w is for water, b is for bitumen, s is for steam and f is for foam,

Q_{b100} = the amount of transfer heat energy required by 500g of bitumen to reduce its temperature from 180^{o}C to 100^{o}C. (for bitumen temperature of 180^{o}C, Q_{b100}= 83716 Joules).

For Qw1 + Qw2 < Q_{b100} then foam temperature (T_f) was calculated using equation:

$$Qw1 + Qw2 + Qw3 = Qb \quad \text{…………………………….... Eq. 4.6}$$

For Qw1+ Qw2 > Q_{b100} then T_f was calculated using equation:

$$Qw1 + x\ Qw2 = Q_{b100} \quad \text{……….…..…………………..…Eq. 4.7}$$

For which: equilibrium temperature T_f = 100^{o}C and steam produced = x*Mw

For Qw1 > Q_{b100} then T was calculated using equation:

$$Qw1 = Qb \quad \text{…………………………………………………..…………Eq. 4.8}$$

Scenario 1

Scenario 1 was developed to understand the change of foam temperature with FWC when foam created in the expansion chamber was sprayed out into the ambient air without any disturbance of other materials. This calculation used 500g of hot bitumen (180^{o}C) and cold water (20^{o}C) that varied between 5g (FWC=1%) and 50g (FWC=10%). The case of hot bitumen only (FWC=0%) was also shown for comparison purposes. It is assumed that the heat transfer process occurs immediately since both bitumen and water are sprayed in the form of very small droplets.

The foam temperature in scenario 1 was calculated using Eq. 4.6 for FWC up to 6% and using Eq. 4.7 for FWC of 7% and higher. It was found that the foam temperature decreased with increasing FWC up to 6% and then it remains constant at 100^{o}C at FWC $\geq$ 7%. As shown in Figure 4.10, the calculated foam temperature was found to vary between 167^{o}C (at FWC=1%) and 100^{o}C (at FWC $\geq$ 7%).

Scenario 2

Scenario 2 was developed to understand the change of foam temperature with FWC when its ERm and HL are measured in the steel cylinder. This scenario is similar to scenario 1 except that foam is sprayed into the steel cylinder. Eq. 4.6 and 4.7 were also applied in the same way as in scenario 1, in which heat energy of water and steel cylinder (vessel) balance to heat energy of hot bitumen (see Appendix Table B.4.2). The result shown in Figure 4.10 is the temperature calculated after 60 seconds (immediately after foam collapse). The final equilibrium temperature will occur after at least 10 minutes (for FWC of 10%). It was found that the foam temperature at 60 seconds in scenario 2 is relatively similar to that in scenario 1, varying from 166^{o}C at FWC 1% to 97^{o}C at FWC 10%. The temperature is a little lower than in scenario 1 due to the gradual heat energy transfer to the steel cylinder. Figure A.4.1 (Appendix A) shows the final temperature of foamed bitumen for various bitumen temperatures (140^{o}C to 180^{o}C), as well as that at 60 seconds.

Scenario 3

Scenario 3 was developed to simulate the real situation in which foam is sprayed directly onto the cold moist aggregate. Since Qw (105901 Joules) is greater than Q_{b100} (83716 Joules), Eq. 4.8 is therefore valid for scenario 3. Since much evidence exists that following mixing, temperature of foamed asphalt does not drop immediately but reduces gradually, three temperature levels are therefore introduced namely (3a) an initial temperature, (3b) an intermediate temperature and (3c) a final temperature. The initial temperature is the temperature at which the foam starts contacting with water on the fine aggregate surface, the intermediate temperature is the temperature at which the foam coats the fine aggregate surface, whereas the final temperature is the equilibrium temperature of the mixture. The results can be seen in Figure 4.10, while the calculation can be found in Appendix B: Table B.4.3 to Table B.4.5.

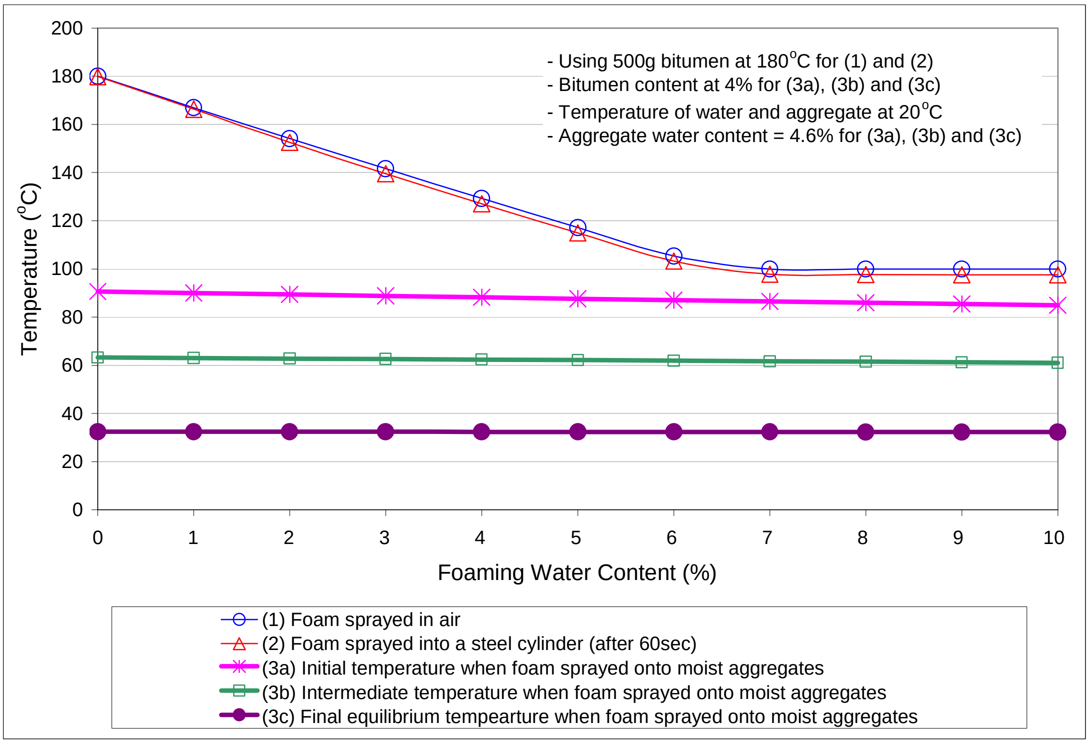


Figure 4.10 - Predicting foam temperature

Based on the result of a binder distribution investigation (see section 6.2.2 Chapter 6), the coated particles only comprise around 15% by mass or about 55% by particle surface area for a mixture using bitumen Pen 50/70 at a FWC of 4% and a bitumen temperature of 180°C. Therefore, the initial temperature was estimated based on the mass of water distributed on the coated particle surface (around 55% of the total), while the intermediate temperature was calculated based on a mass of coated aggregate of around 15%. It was found that the initial temperature was around 90^{o}C to 85 ^{o}C for various applied FWCs, the intermediate temperature was around 63^{o}C to 61 ^{o}C, and the final temperature was found to be 32^{o}C for all applied FWCs (see curve 3a, 3b and 3c in Figure 4.10).

The final equilibrium temperature of foam for scenario 3 is much lower than in scenario 1 due to the mass of cold aggregate being much greater than the bitumen (bitumen mass is 4% of aggregate mass). It is probable that the rate of foam temperature reduction, from 90^{o}C to 32^{o}C, is different at different FWC values. Unfortunately, it is impossible to calculate heat flow rate (Q/t) using Eq.4.9 either based upon foam/bitumen or the aggregate phase. In the mixing process, the contact surface area (A) between foam/bitumen and aggregate can not be defined exactly, nor the thickness (L) at each FWC. If the mass and area of coated aggregate are assumed to be 15% and 55%, the values at the end of the binder distribution phase, the thickness of bitumen will vary between 0.02 and 0.18mm depending upon the fraction sizes. The thickness and surface area of aggregate for each size can also be estimated based on the result of the binder distribution investigation. It is found that the time needed by the foam to cool down to the initial temperature is less than one second, and to the intermediate temperature it is around 14 seconds, whereas the final temperature is reached after around 2 minutes. It is supposed that the actual time to transfer energy is longer than those values since the contact area at initial/intermediate mixing is lower than at the end of mixing. This means the foam temperature during mixing is probably still higher than 50^{o}C (approximately the softening point of bitumen Pen 50/70).

$$\frac{Q}{t} = \frac{k * A * \Delta T}{L} \quad \text{..........................Eq. 4.9}$$

where:
Q = heat energy (J)
t = time (s)
k = thermal conductivity (W/m.^{o}C)
A = surface area (m^2, perpendicular to thickness)
L = thickness (m)
ΔT = temperature difference (^{o}C)

4.4.3 Steam loss during foaming

In the experiment, it was observed that some steam escaped during the foaming process and some water still remained in the foam. The evidence of escaping steam may cause a significant reduction of foam volume and foam quality; the remaining water is probably caused by the presence of water droplets in the wet steam when it fails to become dry steam (e.g. when 40g water is added). If water droplets remain during the foaming process at lower FWC (e.g 5% or using 25g of water), it may be that some of the steam also condenses during the evaporation process.

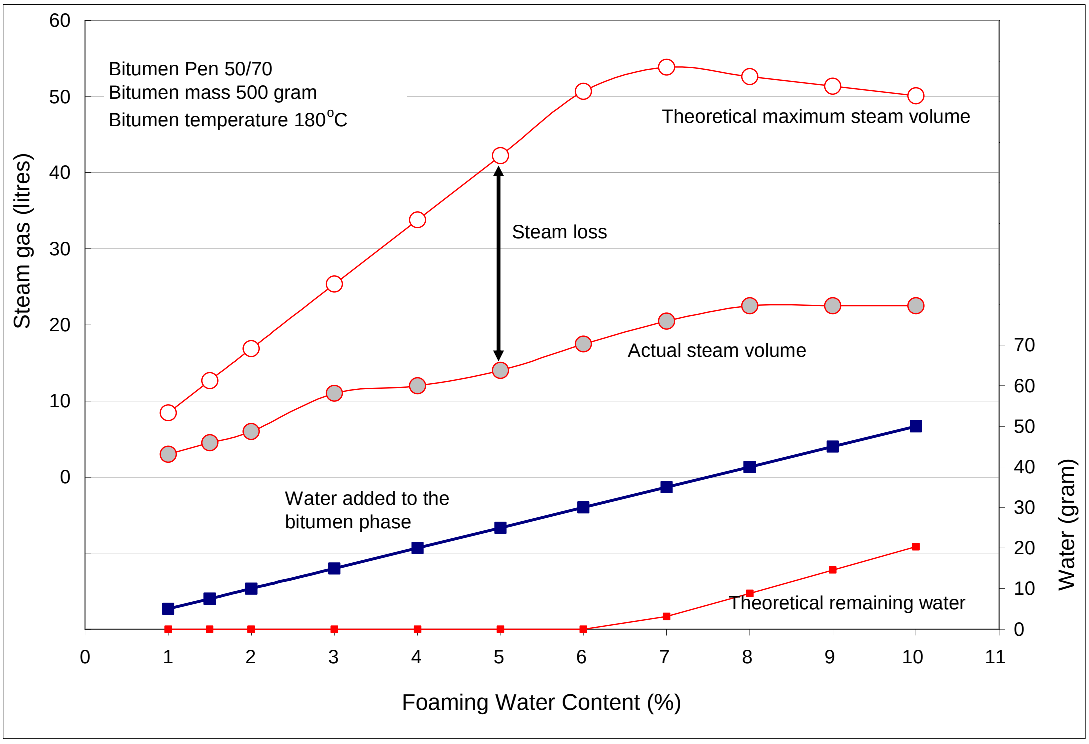


Figure 4.11 - The actual and theoretical maximum steam volume

Figure 4.11 shows the actual and theoretical maximum steam volume when 500g of hot bitumen at a temperature of 180°C is injected by cold water at various FWC values. A calculation example can be seen in Table 4.2. Theoretically, at a FWC of 1% to 6% (water added from 5g to 30g), all the added water is utilised, becoming steam, and hence there is no remaining water in the foam. For this case, as the water added increases, the maximum steam volume (theoretically) also increases significantly. However, at FWC higher than 6% (up to 10%), the energy of the hot bitumen is inadequate to change all the water into steam and hence some water remains in the foam. For this case, the maximum steam volume slightly decreases with FWC. The actual steam volume as shown in Figure 4.11 was calculated based upon the values of maximum expansion ratio (ERm). It can be seen that the actual steam volume is lower than the theoretical maximum steam volume, indicating steam loss (steam escaping or condensing) during foaming process (especially during the foam spraying phase). The steam loss becomes greater as FWC increases up to 6%.

Table 4.2 - Calculation example

Foaming water content (FWC)	5%	8%
Water added (g)	25	40
Qw1(J)= Mw*Sw*(100-Tw)	8372	13395
Qw2 (J)= Ms*Ls	56446	90313
Qw1 + Qw2 (J)	64818	103708
Q_{b100} (J)	83716 >(Qw1+Qw2)	83716 <(Qw1+Qw2)
T_f (^oC), Qw1+Qw2+Ms*Ss*(T_f-100)=Mb*Sb*(180-T_f) T_f= 100	117	100
Max steam (gram)= Mw Max steam (gram)= x*Mw, Qw1 + x Qw2 = Q_{b100},	25	31.15
Max steam volume (litre), V=n*R*T/Pr	42.245	52.630
Theoretical water remained (gram)	0	8.85
ERm for bit Pen 50/70 (measured)	27	35
Actual steam gas volume (litre)= ERm*0.5	14	22.5
Steam loss (litre)= max steam- actual steam	28.25	30.13

The theoretical maximum volume of steam is calculated using Eq. 4.10 as follows:

$$Pr*V = n * R * T$$

$$V = n * R * T / Pr \quad \text{Eq. 4.10}$$

where:

V = volume (litres)

Pr = pressure in atmospheres (atm)

n = number of moles = mass/ atomic mass of compound

R = Universal constant = 82.0545 (atm. Litre/mole. Kelvin)

T = temperature (Kelvin)

At a FWC of 5%, mass of water = 5%*500g = 25g (500g is mass of bitumen used)

atomic mass of water = 2*1.06 + 16 = 18.12

n = 25/18.12 (mole)

T = 100 ^{o}C = 373 ^{o}K

At pressure 1 atm, V = n * R * T / Pr

= (25/18.12) * 82.0545 * 373 / 1

= 42.245 litres.

4.5 Maximum Expansion Ratio (ERm) and Half-Life (HL)

The ERm and HL are broadly used to characterise foam properties since both parameters represent 'foamability' and being easily measured. In this study, foamed bitumen was produced using a mobile laboratory plant, the Wirtgen WLB 10. A description of this machine can be found in Chapter 3 sections 3.3.1 to 3.3.3, whereas the ERm and HL measurement was described in Chapter 2 section 2.4.1.1. The ERm and HL were investigated using three different binder types, and at various FWCs and bitumen temperatures. These three aspects are considered in order to vary the properties of the foamed bitumen. In this study, a code e.g. FB 70/100: FWC5%-180^{o}C was used to designate each foamed bitumen, in this case with bitumen Pen 70/100 produced at FWC of 5% and at bitumen temperature of 180^{o}C.

Figure 4.12 to Figure 4.14 show investigation results of foamed bitumen characteristics in terms of ERm and HL, for which the foams were produced using bitumen Pen.70/100, Pen.160/220 and Pen.50/70 at various FWCs (up to 5%). The experiments were conducted twice for each sample using 500 grams of bitumen. It should be noted that the measurement was conducted visually. Due to the foam volumes changing rapidly, the results were not very precise. The sensitivity of ERm is probably about 3 and that of HL about 5-10 seconds.

Figure 4.15 shows foamed bitumen characteristics, for which the foam was produced using bitumen Pen.70/100 and Pen.50/70 over the full range of FWC (1% - 10%). It should be noted that for FWC greater than 5%, the experiment was conducted using a reduced mass of bitumen (i.e. 250 gram) so that the expanded foam volume did not to exceed the capacity of the measuring cylinder. This means, for the same height of foam in the measuring cylinder, the ERm recorded when using 250g will be twice that when using 500g. It can then be understood that the two methods (using 250g and 500g of bitumen) may give slightly different results due to gravity and depth effects. At a FWC of 5%, for bitumen Pen.50/70, the ERm value when recorded using 500g of bitumen was lower than when using 250g of bitumen (different by about 3), but the HL was approximately the same; conversely, for bitumen Pen.70/100, the ERm exhibited similar values, but the HL of the 250g sample was 2 seconds longer than with 500g. Additionally, the ERm and HL of foam produced using bitumen Pen.70/100 in Figure 4.15 were recorded using a digital video camera in order to investigate its decay characteristics (see section 4.6).

It should be noted that inconsistent results in the measurement of ERm and HL are sometimes found (Repeatability estimates were not made). On several occasions, when the foam characteristics were checked for a particular bitumen grade, the results exhibited differences. This fact can also be seen, for example, in the results for FWC of 1% - 5% for both bitumen grades in Figure 4.15, which are not the same as the corresponding results in Figure 4.12 (FB 70/100 at 180^{o}C) and Figure 4.14 (FB 50/70 at 180^{o}C). Some possible reasons have been identified for this as follows:

- Measurement error is possible due to the need for human judgement of rapid foam volume change,
- Flow meter for FWC may be slightly inaccurate,
- Small fluctuation of temperature may occur in the bitumen tank,
- Bitumen properties for one binder type may have slightly varying values of Penetration, Softening point and other chemical properties.

In general, two important points can be drawn from the results as follows:

- FWC, bitumen temperature and binder type all influence the ERm and HL values.
- Foams with high ERm generally have low HL. At FWC of 1%, the ERm is lowest and the HL is longest for a given binder type and temperature. However, at high ERm (FWC $\geq$ 4%), the HL does not tend to decrease.

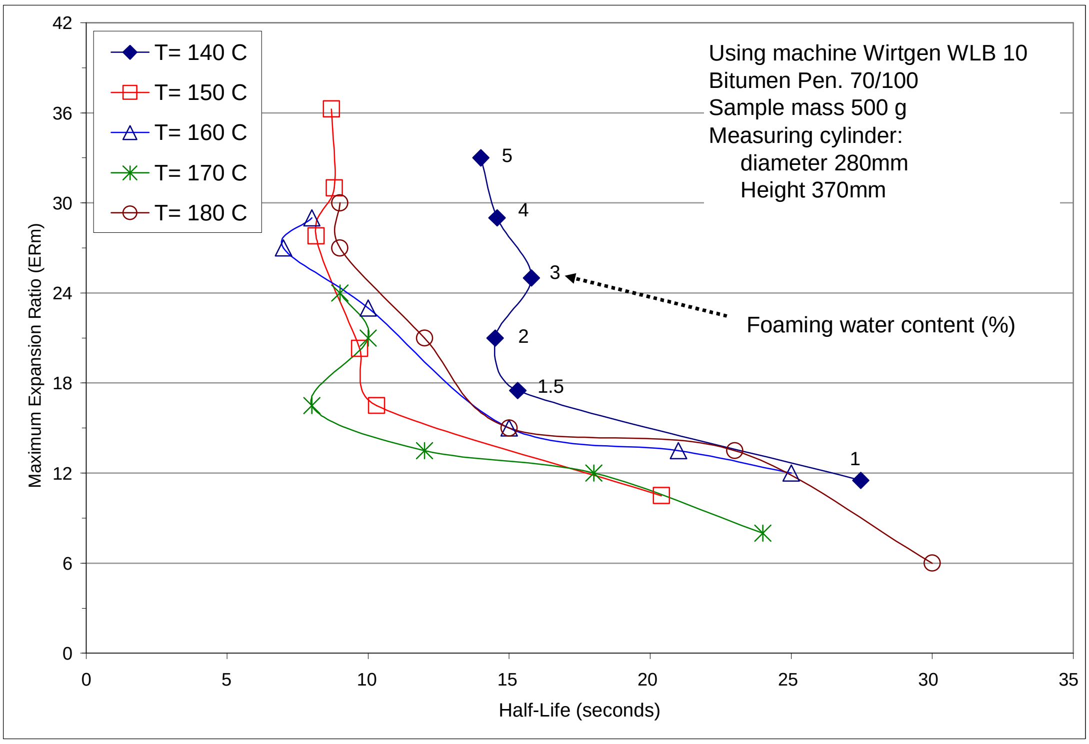


Figure 4.12 - Characteristics of foamed bitumen generated using bitumen Pen. 70/100

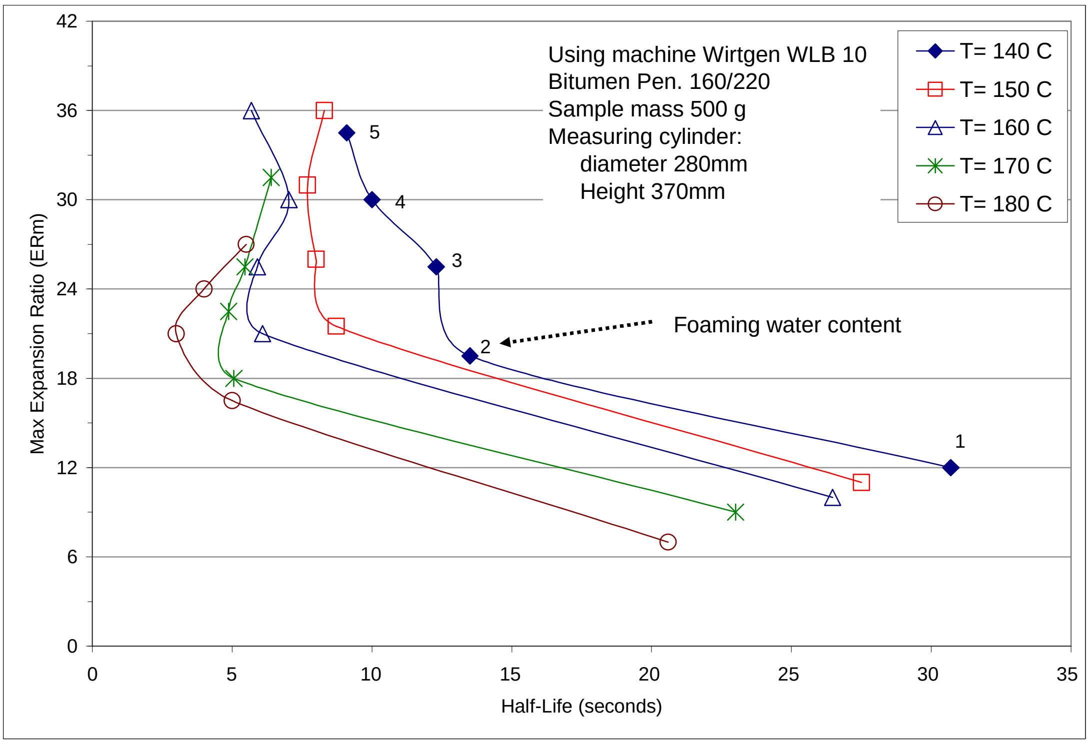


Figure 4.13 - Characteristics of foamed bitumen generated using bitumen Pen. 160/220

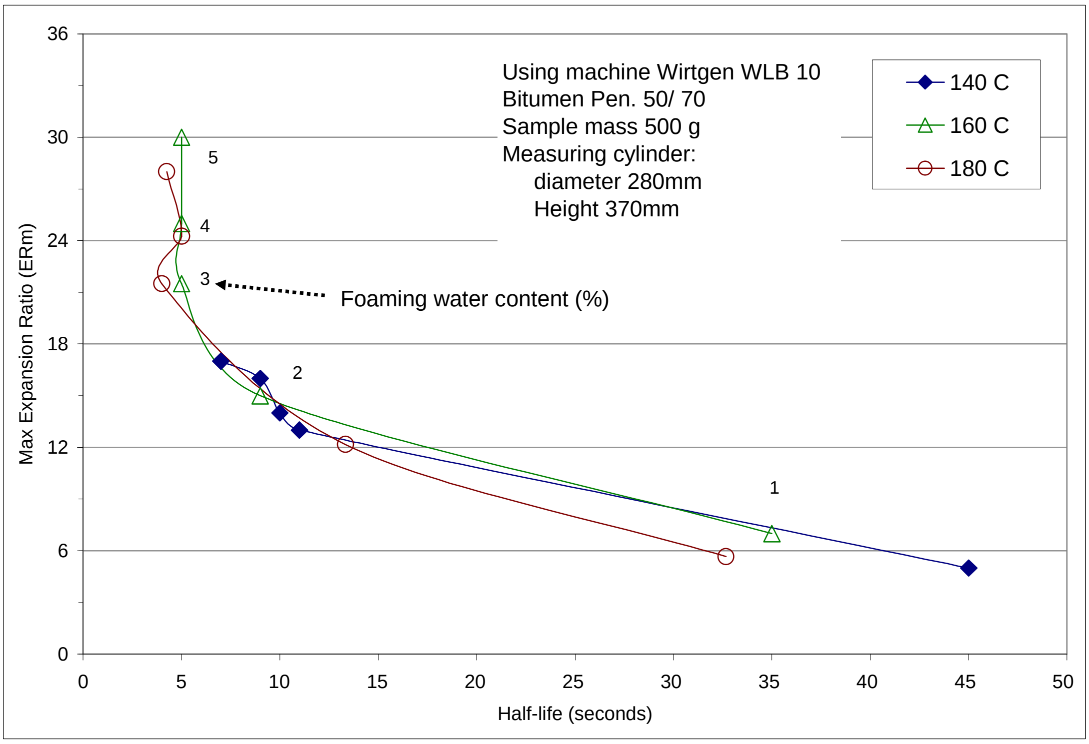


Figure 4.14 - Characteristics of foamed bitumen generated using bitumen Pen. 50/70

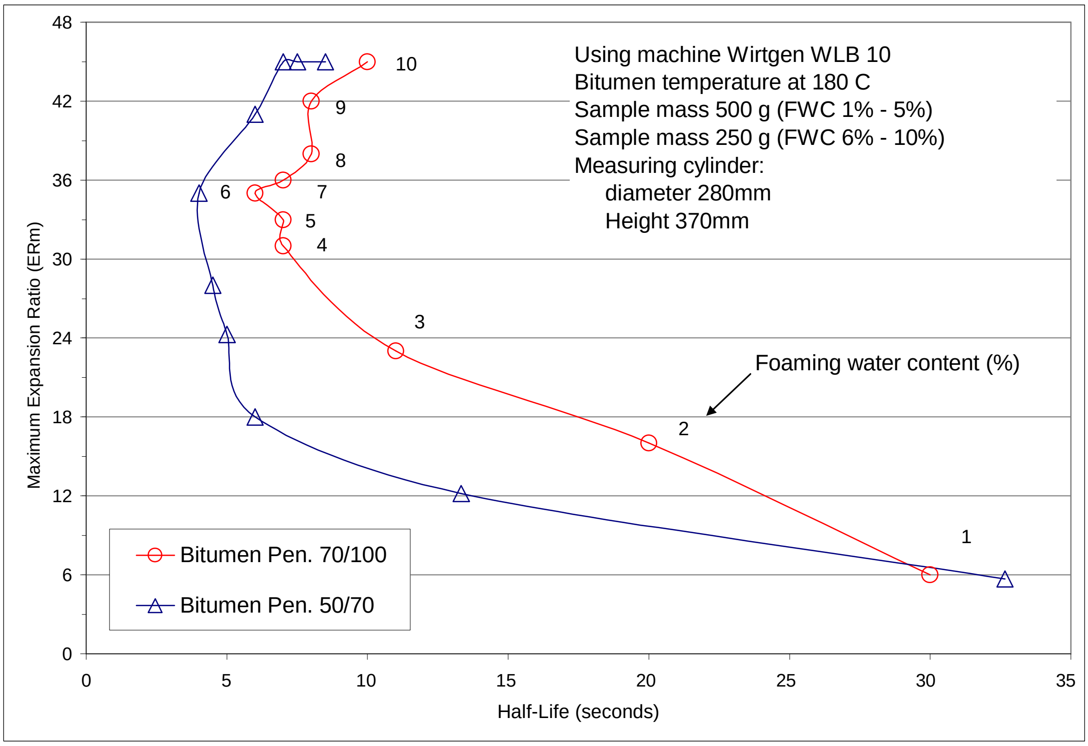


Figure 4.15 - Characteristics of foamed bitumen generated using bitumen Pen. 50/70 and Pen. 70/100 over full range of FWC (1%- 10%)

4.5.1 Effect of foaming water content (FWC)

Figure 4.16 to Figure 4.21 were modified from Figure 4.12 to Figure 4.14 in order to show more clearly the effect of FWC on the ERm and the HL for all binder types. As shown in these figures, in general, the FWC influences the ERm and HL values significantly.

The ERm values increase with increasing amount of foaming water. It can be understood that more FWC application means more water added into hot bitumen. This will generate more steam and hence create more and larger foam bubbles. The foam volume becomes higher or the gas content increases and therefore the measured ERm increases. The HL values tend to behave in an opposite manner to ERm. This can be understood in terms of the foam structure; when gas content increases, the bubble film (lamella) becomes thinner and the resulting foam becomes more unstable. This means that a foam with higher ERm will tend to have a shorter half life. In other words, wet foams will tend to be more stable than dry foams.

Interestingly, increasing the FWC from 1 to 2% resulted in a doubling of the ERm value but at higher FWC's the increase was more moderate. Similarly, increasing the FWC from 1 to 2% causes the HL to decrease sharply and beyond a FWC of 2%, the HL stabilises at an almost constant value.

The constant values of HL at high FWC are thought to relate to the effect of foam temperature. More added water in the foaming process requires more heat energy from hot bitumen and hence the resultant foam temperature will decrease. This low temperature will cause foam bubbles to collapse more slowly. This effect may balance the gas content effect and hence this results in constant HL values at high FWC.

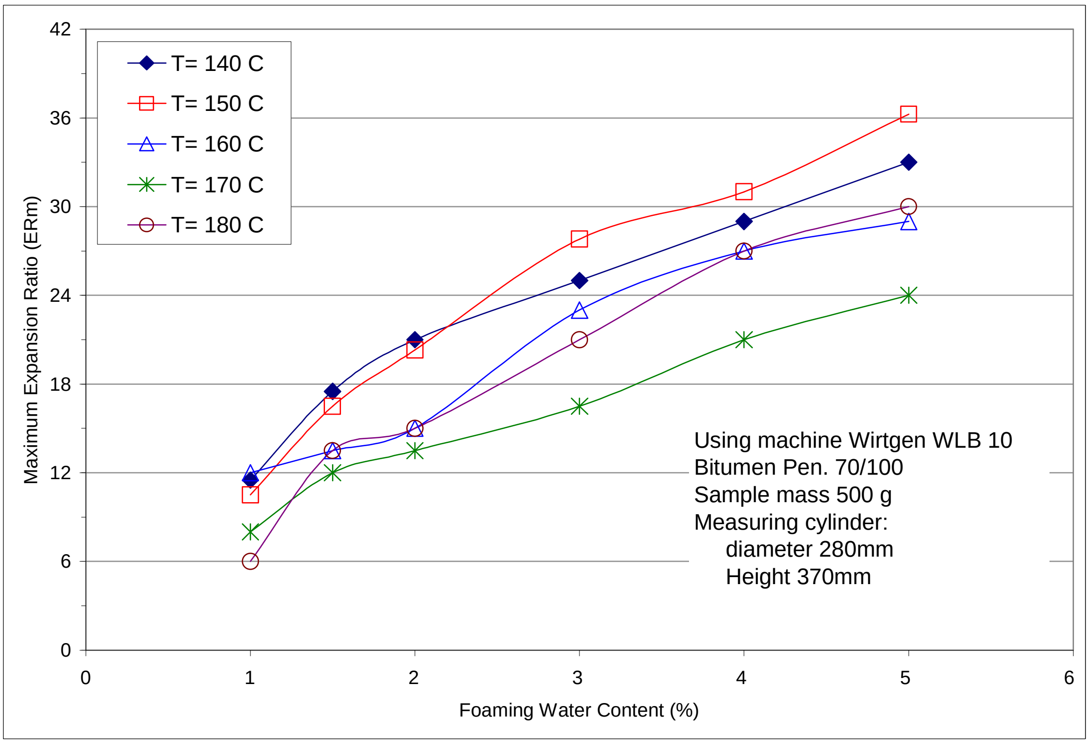


Figure 4.16 - Effect of FWC on the maximum expansion ratio of foamed bitumen produced using bitumen Pen 70/100.

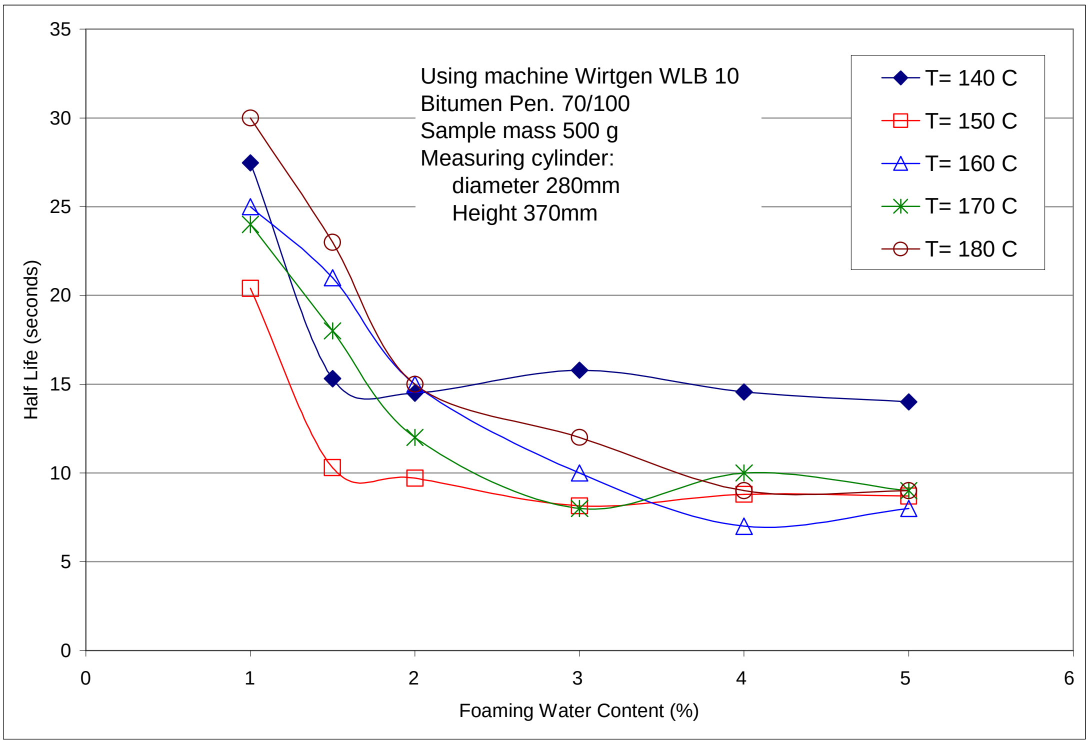


Figure 4.17 - Effect of FWC on the half life of foamed bitumen produced using bitumen Pen 70/100.

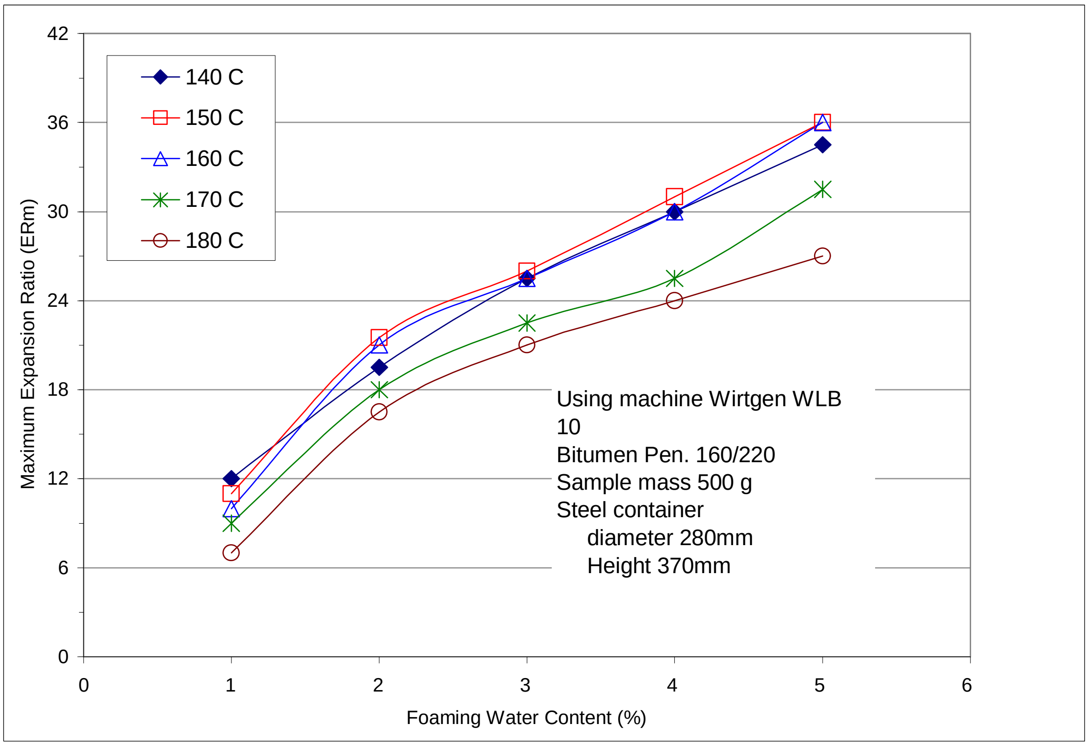


Figure 4.18 - Effect of FWC on the maximum expansion ratio of foamed bitumen produced using bitumen Pen 160/220.

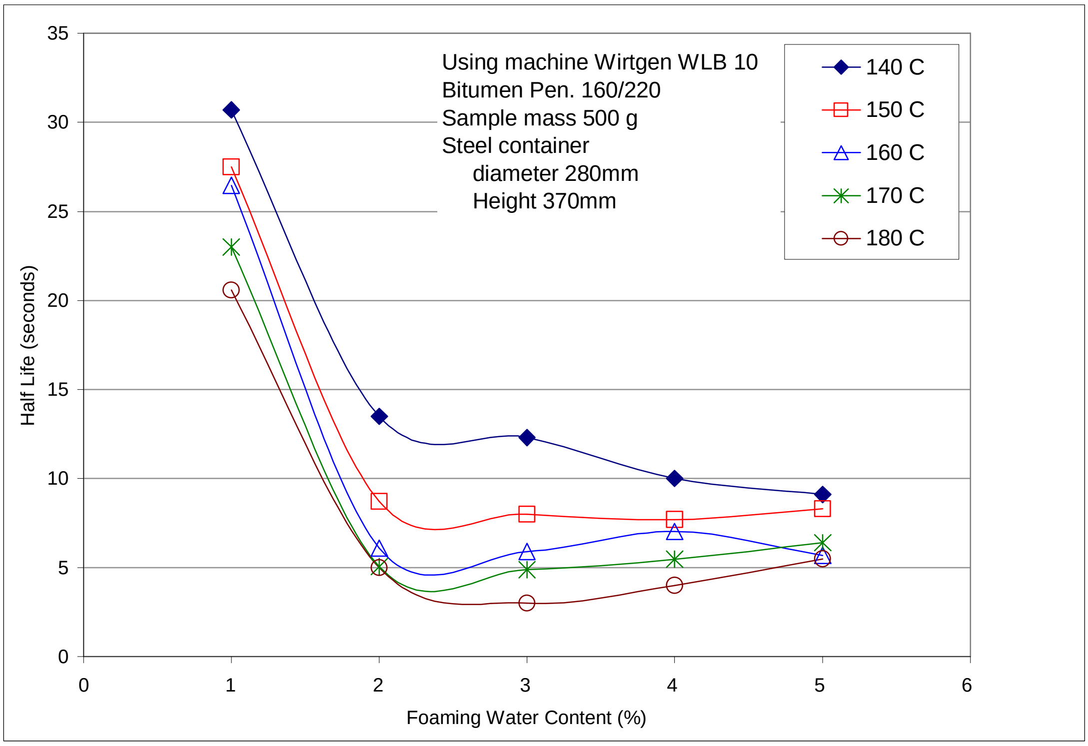


Figure 4.19 - Effect of FWC on the half life of foamed bitumen produced using bitumen Pen 160/220.

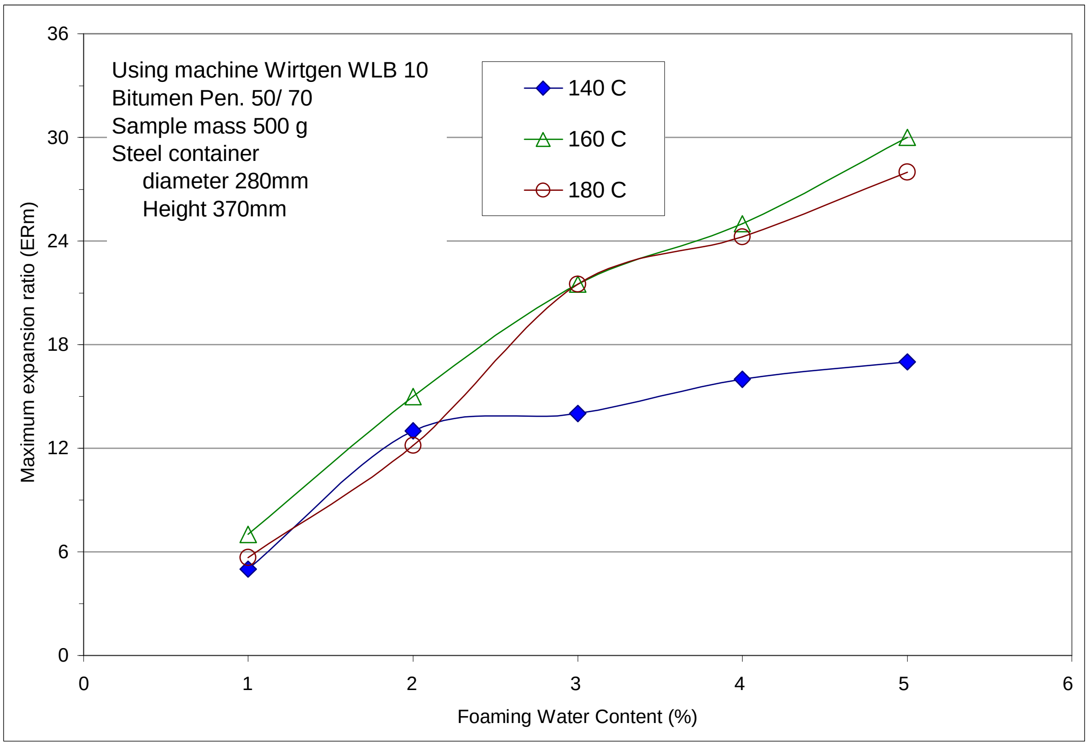


Figure 4.20 - Effect of FWC on the maximum expansion ratio of foamed bitumen produced using bitumen Pen 50/70.

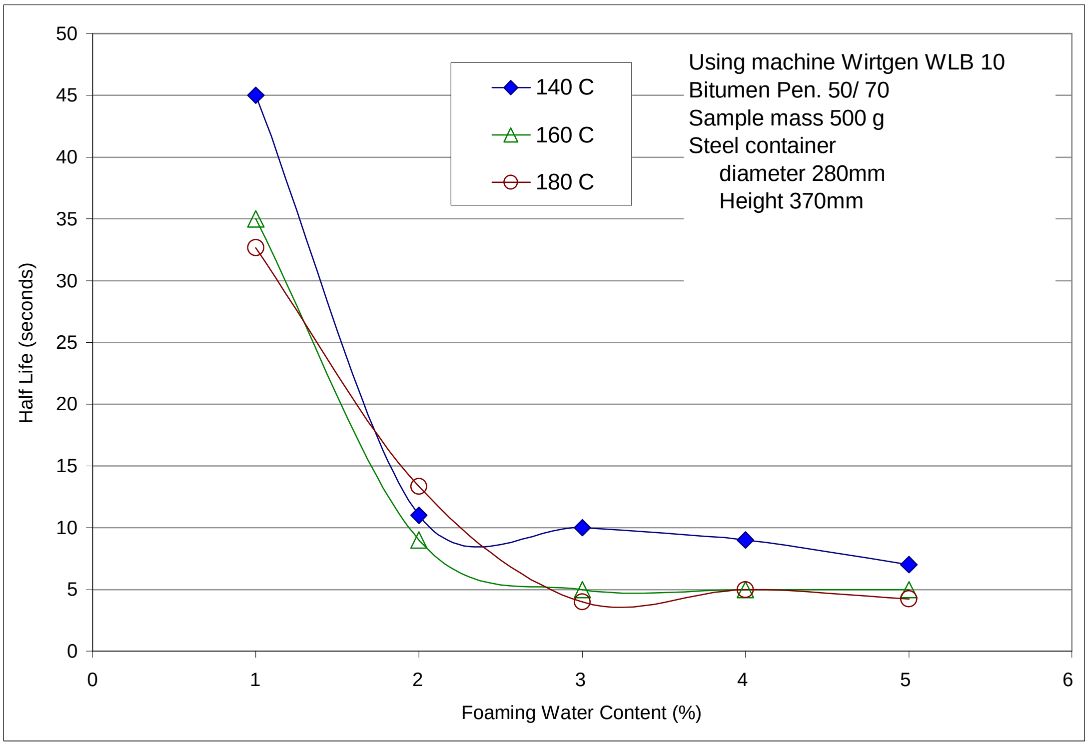


Figure 4.21 - Effect of FWC on the half life of foamed bitumen produced using bitumen Pen 50/70.

4.5.2 Effect of bitumen temperature

Figure 4.22 to Figure 4.27 were also modified from Figure 4.12 to Figure 4.14 in order to show more clearly the effect of bitumen temperature on the maximum expansion ratio (ERm) and the half life (HL) for foamed bitumen produced using bitumen Pen 70/100, Pen 160/220 and Pen 50/70. Bitumen temperature was also found to affect ERm and HL. However, its effect was found not to be as significant as that of FWC effect.

With regard to the effect of bitumen temperature on the ERm value, foam produced using softer bitumen showed opposite characteristics to harder bitumen. For FB 160/220 the ERm values at low temperature (140^oC–160^oC) were higher than at high temperature (160^oC-180^oC), but for FB 50/70 the effect was reversed. This can be seen most clearly for higher FWC applications. Noticeably, FB 50/70s produced at a temperature of 140^oC exhibited very low ERm, especially at FWC applications of 3% to 5%. For FB 70/100, the effect of bitumen temperature on the ERm values was inconsistent.

On the other hand, the HL tends to decrease with increasing temperature for all FWC applications for FB 160/220 and FB 50/70. The exception is FB 50/70 at FWC of 2%. Again, the effect of bitumen temperature on the HL for FB 70/100 is inconsistent. The longest HL value is found to be 45 seconds at FB 50/70: 1%-140^oC and the shortest one is about 3 seconds which is found at FB 160/220: 3%-180^oC.

It is well known that viscosity of bitumen will decrease with increasing temperature. Both the temperature and its resulting viscosity influence the foaming process significantly. It is clear that the increasing bitumen temperature has complex effects on the ERm and HL values. It can then be understood why bitumen temperature variables give differing effects on the foam properties. This can be explained as follows:

- Decreasing bitumen viscosity (bitumen becoming softer) will allow steam bubbles to grow and expand easily within the liquid bitumen phase. This might increase foam volume (or decrease foam density) and hence the ERm value increases.

- If bitumen temperature increases, the resultant foam temperature will also increase and hence the foam will perform more unstably (the HL value decreases).
- If the bitumen viscosity decreases, the surface tension of the bitumen film will also decrease. This might reduce the strength of the thin film in counteracting the internal bubble pressure (in the case of explosive bubbles) and hence the foam will perform less stably (the HL value decreases). On the other hand, decreasing surface tension of the bitumen film may also counteract the Plateau border suction and hence reduce the drainage process (the HL tends to be longer); this may occur rarely since decreasing bitumen viscosity will cause decreasing surface tension of liquid both on the thin film and the Plateau border.
- When foams expand, decreasing surface tension will cause bubbles to tend to fail before reaching their maximum volume and hence potentially reduce the ERm value.

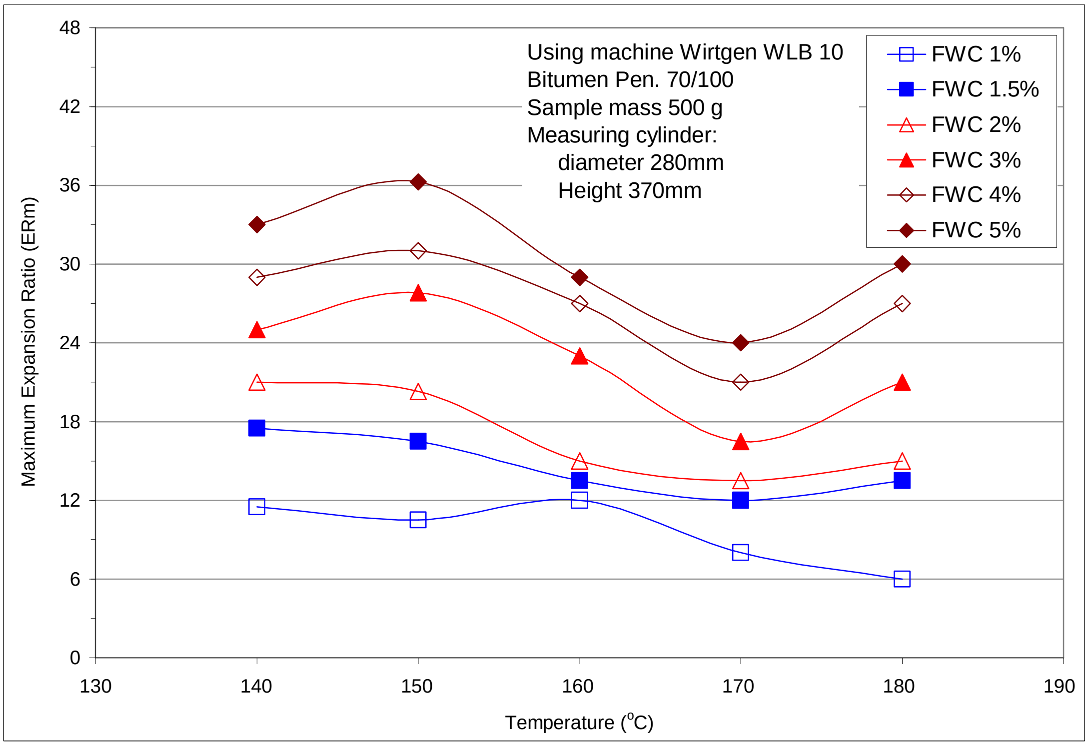


Figure 4.22 - Effect of bitumen temperature on the maximum expansion ratio (ERm) of foamed bitumen produced using bitumen Pen 70/100.

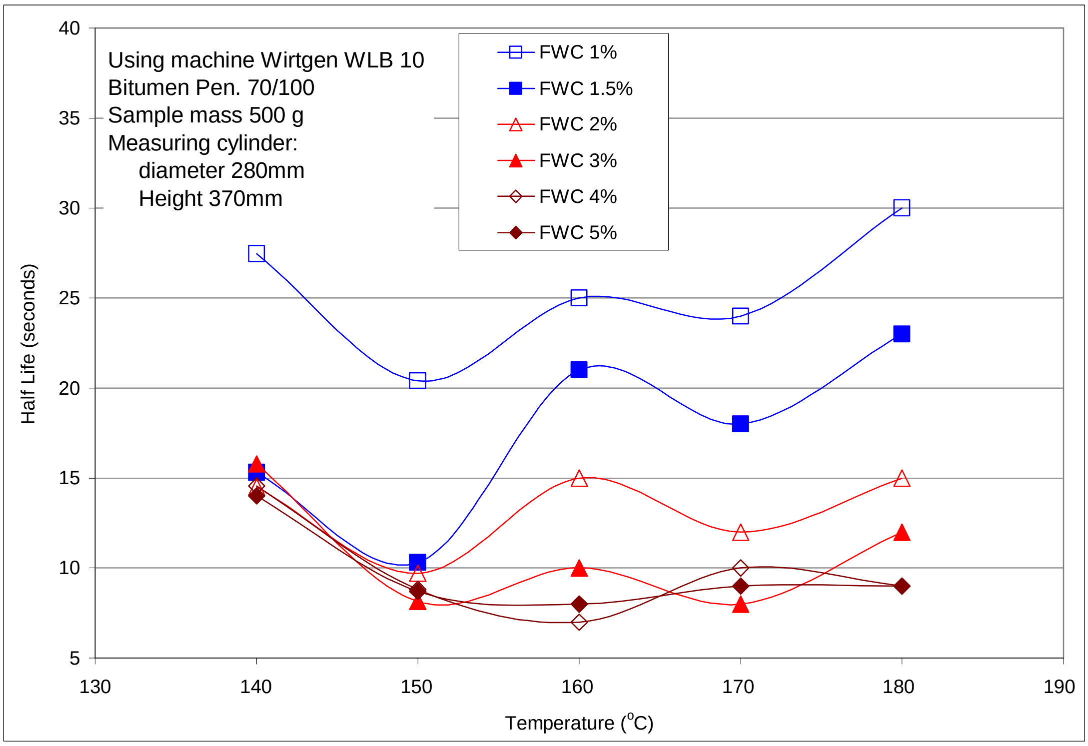


Figure 4.23 - Effect of bitumen temperature on the half life (HL) of foamed bitumen produced using bitumen Pen 70/100.

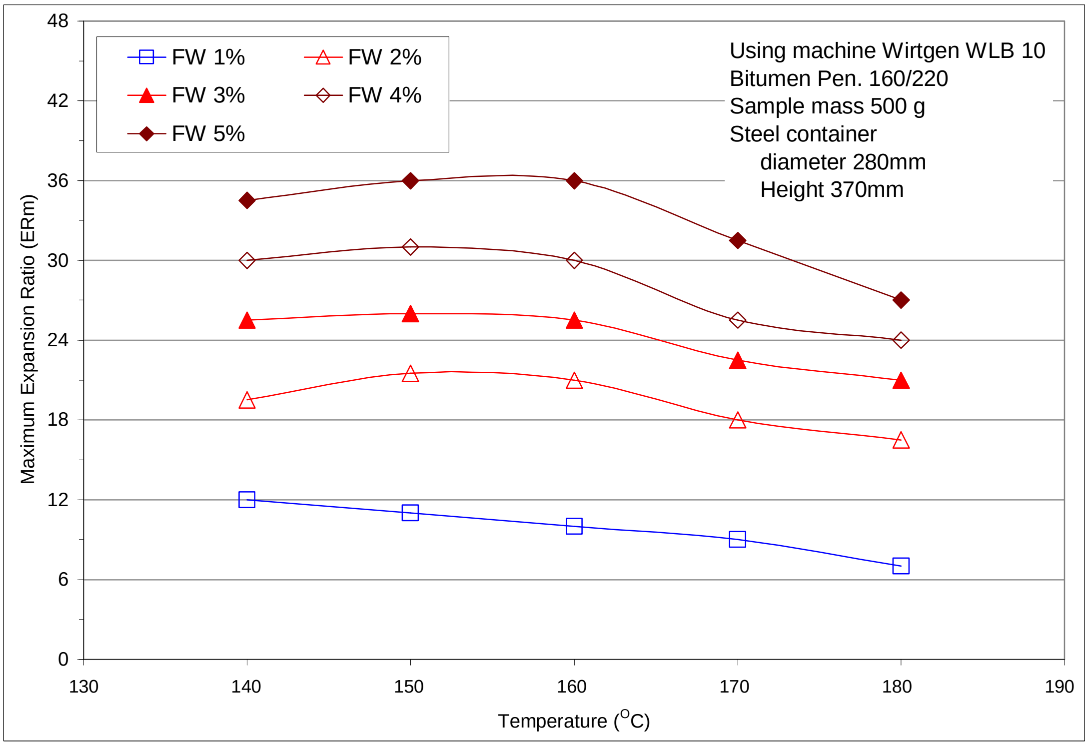


Figure 4.24 - Effect of bitumen temperature on the maximum expansion ratio (ERm) of foamed bitumen produced using bitumen Pen 160/220.

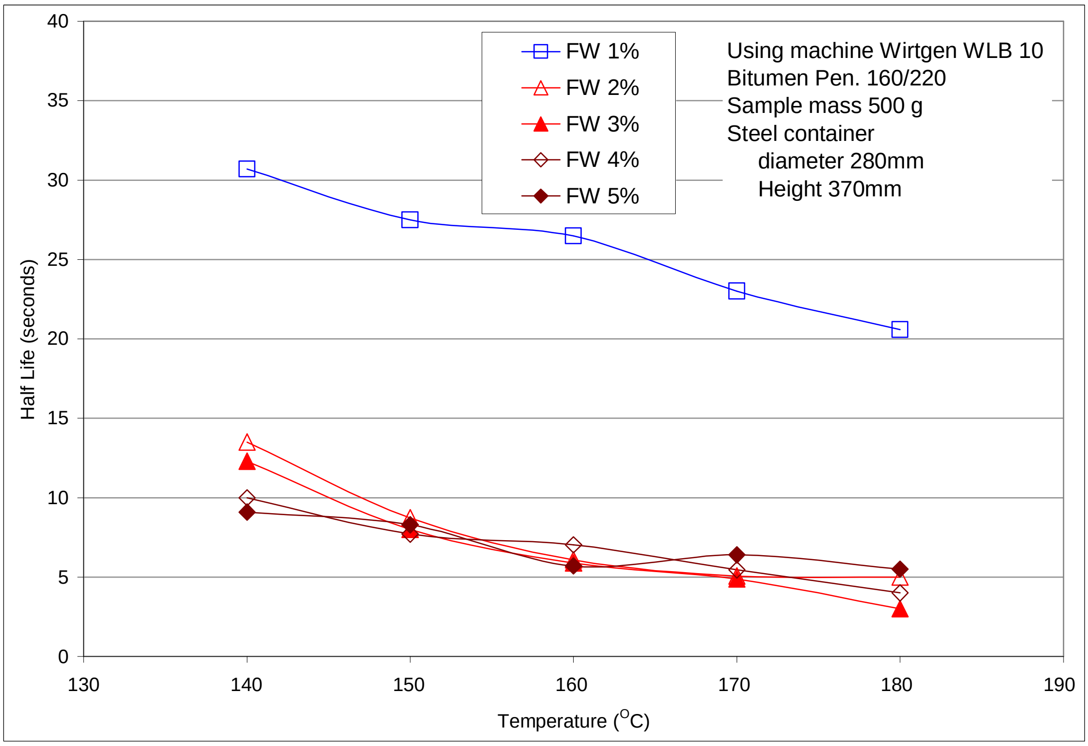


Figure 4.25 - Effect of bitumen temperature on the half life (HL) of foamed bitumen produced using bitumen Pen 160/220.

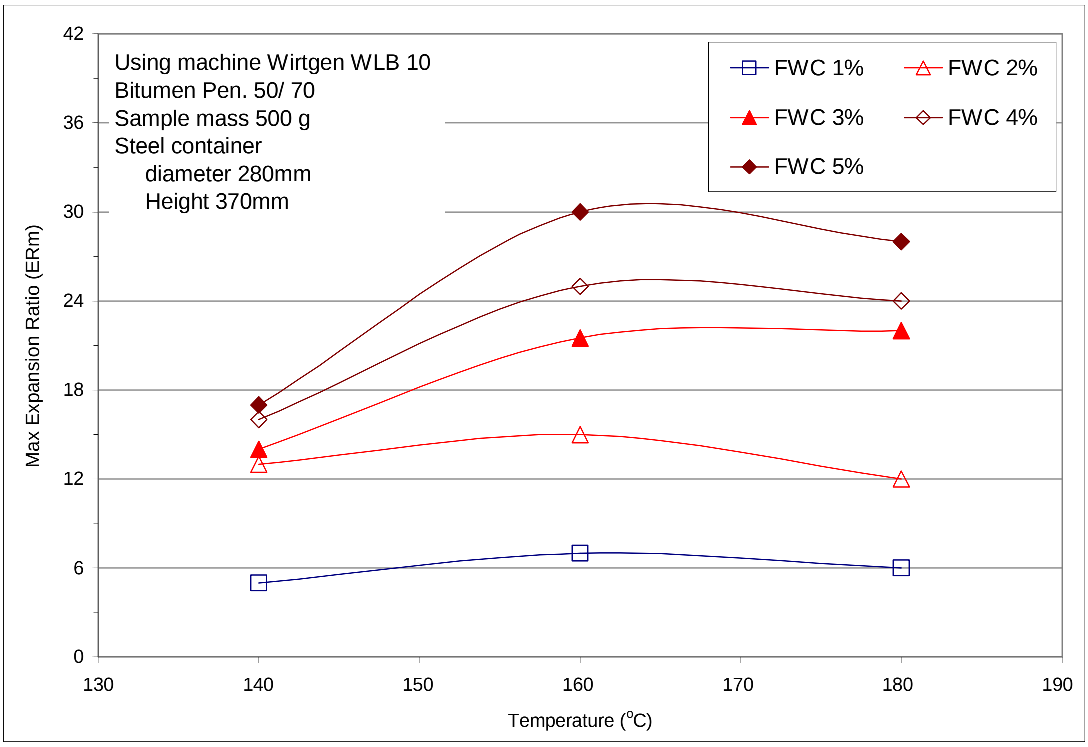


Figure 4.26 - Effect of bitumen temperature on the maximum expansion ratio (ERm) of foamed bitumen produced using bitumen Pen 50/70.

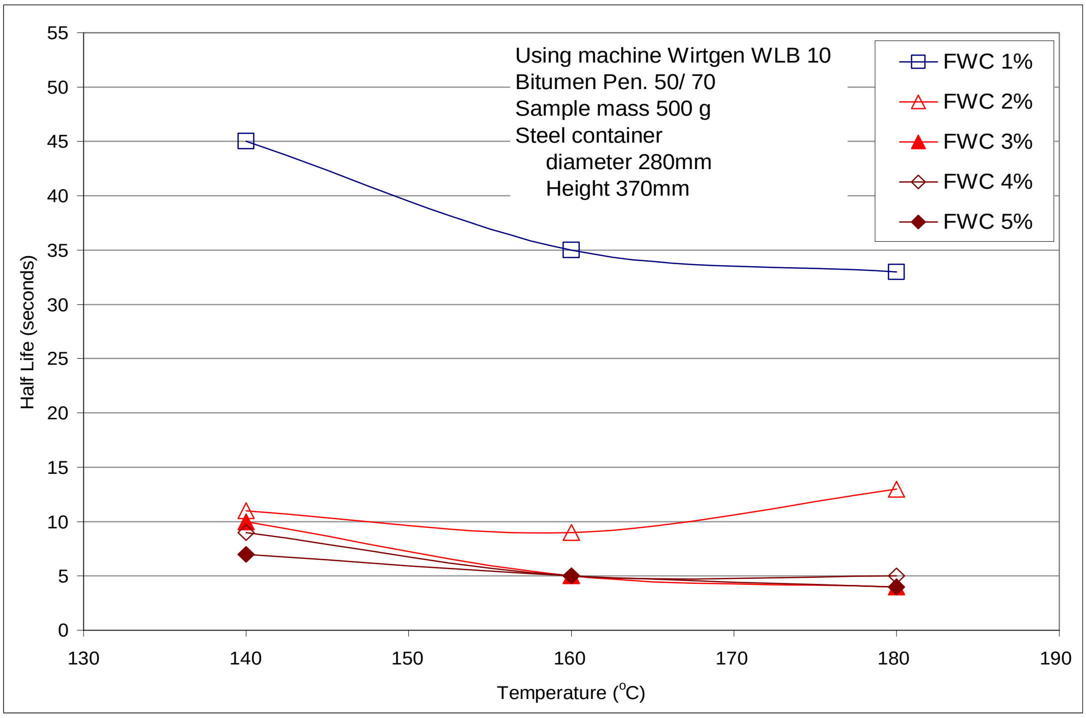


Figure 4.27 - Effect of bitumen temperature on the half life (HL) of foamed bitumen produced using bitumen Pen 50/70.

4.6 Foam Decay and Foam Index (FI)

4.6.1 Characteristics of foamed bitumen decay

Foamed bitumen decay has been investigated using a digital video camera. Any change of foam height in the measuring cylinder with time can be recorded more accurately than by visual observation. This measurement may therefore minimize human error especially on recording the HL value. In this study, the whole foaming process was recorded from the point when foam began to spray to the point when the foam had almost completely dissipated (recorded until 60 seconds). Unfortunately, the volume change when ERm < 6 (for 500g bitumen mass) or < 12 (for 250g bitumen mass) is still not totally accurate since the camera can not clearly record the vertical movement of foam height at this depth.

In this investigation, foams were produced using bitumen Pen 70/100 with varied FWC applications up to 10%. A bitumen mass of 500 grams was used to investigate the foam decay at FWC of 5% and less, whereas a bitumen mass of 250 grams was utilised at higher FWCs to enable the foam volume to be accommodated in the measuring cylinder. At a particular height level in the cylinder, the ERm of a 250g mass is double that of a 500g mass. The results were presented in Figure 4.28 to Figure 4.30.

Figure 4.29 shows the test results of foam decay at FWC of 5% measured using bitumen masses of both 500g and 250g. It can be seen that the measurement using 250g results in a slightly longer half life, by about 2 seconds. This value was used as an approximate correction factor on the test results at higher FWC applications. It is found that the HL tends to be longer with higher ERm values for FWC greater than 5%, which is thought to relate to the effect of foam temperature as discussed in Section 4.5.1. HL is influenced by both foam structure and foam temperature. Dry foams are more unstable than wet foams, but foams at a low temperature are more stable than at a high temperature. Based on Figure 4.28 and Figure 4.30, the HL is controlled by foam structure at FWC $\leq$ 4% (HL decreases with FWC) and by foam temperature at FWC $\geq$ 5% (HL increases with FWC), and it is constant at FWC of 4% and 5% where the effects of structure and foam temperature approximately balance.

Interestingly, as shown in Figure 4.28 and Figure 4.30, the decay geometry appears different between low and high FWC. At low FWC (FWC of 1-2%), the maximum volume (ERm) occurs immediately after spraying finishes (or at a spraying time of 5 seconds, see Figure 4.28). For FWC at 3%, the foam remains standing at maximum volume for several seconds. However, at higher FWCs (>3%), after spraying finishes, the foam volume still increases gradually and hence the ERm occurs several seconds after spraying finishes. The reason for these observations can be explained as follows. It was known that some bubbles may rupture before reaching the peak volume due to their surface tension being low, whereas unbroken bubbles may accumulate continually and this process takes more time to achieve the maximum volume. Logically, the higher FWC will produce more steam bubbles and hence the unbroken bubbles at high FWC will be of great volume and continue for a longer time than at low FWC. It can then be understood that ERm at high FWC will occur later than at low FWC.

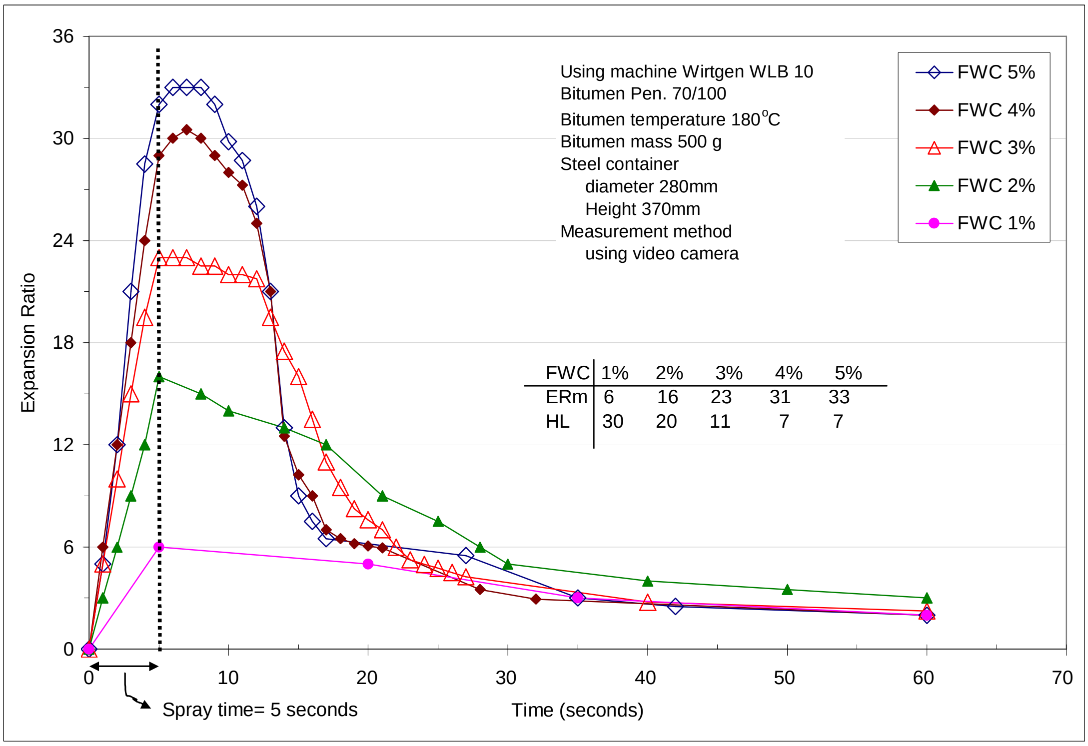


Figure 4.28 - Foam decay at FWC up to 5% using bitumen Pen 70/100

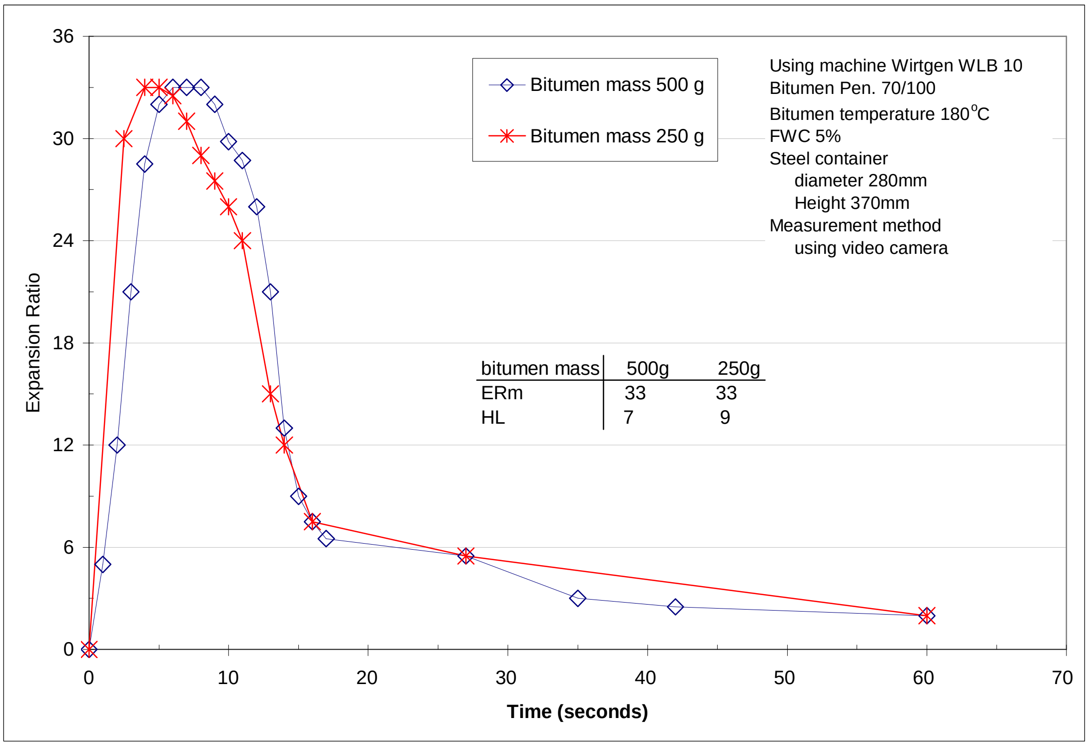


Figure 4.29 - Comparison of foam decay measurement methods between using bitumen (Pen 70/100) mass of 500 g and 250 g at FWC of 5%.

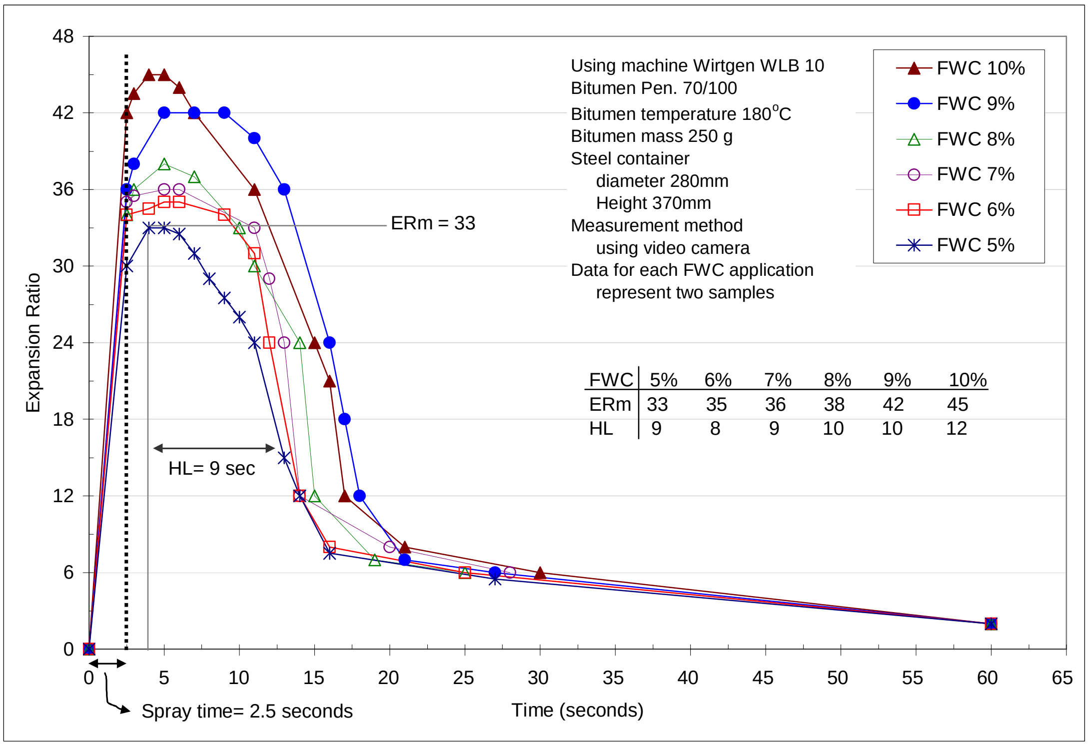


Figure 4.30 - Foam decay for FWC greater than 5% using bitumen Pen 70/100

4.6.2 Foam Index (FI)

The area under the foam decay curve is representative of the accumulation of foam volume and therefore it may correlate to the surface energy of foam. Therefore, foam decay becomes an important factor in order to characterise foamed bitumen. The area under the foam decay curve has been defined by Jenkins (1999) in terms of a 'foam index' (FI), as discussed in Chapter 2. For this study, it is therefore important to know the influence of FWC on the FI value.

Table 4.3 provides the calculations of FI for foamed bitumen produced using bitumen Pen.70/100 at a bitumen temperature of 180°C. These calculations use the equation and graph shown in Chapter 2 section 2.4.1.2. The resulting values of FI are plotted in Figure 4.31, including the values of corresponding ERm and HL. It can be seen that the FI value is mainly affected by the ERm and hence it increases with FWC. Figure 4.32 shows the correlation between values of ERm, HL and FI in Jenkins theory (1999), for which the FI value will be low at low ERm, even though the HL value is high; and the FI value can be high at low HL due to the ERm value being high.

Table 4.3 - Calculation of Foam Index (FI) value for foamed bitumen produced using bitumen Pen.70/100 at bitumen temperature of 180°C

FWC (%)	1	2	3	4	5	6	7	8	9	10
HL (s)	30	20	11	7	7	6	7	8	8	10
ER	6	16	23	31	33	35	36	38	42	45
c=ERm/ERa	0.93	0.91	0.83	0.77	0.77	0.74	0.77	0.79	0.79	0.825
A1	16	186	190	190	208	193	234	288	330	452
A2	31	84	127	178	190	206	207	215	238	249
FI (sec.)	47	270	317	368	397	399	441	504	568	701
FI (10 sec.)	5	27	32	37	40	40	44	50	57	70

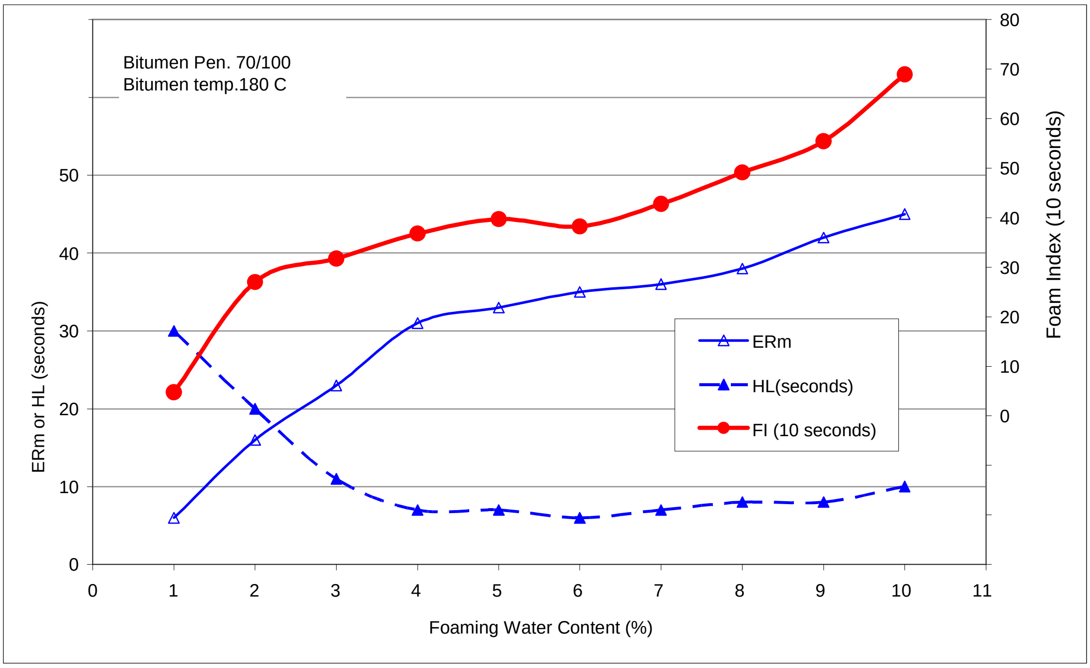


Figure 4.31 - Effect of foaming water content (FWC) on the Foam Index (FI) value

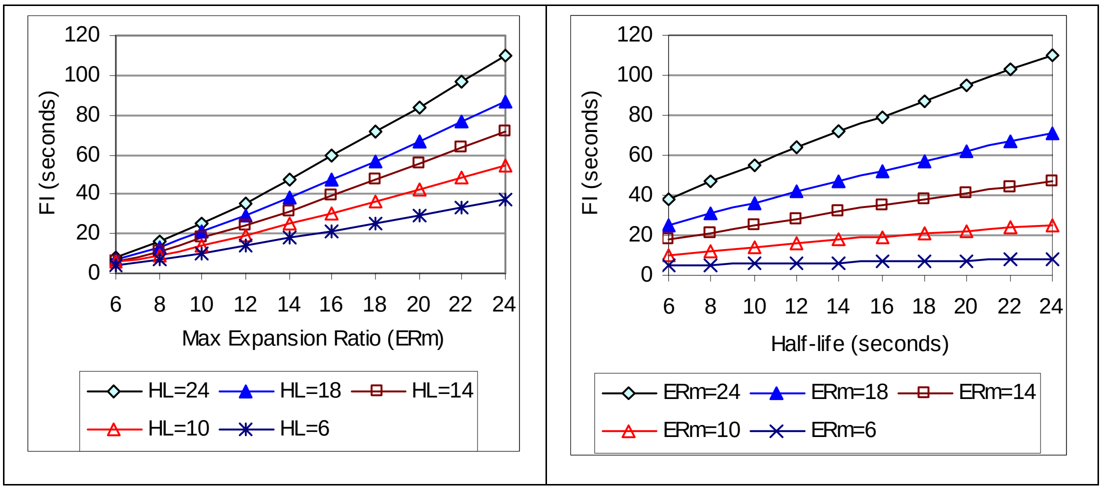


Figure 4.32 - Effect of ERm (left) and HL (right) on the FI value in Jenkins (1999) theory

4.7 Flow Behaviour of Foamed Bitumen

Flow behaviour was investigated at selected bitumen temperature and FWC values. The result may relate to the workability of foamed bitumen during mixing process. Foam flow may indicate the apparent foam viscosity, which may complement Jenkins' (2000) and Saleh's (2006a) investigations (see Chapter 2 section 2.4.1.3). The method used in this study was adapted from the standard tar viscometer technique, i.e. measurement of flow rate through an orifice to establish viscosity of the material (Read and Whiteoak, 2003). In the standard tar viscometer, a metal cup is filled with cutback bitumen or emulsion at a standard temperature and the time is recorded in seconds for a standard volume of material to flow out through the orifice in the bottom of the cup. The absolute viscosity, ή (in Pa.s), is a multiplication of flow time (seconds), density (g/ml) and a constant. For the standard tar viscometer using a 10mm orifice, the constant value is 0.400.

In this study, 200 grams of sprayed foamed bitumen produced using bitumen Pen 70/100 was collected in a container (diameter 200mm and height 300mm) and allowed to flow directly through an orifice(s) created at the bottom of the container. The foam collected through the orifice was then directly weighed with time. Using this method, the change in foam consistency with time was observed up to the point when flow values were too small to record. The foam flow behaviour was investigated using three different orifice diameters i.e. 3mm (9 holes), 9mm (1 hole and 3 holes) and 27mm (1 hole). More than one hole was used in order to accelerate the flow of foam. The ratio between the size of orifice and bubble size controls the foam flow behaviour. Based on observation, foam bubbles may expand up to 30mm diameter (Jenkins, 2000, predicted a bubble radius of about 10mm). It is therefore supposed that foam flow through a 27mm orifice will be more likely to represent bulk foam flow than foam flow through a smaller orifice. On the other hand, foam flow through a 3mm orifice might be more representative of foam drainage than foam flow through a larger orifice.

It should be noted that the test used in this study is not a standard method and the results are only expected to give an indication of viscosity/ flow behaviour/

consistency of foamed bitumen. The results, of course, are very useful as a tool to distinguish foamed bitumens with different properties. The completed test results can be found in Appendix A: Figure A.4.2 and A.4.3. Figure A.4.2 shows the flow rate through a 9mm orifice of foamed bitumen produced at various selected bitumen temperatures and FWC applications. Figure A.4.3 demonstrates the flow behaviour of foamed bitumen produced at a bitumen temperature of 180^{o}C and at various FWC applications using three orifice diameters.

4.7.1 Characteristics of foamed bitumen flow

Figure 4.33 shows the test result of foamed bitumen and hot bitumen flowing out through 3mm orifice (9 holes). The temperature of the hot bitumen was 180^{o}C whereas the foamed bitumen was produced at a temperature of 180^{o}C and a FWC of 5%. After studying many foam flow rate curves at various temperatures and foaming water contents, the foaming curve can be divided into 3 phases as shown in the figure, i.e. foam state, fluid state and viscous state. For hot bitumen, however, there are just 2 phases, i.e. fluid and viscous states. The foam state starts at the beginning of foam spraying and finishes when the foam starts to flow out rapidly. In this phase, foam appears to have difficulty in flowing out through the orifice due to the interconnecting bubbles within the foam structure blocking the orifice. The fluid state is indicated by the flow rate increasing rapidly; and it lasts longer than the first phase. Most of the foam flows out in this phase. The last phase is the viscous state, for which foam flow through the orifice becomes very slow due to it becoming stickier.

It was found that, in all cases, the foamed bitumen including hot bitumen was unable to flow out of the container completely. This was attributed to the bitumen/foam cooling down rapidly and becoming too stiff to flow within a reasonable experimental time. As an example, only 155g of foam produced at a temperature of 180^{o}C and FWC of 1% was able to flow through a 27mm orifice, out of a 200g total (see Appendix A: Figure A.4.2).

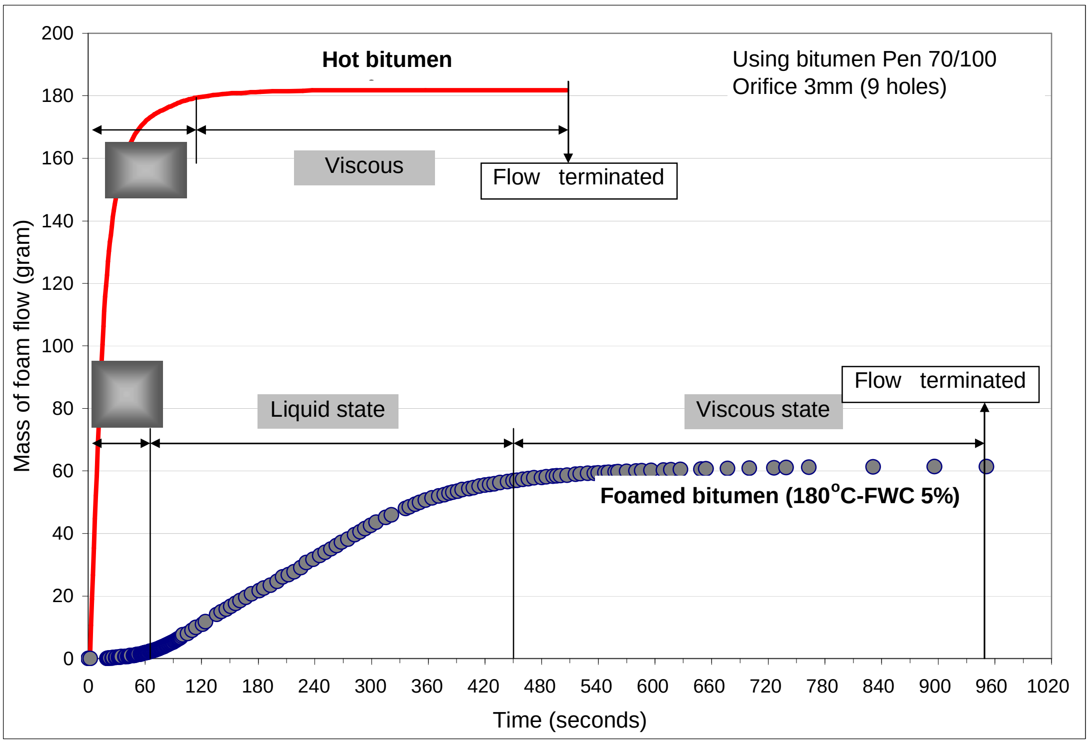


Figure 4.33 - Characteristics of foamed bitumen flow through orifices compared to that of hot bitumen flow (using bitumen Pen 70/100).

4.7.2 Effect of foaming water content (FWC) on the foam flow

The effects of FWC on the flow behaviour can be clearly observed from the test results (see Figure 4.34). In general, the rate of foam flow through orifice(s) tends to increase with decreasing FWC.

Figure 4.34 shows the effects of FWC on the rate of foam flow through 9x3mm orifices. A foam with a higher FWC contains by default a higher gas content thus resulting in larger sized bubbles which in turn promotes stronger interaction within the foam structure and retards flow. This phenomenon gives rise to non-Newtonian rheological effects (Kraynik, 1988). It is thus suggested that flow rate may able to be linked to viscosity as in the tar standard viscometer. It can thus be realized that a foamed bitumen with a higher FWC tends to have a decreased flow rate and hence higher apparent viscosity, which is in line with the findings of Assar and Burley (1986).

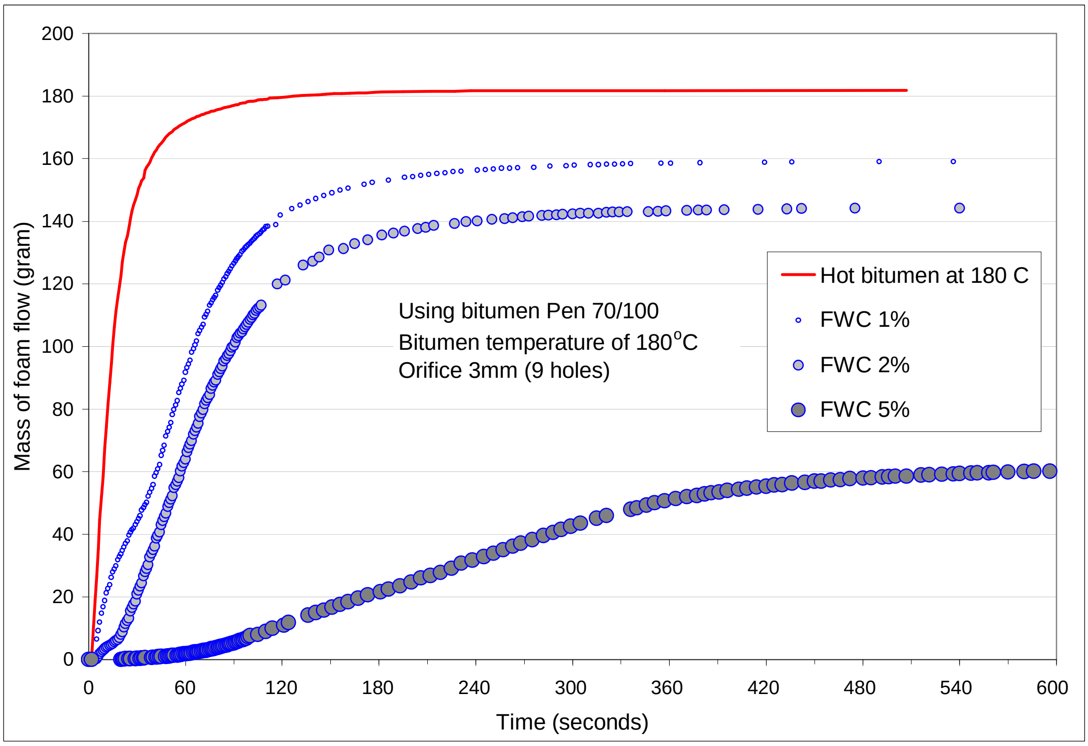


Figure 4.34 - Effect of foaming water content on the flow behaviour of foamed bitumen (produced using bitumen Pen 70/100 at a temperature of 180°C).

When foam flow was examined using larger orifices, the effect of FWC on the flow rate of foam followed a similar trend to that through 3mm orifices. However, the effect of FWC was less significant when a larger orifice was used. It is noted that foam produced at FWC 1% flowed extremely fast through a 27mm orifice, which was not the case at other FWC. It appears that at lower FWC and with a large orifice, the foam can flow out without any obstacle from foam bubbles and this looks more representative of bulk foam flow; whereas at higher FWC and with a small orifice, the flow is more representative of foam drainage.

4.7.3 Effect of bitumen temperature on foam flow

The effects of bitumen temperature on flow behaviour can also be observed from the test results (see Figure 4.35). Generally, the rate of foam flow through an orifice tends to decrease with decreasing bitumen temperature. This indicates that decreasing bitumen temperature results in reduction of foam temperature and hence reduces the rate of foam flow.

In addition, the effect of bitumen temperature at low FWC is clearer than at high FWC (see Appendix A: Figure A.4.2). As an example, the flow rate of FB 70/100: FWC 1% produced at 180°C was clearly faster than that produced at 140°C; but this observation was unclear for FB 70/100: FWC 5%.

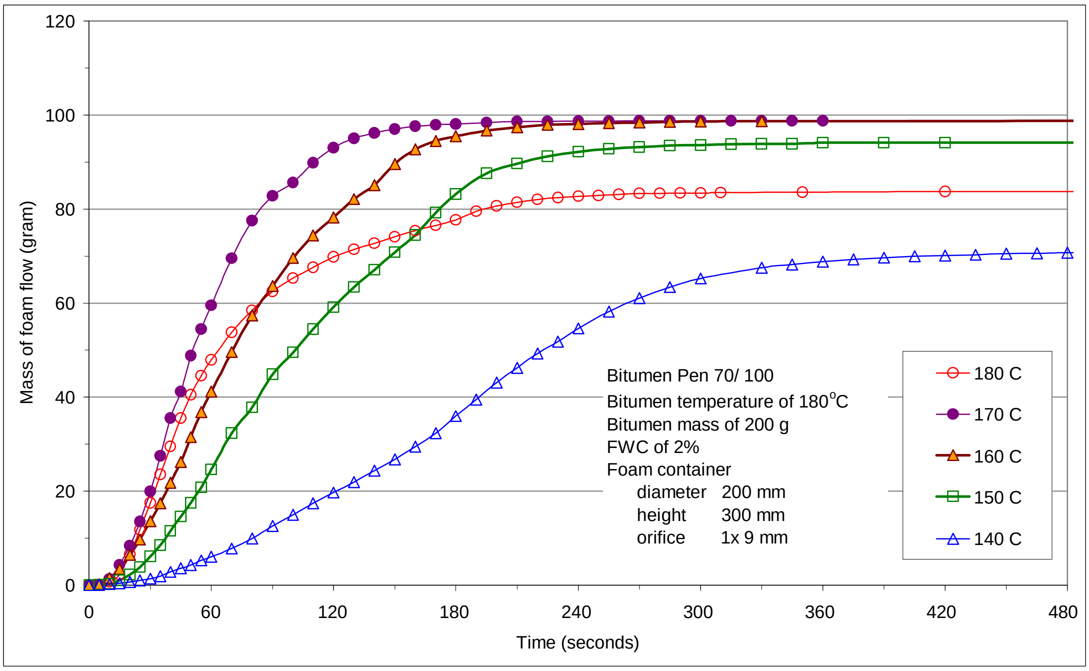


Figure 4.35 - Effect of bitumen temperature on the flow behaviour of foamed bitumen (foam produced using bitumen Pen 70/100 at FWC of 2%).

4.7.4 Foamed bitumen life

From Figure 4.34, a foam with a higher FWC indicates slower drainage of liquid since a longer time was required to terminate the flow. This means that a high quality (high ERm) foam tends to be slower in temperature reduction and hence have a longer life (the whole life of foam before it changes to a liquid state).

Flow rate testing results can also be utilised to predict foam life (FL). This value may be useful in terms of the allowable mixing time, when foam is mixed with aggregate. This value does not represent 100% foam life due to the line of flow rate becoming asymptotic. To predict the FL value, the acceleration of foam flow should be calculated. Therefore, curves of flow acceleration (milligrams per second squared) against time (seconds) can be constructed. The curve peak within the foam state region indicates the foam life (reduced by the spraying time of 2 seconds) since beyond this point foam changes to a liquid state (Figure 4.33). The curve peak should be selected properly since the curve fluctuates as the foam is sprayed into the container. Figure 4.36 demonstrates foam life-time for one test. Table 4.4 provides the foam life values resulting from foam flow rate testing as defined in Figure 4.36. These values are shown in Figure 4.37 at various bitumen temperatures (a) and foaming water contents (b) together with the corresponding half-life resulting from measurement as shown in Figure 4.12.

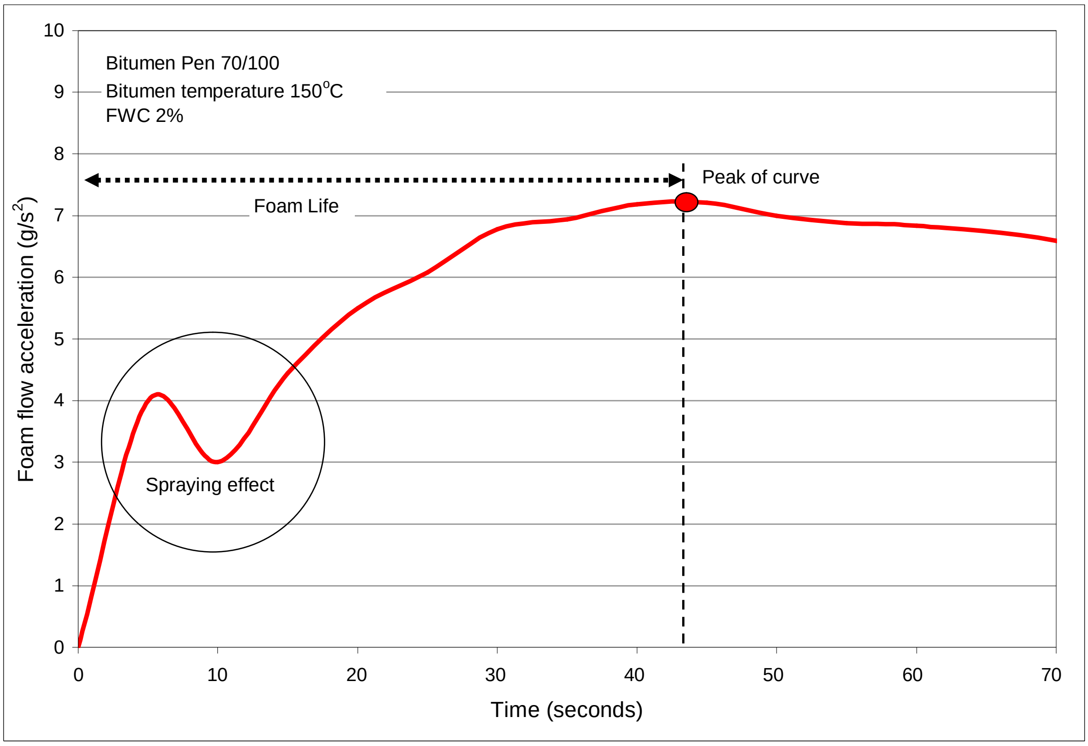


Figure 4.36 - Prediction of foam life

It can be seen that at a FWC of 2% (Figure 4.37a), the HL values fluctuate with various temperatures, but the foam life is clearly higher at lower temperature. It is noted that for FB 50/70 and FB 160/220, the HL of foam is also found to increase at lower temperature. In contrast, at a bitumen temperature of 140°C (Figure 4.37b), the difference between FL and HL at a FWC of 5% is much higher than at a FWC of 1% or 2%. The trend of FL values that increases sharply at FWCs higher than 2% is supposed to be effect of drainage process. At high FWC, the drainage tends to be more slowly and hence foam remains wet which causes foam being more stable.

The results indicate that the life of the foam may be a useful quantity to be considered in order to characterise foam stability, in addition to the HL. For general foam, Breward (1999) found that foam collapses after approximately 90 seconds considering fluid drain from the foam and ignoring the slip factor between the lamella and Plateau border.

Table 4.4 - Foam Life (seconds) predicted from Flow Acceleration Curve

Foaming Water Content (%)	Temperature (^oC)				
	140	150	160	170	180
1	58	-	-	-	58
2	48	43	33	33	33
5	138	148	-	-	48

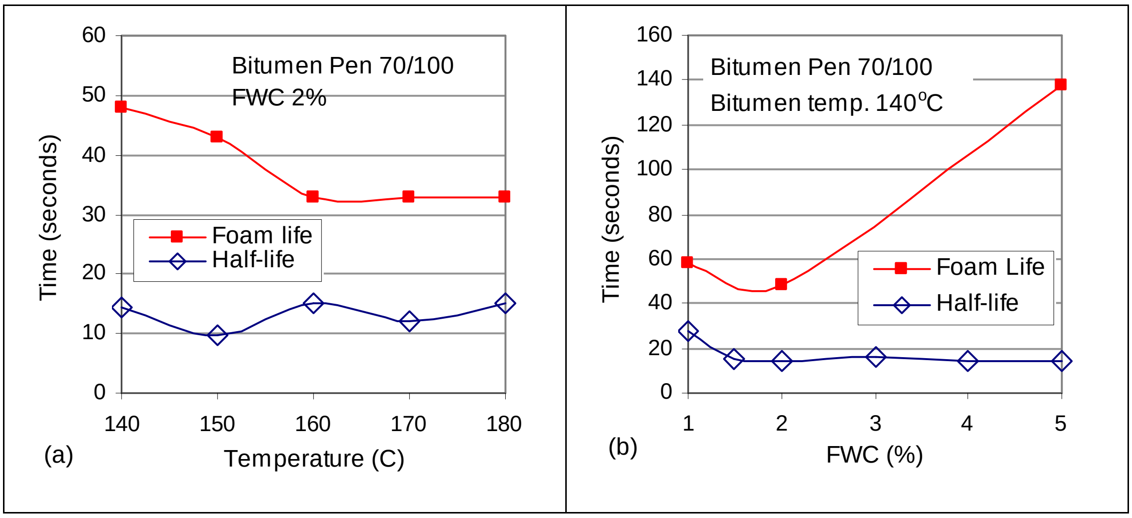


Figure 4.37 - Foam life and half-life at various temperatures and FWCs

4.8 Estimating Foamed Bitumen Viscosity

It is required to know foamed bitumen viscosity since it is an important property in terms of the workability of foam binder in the mixing process. Based upon the results of foam flow investigation, a foam with a higher FWC is found to flow slower than a foam with a lower FWC. In line with Kraynik (1988), it is suggested that foam flow can be linked to its viscosity. It can be deduced that a foam with a higher FWC tends to have a decreased flow rate and hence higher viscosity, which agrees with Marsden and Khan's finding (see Heller and Kuntamukkula, 1987).

Apparent foam viscosity may also be estimated by adapting the Taylor equation (1932) for dilute emulsion as expressed in Eq.4.11.

$$\mu_e = \mu\{1 + Ø_d \left[\frac{5\lambda + 2}{2(\lambda + 1)} \right] \} \text{ ……………………….Eq. 4.11}$$

where:

μ_e = viscosity of dilute emulsion

μ = viscosity of continuous phase

$Ø_d$ = volume fraction of dispersed phase

$\lambda = \frac{\mu_d}{\mu}$

μ_d = viscosity of dispersed phase

Kraynik (1988) discussed the possibility of applying the Taylor equation for a dilute suspension of spherical gas bubbles. In this case, liquid acts as the continuous phase and gas acts as the dispersed phase. Because $\mu_d <<< \mu$, so that $\lambda \approx 0$, then a simple equation was proposed as shown in Eq. 4.12. Mitchell (1971) also developed an equation based upon a drill oil foam experiment. The equation as shown in Eq. 4.13 was developed using a statistical approach for foam behaving like a Newtonian fluid for gas contents up to 55%, independent of shear rate.

$$\mu_{ef} = \mu_l [1 + Ø_d] \text{ ……………………………Eq. 4.12}$$

$$\mu ef = \mu l [1 + 3.6 Ød] \text{ …..…..……………...Eq. 4.13}$$

where:

μ_{ef} = effective viscosity of foam

μ_l = viscosity of liquid or continuous phase

$Ø_d$ = volume fraction of gas or dispersed phase

From both of Kraynik and Mitchell’s equations, it can be noted that the foam viscosity is proportional to liquid viscosity and gas content. The apparent foam viscosity is higher than the liquid viscosity due to the presence of bubbles disturbing the external flow. Fernando and Shah (2000) added that the apparent foam viscosity increases exponentially with gas content. It can be deduced that the coefficient of gas content (3.6 in Eq. 4.13) for wet foam is lower than for dry foam. There is no equation proposed for dry foam that expresses the relationship between apparent foam viscosity, liquid viscosity and gas content.

If the viscosity of liquid bitumen and the gas content are known, the apparent foam viscosity can be estimated. Gas content can be calculated from ERm and obtained experimentally. Liquid bitumen viscosity can be estimated from its temperature. It is assumed that foam is an isothermal material in an equilibrium state, hence bitumen temperature is approximately equal to foam temperature, which can be approximated from the heat transfer process, from hot bitumen to the cold water (see section 4.4.2).

In this study, the effective apparent viscosity of foam is calculated using Kraynik's equation, for which the foam temperature is estimated when it is sprayed into the measuring cylinder (see scenario 2 in Figure 4.10). The bitumen viscosity was investigated over the range of foam temperatures after 60 seconds spraying. The result as shown in Figure 4.38 is therefore used to identify the viscosity at various ERm values.

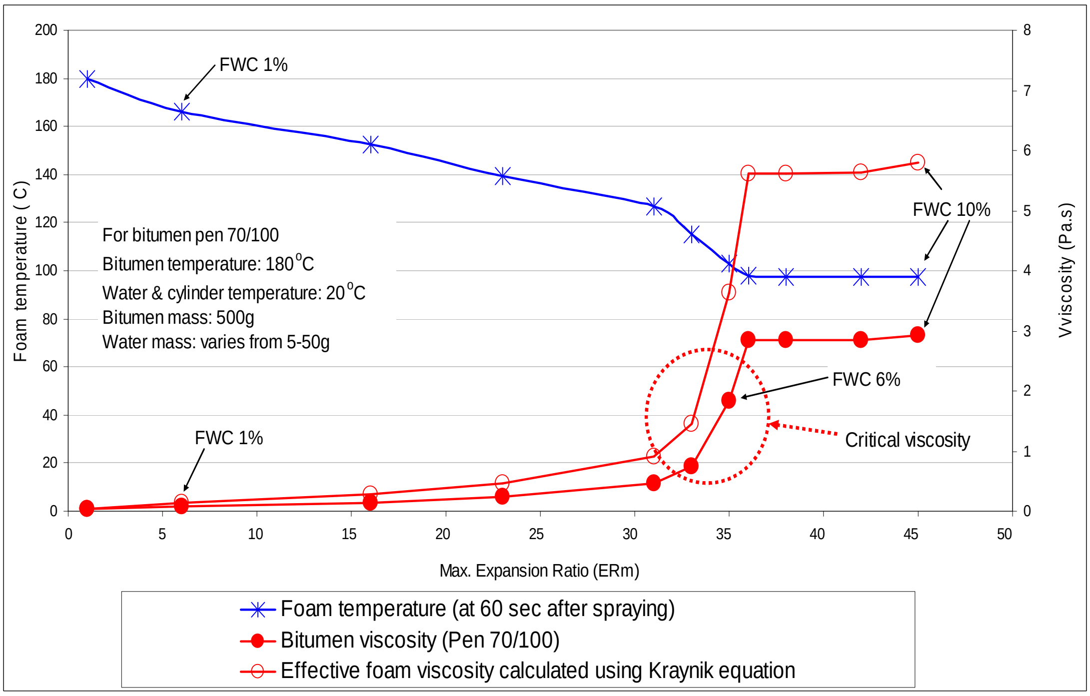


Figure 4.38 - Apparent foam viscosity at various ERm values

As shown in Figure 4.38 , the apparent foam viscosity (for FB 70/100 at 180^{o}C) increases with increasing ERm value. It can be seen that it increases gradually up to an ERm of around 30 and then increases sharply beyond this point. It can be seen that an ERm of around 35 (or at FWC of 6%) is the critical area of foam viscosity. It should be noted that foam viscosity is mainly affected by the liquid bitumen viscosity, which is dependent upon foam temperature, resulting from the heat transfer process. Thus, it can be deduced that foam viscosity will be depend on gas content, bitumen properties and foam temperature.

4.9 Properties of Collapsed Foamed bitumen

The collapsed foam has been investigated using the Penetration test at 25^{o}C, the rolling thin film oven test (RTFOT) and bulk density. The main aim of these tests is to explore the effect of FWC on the foamed bitumen properties after collapse.

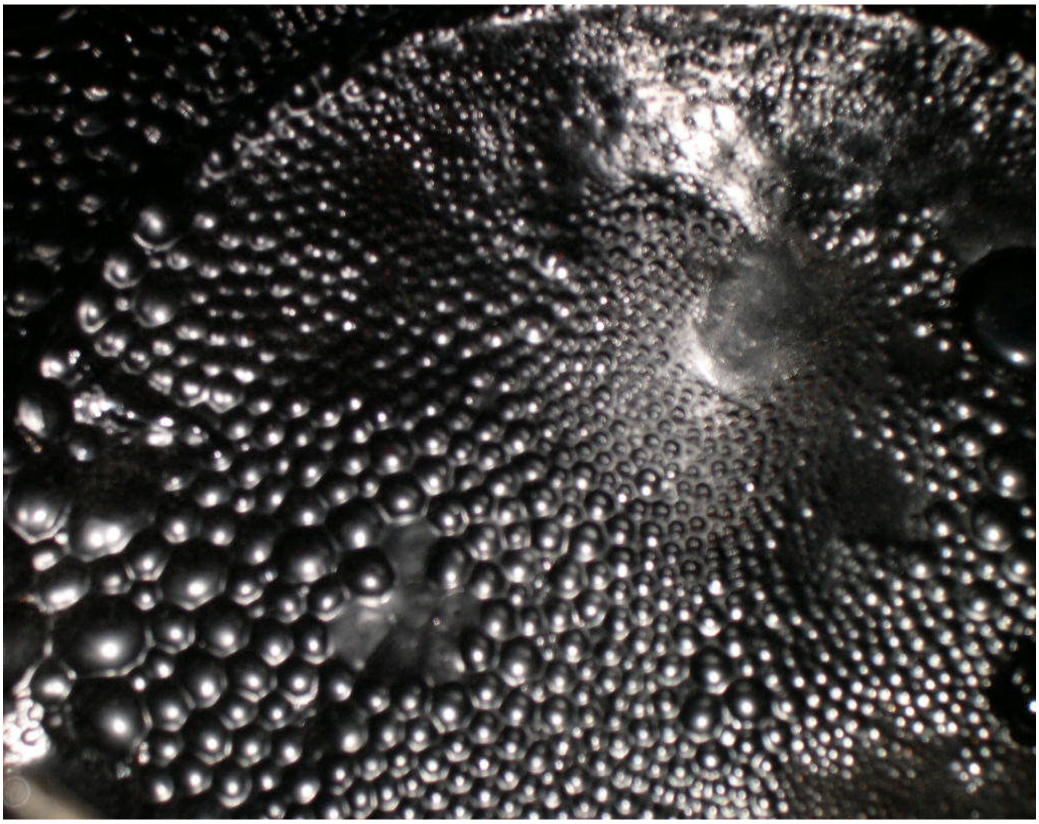


Figure 4.39 - Appearance of the collapsed foamed bitumen remaining in the measuring cylinder several days after foaming

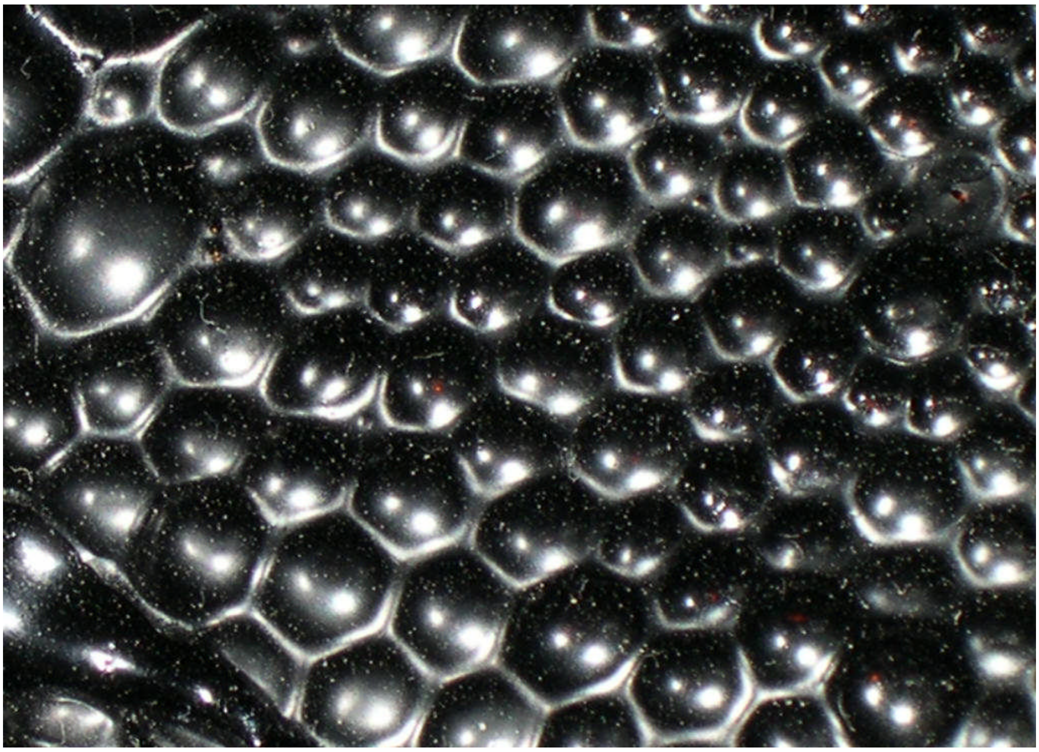


Figure 4.40 - Appearance of bubble structure of the collapsed foamed bitumen

4.9.1 Appearance of the collapsed foamed bitumen

Naturally, foamed bitumen never absolutely returns to its original pure bitumen state, until mixed with aggregate. It is supposed that large number of tiny bubbles and misty water are trapped inside bitumen. This causes the collapsed foamed bitumen to be softer than normal bitumen at ambient temperature. Figure 4.39 shows collapsed foamed bitumen captured in the measuring cylinder several days after foaming. It can be seen clearly that the bubbles still exist on the surface, indicating that the gas is also still trapped inside the bitumen. This effect may occur when the bubbles try to rise from the bitumen after it has become hard. The structure of collapsed foam as shown in Figure 4.40 is similar to foam structure in two dimensions (see Figure 4.3).

4.9.2 Penetration test

Penetration tests have been performed at a temperature of 25^{o}C in accordance with BS 2000-49: 2000. The samples are collected directly from the foaming plant in penetration containers and stored in a cool room (5^{o}C) followed by testing at a selected age (see Figure 4.41). Unfortunately, the remaining bubbles on the tested sample surface cause the collapsed foam to be inhomogeneous, and hence the penetration result was not representative of the whole bitumen sample. As shown in Table 4.5, the penetration of collapsed foam at different FWC values, tested at different foam ages, was inconsistent. When the bubbles were removed by a heating and stirring process, the penetration became close to the original value, i.e. 72 for all FWCs. This means that the foaming process does not significantly change bitumen penetration. It may be the short period of bitumen heating during the foaming process is not sufficient to cause bitumen ageing.

Figure 4.41 - Appearance of the collapsed foamed bitumen in the penetration test container.

Table 4.5 - Results of penetration test on collapsed foam and original bitumen

Foaming water Content (%)	Foam age (days)					after removing bubble
	1	3	7	30	58	
1	69	66	73	60	95	73
2	72	68	68	60	57	73
5	69	67	68	119	55	73
10	78	73	70	58	56	70
Original bitumen						72

Note: Penetration values are in 0.1mm

4.9.3 The Rolling Thin Film Oven Test (RTFOT)

The RTFOT was designed to simulate the short term ageing of bitumen in the laboratory. In this study, the test was used to assess the effect of the foaming process. The samples were collected directly from the foaming plant and tested the next day.

Tests were performed according to BS EN 12607-1: 2000. Samples in glass containers are rotated with an airflow rate of 4000 ml/minute and at temperature of 163^{o}C for 60 minutes. The results can be seen in Table 4.6.

As shown in Table 4.6, the penetration value after ageing of foamed bitumen at various FWCs is not significantly different from the original one. This means that ageing during the foaming process is not significant and hence the binder durability in service will improve. On the other hand, the recorded mass loss of foam is different from the original bitumen and its value at a FWC of 10% is different from other foams. Considering no significant ageing during foaming process, it is supposed that the mass loss during testing of foam samples is not sample ageing but is caused by evaporation of water trapped in the foam sample. This most obviously occurred on the foam sample at a FWC of 10%, with a mass loses of 2.26%, higher than all others.

Table 4.6 - Results of RTFOT on collapsed foam and bitumen

Property	Foaming water content (FWC)				Original bitumen
	1%	2%	5%	10%	
Mass loss (%)	-0.02	0.01	-0.02	2.26	0.18
Pen at 25^{o}C after test (0.1 mm)	47	49	46	43	45

Note: Penetration at 25^{o}C of original bitumen = 72 (0.1 mm)

4.9.4 Bulk density test

The bulk density test was conducted in order to investigate the presence of air bubbles in the collapsed foam. The test has been adapted from BS EN 12697-6: 2003. Three foams (using bitumen Pen 50/70) at different FWCs have been investigated and the results can be seen in Figure 4.42. Prediction of the amount of air bubbles (void content) based on the collapsed foam bulk density can be found in Table 4.7.

The results show that the bulk density of collapsed foam at FWC 10% is lower than at FWC 5% or 2%. Its void content is also found to be highest (around 6.4%). These voids should come from the injected air during the foaming process, which is proportional to the amount of injected water.

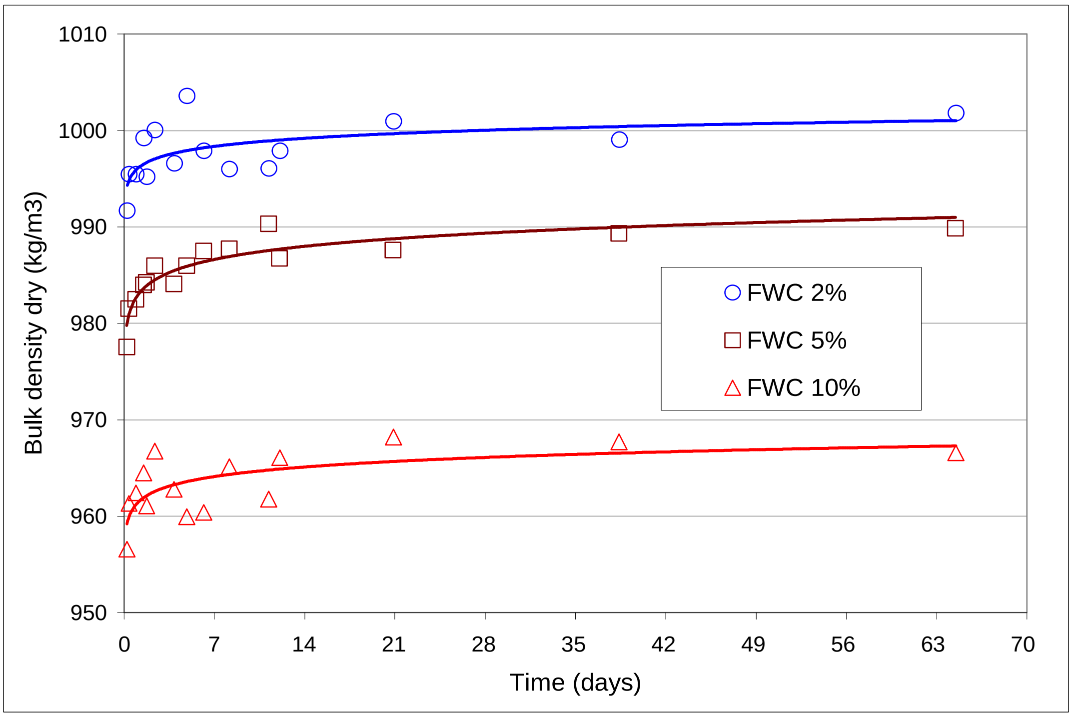


Figure 4.42 - Bulk density test of collapsed foam with different foaming water contents

Table 4.7 - Prediction of void content of collapsed foamed bitumen

FWC, %	0 (hot bitumen)	2	5	10
Density, kg/m3	1030	1000	990	968
Volume at mass of 1000kg, m3	0.9709	1.0000	1.0101	1.0331
Void content (relative to original bitumen), %	0	2.997	4.038	6.406

4.10 Discussion and Conclusions

An understanding of foamed bitumen characteristics is very demanding since the structure of foam-aggregate mixture is not fully defined. The way that foam works in the mixture at every step of construction, namely the mixing process, storage period, compaction, curing period and service live, remains poorly understood. In this study, foamed bitumen properties have been explored through a combination of theoretical and experimental work as presented in previous sections.

It is important to confirm that foamed bitumen can be included within the general foam family. Foamed bitumen composition, with both wet and dry foam formation, gives an indication that foamed bitumen behaviour complies with expected general foam properties. There are therefore some general properties that are valid for general foam and which are also valid for characterising foamed bitumen. This includes the categories of 'foam quality', for which wet foam ranges between 52% and 87%, stable dry foam between 87% and 96%, and beyond 96% the dry foam becomes unstable.

Foamed bitumen can only be successfully formed by the presence of a surfactant which is primarily contained in asphaltenes (see Barinov, 1990). Low penetration bitumen normally has a high asphaltene content and will therefore produce a better quality foam. However, foam is a complex material. Its properties are not only affected by the surfactant present on the thin lamella (bubble walls), but are also affected by many aspects such as gas content, structure, modulus, viscosity, surface tension and drainage. Thus, the optimum foam properties will be a balance of all those factors and it seems a moderate binder grade will be more suitable for foamed bitumen.

Foamed bitumen characteristics are strongly predicted to have a close link with foam temperature since foam is generated by a heat transfer process. In this respect, foamed bitumen is a unique soft material. Heat energy is needed by the water to develop steam gas. On the other hand, high temperature causes the bitumen viscosity and surface tension to decrease, which initiates bubble collapse. Both viscosity and surface tension have complex effects and they are interrelated. A bitumen with low viscosity gives the opportunity for the steam gas to create bubbles and hence it enhances foam quality. The surface tension, which is strongly dependent upon viscosity, is not only primarily required by thin lamella to balance the internal pressure of an explosive bubble, but also required to balance Plateau border suction, in order to resist liquid drainage. All these complex aspects of the behaviour of foamed bitumen are likely to be linked with its temperature, as a result of the heat transfer process. Thus, an essential balance is required in order to generate foamed bitumen with optimum properties. It is therefore important to investigate these essential properties through manufacture and testing foamed bitumen samples to confirm the complex characteristics.

The effect of foaming water content (FWC) on foamed bitumen characteristics in terms of maximum expansion ratio (ERm) and half-life (HL) has been identified. It is clearly evident that the value of ERm increases with increasing FWC, however the HL value follows an opposite trend to ERm. These characteristics agree with general foam properties, in which dry foam (high ERm) tends to be more unstable than wet foam (low ERm). The constant values of HL at high FWC are thought to relate to the effect of foam temperature. More added water in the foaming process results in a lower resultant foam temperature. This low temperature will cause foam bubbles to collapse more slowly. This effect may balance the gas content effect and hence this results in constant HL values at high FWC.

Bitumen temperature is also noted as a factor affecting ERm and HL, although not as significant as FWC. It is well known that temperature affects hot bitumen viscosity significantly. Since binder type (penetration grade) also influences viscosity, it can be understood that bitumen temperature and binder type therefore represent a combined factor in affecting the foaming process. In general, for FB 160/220, higher bitumen temperature produces lower ERm, whereas for FB 50/70, this trend is reversed, while FB 70/100 performed inconsistently with temperature. On the other hand, the HL generally tends to decrease with increasing temperature for all foams except for FB 70/100. So, according to the ERm value, an optimum bitumen viscosity is required to generate the highest ERm. In this case, a moderate bitumen grade appears more suitable than an extreme soft or hard bitumen. However, considering the HL value, it appears that a bitumen with low pen and temperature will be more suitable.

The foam decay curve is a fundamentally meaningful piece of data. It represents the accumulation of all bubbles that exist during the foam life. Jenkins (1999) has defined the area under the foam decay curve, namely the FI concept. In this study, it was found that the effect of FWC on FI values is mainly related to ERm, and therefore their values over various FWC are roughly proportional.

Flow of foamed bitumen through orifice(s) at the bottom of a container has been studied using FB 70/100. Although this test does not use any standard procedure, the results can be used to evaluate foamed bitumen with various properties in a situation in which the bitumen liquid is allowed to drain. It is noted that the interpretation of foam flow might be dependent upon the size of orifice, and that foam flow through a 27mm orifice will be more likely to represent bulk foam flow, whereas foam flow through a 3mm orifice might be more representative of foam drainage. The results can be used to indicate (1) flow characteristics, (2) effect of FWC and bitumen temperature on foam flow behaviour, and (3) prediction of the life-time of the foam.

Three regimes of flow behaviour have been identified, i.e. foam state, liquid state and viscous state. These states indicate the change of foam consistency during its life and this may be useful guidance in deciding when foam properties should be measured.

The effects of foaming water content (FWC) and bitumen temperature on flow behaviour can be clearly observed, namely that the rate of foam flow through orifice(s) tends to decrease with increasing FWC and decreasing bitumen temperature. In agreement with Kraynik (1988) and also Marsden and Khan's finding (Heller and Kuntamukkula, 1987), foam flow rate can be linked to its viscosity and hence it can be deduced that a foam with a higher FWC tends to have a higher apparent viscosity. In the case of decreasing bitumen temperature, which resulted in a reduction of foam flow rate, this can be understood since a reduction in foam temperature enhances foam viscosity. In this study, the effective foam viscosity was also estimated using the Kraynik equation. It was found that the viscosity was dependent upon gas content and bitumen viscosity, in which foam temperature plays a noticeably important role. An important point is that, at an ERm of around 25 (for FB 50/70 at 180^{o}C), foam viscosity can be seen to reach a critical point at which the viscosity value changes dramatically. The term 'kinematic viscosity', ratio between viscosity and density is likely more suitable to represent the resistance of foam flow since foam viscosity is dependent on its density, for which the viscosity increases at lower density (gas content increases). While the term 'effective or apparent viscosity' is used to describe a rheology that is affected by the presence of compressible gas bubbles.

The life-time (FL) of foamed bitumen can also be estimated using the flow rate test results. The trend of FWC and the bitumen temperature effect on FL is found to be different from the trend of half-life (HL) for FB 70/100. It is found that the FL tends to increase at lower bitumen temperature or at higher FWC. It is noted that for FB 50/70 and FB 160/220, the HL of foam is also found to increase at lower temperature. The trend of FL values that increases sharply at FWCs higher than 2% is supposed to be the effect of the drainage process. At high FWC, the drainage tends to be more slowly and hence foam remains wet which causes foam being more stable. This indicates that the life of the foam may be a useful quantity to be considered in order to characterise foam stability, in addition to the HL.

Investigating the collapsed state of foamed bitumen is also interesting and challenging. The fact that the loose foamed asphalt mixture can be stored for months gives an indication that the collapsed foam does not completely return to its original bitumen. This fact has been confirmed by the bulk density test results, for which the density is found to be lower at a high FWC. It is supposed that large number of tiny bubbles and water droplets are still trapped inside the collapsed foamed bitumen. However, these bubbles cause problems when the bitumen is tested for penetration, so that the results are not valid. From the RTFOT, the penetration values of collapsed foam after ageing and removal of the bubbles are comparable to the original bitumen, indicating that no significant ageing occurs during the foaming process.

In conclusion, some important aspects of foamed bitumen characterisation have been pointed out. The theoretical and experimental works have been combined comprehensively in order to explore the effect of FWC (or ERm/ gas content) and bitumen temperature at different bitumen grades (or viscosities) on the overall foam properties. It is expected that these will all prove to be important keys in unlocking an understanding of foamed asphalt mixtures.

5 INVESTIGATING FOAMED ASPHALT PERFORMANCE

5.1 Introduction

This chapter discusses foamed asphalt performance with emphasise on the effect of foamed bitumen properties. The mixture performance was investigated in terms of stiffness (using Indirect Tensile Stiffness Modulus test), resistance to permanent deformation (using Repeated Load Axial Test) and fatigue performance (using Indirect Tensile Fatigue Test). The theory of testing in the indirect tensile mode will firstly be described in this chapter. The general procedure to prepare the specimens will be explained prior to discussing test results. The specimens were produced using different foamed bitumen properties, varied according to binder type, applied FWC and bitumen temperature. Additionally, workability of foamed asphalt in the mixing and compaction process will be also studied. The main aims of this investigation are to improve understanding of foamed asphalt properties and moreover to explore the principal effects of foam properties on the mixture performance.

5.2 Theory and Testing Description

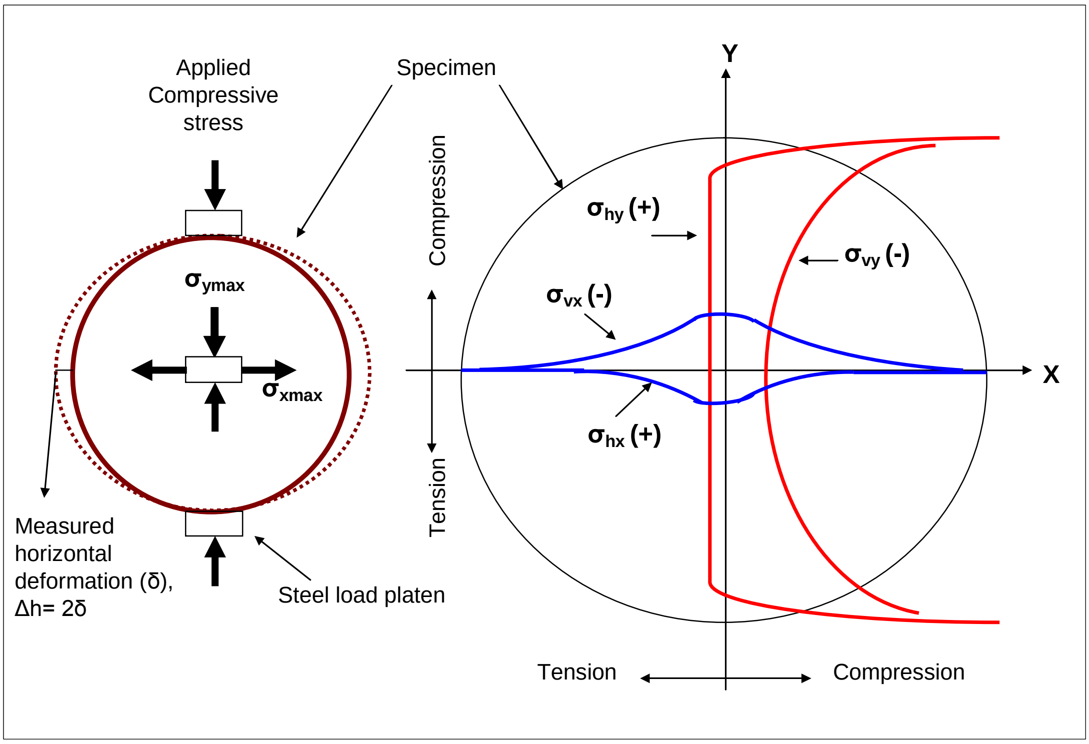
5.2.1 Theory of indirect tensile mode

In the indirect tensile mode, the (repeated) compression load is applied across the vertical diameter of a cylindrical specimen and it results a biaxial stress distribution in the specimen as shown in Figure 5.1. It can be seen that both a vertical compressive stress (σ_{vx}) and a horizontal tensile stress (σ_{hx}) are induced on the horizontal diameter of the specimen. The magnitudes of the stresses vary along the diameter and they reach a maximum value at the centre of the specimen. By measuring the horizontal deformation (Δh), the maximum strain at the centre of the specimen and hence the stiffness modulus can be calculated. This calculation depends on a number of assumptions as follows:

- The specimen is subjected to plane stress conditions ($\sigma_z = 0$).
- The material is linear elastic.

- The material behaves in a homogeneous and isotropic manner.
- Poisson's ratio (v) for the material is known.
- The vertical load (P) is applied as a line loading.

When the above assumptions are met, then the stress conditions in the specimen agree with the theory of elasticity. This theory shows that when the width of the loading strip is less than or equal to 10% of the diameter of the specimen and the distance of the element of material from the centre is very small then Eq. 5.1 to Eq. 5.4 can be applied (Read, 1996).



Note:	σ_{vx}	= vertical stress across x-axis (compression)
	σ_{hx}	= horisontal stress across x-axis (tension)
	σ_{vy}	= vertical stress across y-axis (compression)
	σ_{hy}	= horisontal stress across y-axis (tension)

Figure 5.1 - An induced biaxial stress distribution under (repeated) compression load in indirect tensile mode

$$\sigma_{hx(\max)} = \frac{2P}{\pi . d . t} \qquad \textbf{Eq. 5.1}$$

$$\sigma_{vx(\max)} = \frac{-6P}{\pi . d . t} \qquad \textbf{Eq. 5.2}$$

The average of horizontal and vertical stress can then be expressed as follows:

$$\overline{\sigma_{hx}} = \frac{0.273P}{d.t} \qquad \textbf{Eq. 5.3}$$

$$\overline{\sigma_{vx}} = \frac{-P}{d.t} \qquad \textbf{Eq. 5.4}$$

where:

d = specimen diameter

t = specimen thickness

P = applied compression load

$\sigma_{hx\,(max)}$ = maximum horizontal tensile stress at the centre of the specimen

$\sigma_{vx\,(max)}$ = maximum vertical compressive stress at the centre of the specimen

$\overline{\sigma_{hx}}$ = average horizontal tensile stress

$\overline{\sigma_{vx}}$ = average vertical compressive stress

Considering the average principal stresses in a small element subjected to biaxial stress conditions, the horizontal tensile strain would be:

$$\overline{\varepsilon_{hx}} = \frac{\overline{\sigma_{hx}}}{S_m} - v\frac{\overline{\sigma_{vx}}}{S_m} \qquad \textbf{Eq. 5.5}$$

$$\overline{\varepsilon_{hx}} = \frac{0.273P}{S_m.d.t} + \frac{vP}{S_m.d.t} \qquad \textbf{Eq. 5.6}$$

Horizontal deformation can be calculated from:

$$\Delta h = \overline{\varepsilon_{hx}}.d \qquad \textbf{Eq. 5.7}$$

$$\Delta h = \frac{0.273P}{S_m.t} + \frac{vP}{S_m.t} \qquad \textbf{Eq. 5.8}$$

The stiffness modulus of the material therefore can be calculated from:

$$S_m = \frac{P(0.273 + v)}{\Delta h.t} \qquad \textbf{Eq. 5.9}$$

where:

$\overline{\varepsilon_{hx}}$ = average horizontal tensile strain

v = Poisson's ratio (assumed)

S_m = Stiffness modulus

Δh = horizontal deformation (measured)

For analysis of indirect tensile fatigue, the strain is calculated at the centre of the specimen so that Eq. 5.1 and Eq. 5.2 can be used. Therefore Eq. 5.5 can be expressed in the form of:

$$\varepsilon_{hx(\max)} = \frac{\sigma_{hx(\max)}}{S_m} - v\frac{\sigma_{vx(\max)}}{S_m} \qquad \textbf{Eq. 5.10}$$

Substituting Eq. 5.1 and Eq. 5.2 into Eq. 5.10 gives:

$$\varepsilon_{hx(\max)} = \frac{2P}{S_m.\pi.d.t} + \frac{v6P}{S_m.\pi.d.t} \qquad \textbf{Eq. 5.11}$$

Substituting Eq. 5.1 into Eq. 5.11 gives:

$$\varepsilon_{hx(\max)} = \frac{\sigma_{hx(\max)}.(1+3v)}{S_m} \qquad \textbf{Eq. 5.12}$$

Where:

$\varepsilon_{hx(\max)}$ = maximum horizontal tensile strain at the centre of the specimen

It can be seen in Eq. 5.12 that the stiffness modulus and the maximum tensile stress values are required to calculate the tensile strain in fatigue test.

5.2.2 The Nottingham Asphalt Tester

The Nottingham Asphalt Tester (NAT) is a set of apparatus that comprises four units (Ibrahim, 1998) i.e. (1) a main test frame placed in a temperature control cabinet, (2) an interface unit for the acquisition of data and controlling the test, (3) a pneumatic unit connected with the interface unit and an actuator mounted above the test frame for controlling the applied load, and (4) a computer unit. Figure 5.2 shows a NAT. Using these four main units, the NAT enables the production of a controlled repeated load to a specimen placed in a temperature-controlled cabinet by means of software installed in a computer unit. The computer controls a voltage/ pressure (V/P) converter and operates a solenoid valve via the interface unit. Using software, air pressure is introduced into a reservoir in the pneumatic unit and the solenoid valve is switched to apply a load to a specimen. The load is then measured using a load cell and the V/P converter is adjusted to achieve the required level. The actuator has varying capacities e.g. 5 kN (standard) and 10 kN. These capacities will limit the precision of the equipment to carry out an assessment. For the ITSM test, when using a smaller capacity and the target horizontal deformation is not being achieved, a large size would be required (Widyatmoko, 2002).

The NAT can be configured for testing specimens for three different purposes i.e. Indirect Tensile Stiffness Modulus (ITSM) test, Indirect Tensile Fatigue Test (ITFT) and Repeated Load Axial Test (RLAT).

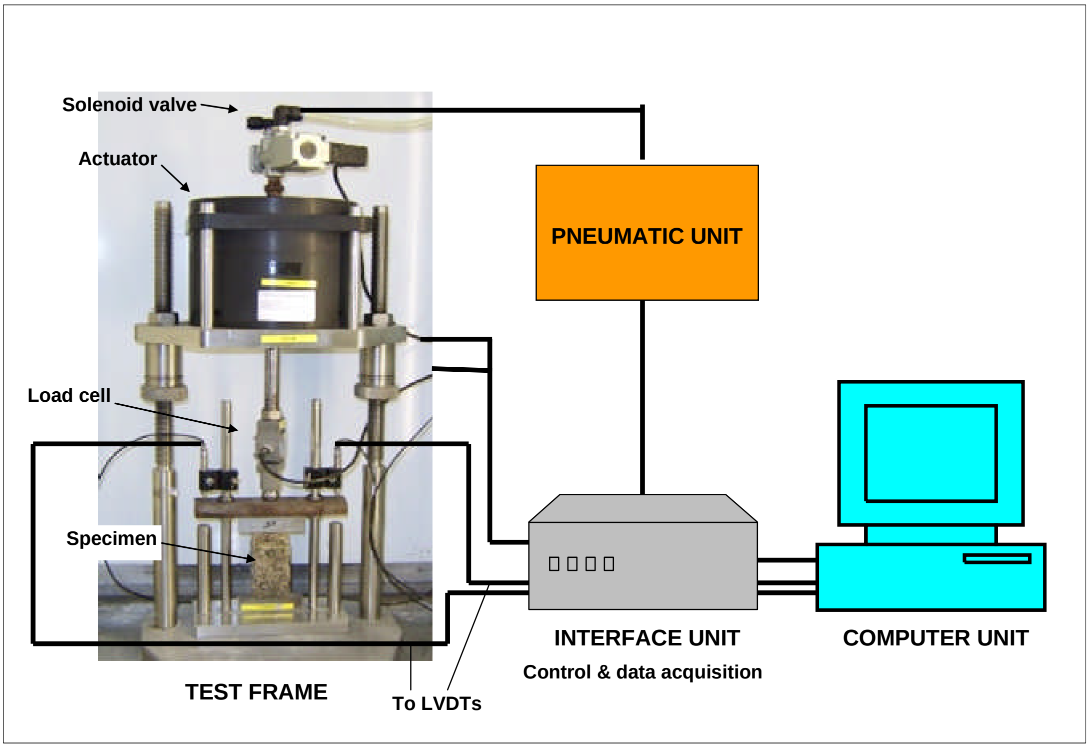


Figure 5.2 - The Nottingham Asphalt Tester configuration for testing bituminous mixtures

5.2.3 Indirect Tensile Stiffness Modulus (ITSM) test

The ITSM test can be carried out in accordance with BS DD 213:1993. This test measures the visco-elastic response of a material. The testing method uses cylindrical specimens either prepared from field cores or laboratory moulded, with a diameter of 100mm or 150mm and a thickness of between 30mm and 80mm. The standard target parameters pertaining throughout testing are as follows:

Test temperature	: 20^oC
Rise time (i.e. time to peak load)	: 124 ± 4 ms
Horizontal deformation	: 5 ± 2 μm (for diameter 100mm)
	: 7 ± 2 μm (for diameter 150mm)

Prior to testing, specimens should be stored in a conditioning cabinet at the test temperature of 20^oC for at least two hours. The stiffness modulus of bituminous mixtures, S_m, can be determined using Eq. 5.9, in which Δh is the mean amplitude of the horizontal deformation obtained from two or more applications of the load pulse and the Poisson's ratio for bituminous mixtures is normally assumed to be 0.35.

The test frame for ITSM determination can be seen in Figure 2.8 (Chapter 2). The specimen is centrally positioned between the lower and upper platens. The deformation measuring devices should be located symmetrically about an axis through the centroid of the specimen and perpendicular to the direction of loading and the axis of symmetry of the specimen.

Input parameters required for testing are specimen temperature, specimen thickness and diameter, Poisson’s ratio, load rise time and target horizontal deformation. The rise time, a time needed by the load to achieve the peak value, is normally 124 ms. This value is generally slower than that associated with moving traffic for which a value of 20 to 30 ms would be more appropriate (depending on the vehicle speed and the depth below the surface), but this would be difficult for the pneumatic system to achieve (Needham, 1996).

The testing normally applies five conditioning load pulses to bed the test specimen onto the loading platens and to enable the equipment to adjust the load to give the specified horizontal diametral deformation. During these conditioning load pulses, the resulting stiffness moduli are displayed on the computer screen.

A further five load pulses are applied to the specimen. For each load pulse application, the resulting stiffness modulus (in MPa), as well as the vertical force (in kN), the horizontal stress (kPa), the load area factor (LAF), the peak horizontal deformation (in microns) and the rise time (in ms), are displayed and their average values are recorded. The resultant horizontal deformation at right angles to the load is measured by two linear variable differential transformers or LVDTs. LAF is the ratio of the area under the load vs loading time curve, integrated from the beginning of the applied load to the rise time (when the peak load is achieved), to the product of the rise time and peak load. A LAF of 0.60 is recommended (BS DD 213:1993).

According to BS DD 213:1993, it is recommended to conduct the test twice for each specimen. In the second test, the specimen is rotated through $90^o \pm 10^o$ about its horizontal axis. The mean value of the stiffness modulus from this test should be within 10% of the mean value recorded for the first test. The mean value of the two tests is then recorded as the stiffness modulus of the specimen. If the difference between the two test results is greater than 10%, the test can be repeated once again in a different position. If the difference persists, the mean value for each test is reported individually.

5.2.4 Repeated Load Axial Test (RLAT)

Under the NAT procedure, the RLAT protocol can be seen in BS DD 185: 1994 and BS DD 226: 1996. As with all NAT based tests, the RLAT uses a cylindrical specimen with a diameter of 100mm or 150mm and thickness preferably between 40mm and 100mm. In the test, the specimen is positioned vertically between the upper and lower steel loading platens, which are slightly wider than the specimen. The repeated load is applied axially, while the vertical deformation of the specimen is measured by two LVDTs mounted on the upper loading platen as shown in Figure 2.13 (see Chapter 2). The pulsating load consists of a square wave form with a frequency of 0.5 Hz, i.e. a pulse of one second duration followed by a rest period of one second duration. This simulates the slow moving traffic that leads to the most deformation in a real road.

The input parameters for testing are temperature, specimen thickness and diameter, and stress and number of load pulses including these for the conditioning stage. The standard test uses a vertical stress of 100 kPa at temperature of 30^{o}C for 1800 pulses. Testing is initiated by a stress of 10 kPa for a duration of 10 minutes (the conditioning stage) to ensure that the loading platens are properly seated onto the specimen prior to running the testing. If desired, testing can be continued up to 3600 pulses. During testing, if the specimen deforms more than 8 mm before reaching the specified number of pulses, the test is then terminated. The test output consists of vertical deformations of the specimen plotted against number of load cycles.

5.2.5 Indirect Tensile Fatigue Test (ITFT)

The ITFT in the Nottingham Asphalt Tester (NAT) can be performed in accordance with BS DD ABF: 2000 and BS DD ABF: 2003. The test is carried out at a standard temperature of 20±1^{o}C using 100±3 mm diameter specimens. A thickness of 40±5 mm is recommended. The test is performed at various stress levels at a rate of 40 pulses/minute. Each specimen is repeatedly loaded until it fails by cracking or a vertical deformation of 9 mm has been achieved. It is recommended to apply the first stress level at 600 or 500 kPa (BS DD ABF: 2003). It may be that a lower stress level should be selected for weak specimens. If the number of cycles N is less than 200 for first test, the stress level can then be reduced for the next test. The target test stress levels should be selected to give as wide a range of lives as possible.

Configuration of the test frame and specimen position in ITFT mode is similar to that in ITSM mode, but without a mounting frame and an alignment jig for LVDT instalment. The LVDT is mounted on the upper loading platen to measure the vertical deformation during the test. In the latest version of the ITFT frame, the vertical deformation is measured directly on the upper loading platen. The cylindrical specimen is positioned in between the loading platens. A number of specimens from each mixture are tested at a range of applied loads resulting in different target stress levels at the centre of each specimen. Therefore the results can be plotted as the maximum stress against the number of cycles to failure (N_f) using logarithmic scales. Linear regression analysis is used to describe the resultant fatigue relationship.

As described in Chapter 2, strain is the principal fatigue criterion for cracking; the relationship between strain and number of cycles is therefore required. The maximum tensile strain $\varepsilon_{hx\ (max)}$ at the centre of specimen, for which the initial strain value at the beginning of test is used, can be determined using Eq. 5.12. It can be seen that the tensile stress $\sigma_{hx\ (max)}$ and the stiffness modulus S_m are required to calculate the tensile strain. The stiffness of the specimen can be pre-determined using an ITSM test at the stress state to be used in the ITFT. This can be performed either using stress control or horizontal deformation control. If the test is conducted using stress control, various stress levels can be applied; if not, the measurements can be taken at different target horizontal deformations (e.g. up to 20 μm) and then the corresponding stress is recorded. However, as Needham (1996) reports, for cold-mix materials that exhibit relatively low strength, applying high stress in an ITSM test can lead to specimen damage. Therefore, it might be impossible to define the stiffness modulus of a specimen at a selected stress with any degree of accuracy.

In this study, the procedure used for establishing fatigue characteristics is as follows:

- Prepare cylindrical specimens (at least 10 samples),
- Determine ITSM value for each specimen using either stress mode or horizontal deformation target,
- Determine the maximum horizontal stress for each specimen using Eq. 5.1 (for horizontal deformation target),
- Calculate the initial maximum horizontal strain for each specimen using Eq.5.12,
- Conduct ITF test at stress level determined from the ITSM test
- Establishing
  - N/vertical deformation vs N plot
  - Stress vs N plot
  - Strain vs N plot

5.3 Specimen Preparation

5.3.1 Materials

The aggregate used in this study was virgin crushed limestone (VCL) collected from Tarmac Dene Quarry (UK). The characteristics of this aggregate have been specified in Chapter 3 Section 3.2.1. Particle gradation was designed to be within the ideal grading envelope for foamed asphalt as recommended by Akeroyd and Hicks (1988). The maximum aggregate size was 20 mm with 51.20% fines (< 6 mm) and 8.60% filler. This aggregate has a low Plasticity Index (PI) i.e. 2.7%. The maximum dry density (MDD) and optimum moisture content (OMC) were found to be 2.242 Mg/m3 and 6.4% respectively, determined in accordance with BS EN 13286-2: 2004 (modified Proctor).

Three bitumen grades were used in this study i.e. Pen 50/70, Pen 70/100 and Pen 160/220. Their properties have been described in Chapter 3 (Section 3.2.3) and Chapter 4 (Section 4.2). Foamed bitumen was generated using a laboratory mobile foaming plant type Wirtgen WLB 10 in which the three bitumen grades were foamed at a water pressure of 6 bars and an air pressure of 5 bars. The characteristics of foamed bitumen were varied by applying different foaming water contents (FWC) and temperatures.

5.3.2 Procedure to prepare specimens

Specimen preparation has been discussed in Chapter 3 (Section 3.3.4). It seems the mixing technique, the compaction method and the curing process hold an important key. These three aspects will only combine to produce an optimum performance mixture when aggregate gradation, water content and foamed bitumen characteristics are designed correctly. Table 5.1 describes the procedure and calculations for preparing the specimens used in this study.

Table 5.1a - Specimen preparation (Materials and mixing preparation)

A. Materials preparation		
Crushed limestone aggregate	Composition: 20mm = 25% 6mm = 8% 14mm = 12% Dust = 39% 10mm = 13% Filler = 3%	Compaction data: MDD = 2242 kg/m3 OMC = 6.4%
Bitumen	Select bitumen grade	
Foamed bitumen	• Foamed bitumen content (excluding water) = 4% of the total aggregate mass • Select bitumen temperature. • Select foaming water content (FWC).	
Batching	• One batch is prepared for 3-4 samples @ 1200g per sample. • For specimens mixed using dough hook agitator, all aggregate fractions are directly mixed with foamed bitumen. • For specimens mixed using flat agitator, only aggregates with maximum size of 10 mm are mixed with foamed bitumen and then the aggregates size of 14 and 20 mm are added before compaction.	
B. Mixing preparation		
Setting for bitumen spraying	• Mass of aggregate for a batch = 4800g (an example for 4 samples). • Mass of foamed bitumen = 4% × 4800g = 192g. • Bitumen used = 1.25 × 192 = 240g (an example using flat agitator). • Setting timer = 240/100 = 2.40 seconds.	
Adding water to the aggregates	• Optimum Moisture Content (OMC) = 6.4%. • Moisture content used = 72% OMC= 72% * 6.4% = 4.6%. • Moisture used = 4.6% × 4800g = 221 g. • When batching using flat agitator, 180g of water is added onto 10 mm graded aggregates and further 41g of water is added onto aggregate size of 14 and 20mm before compaction.	
Mixing technique	• Using Hobart mixer with capacity of 20 quarts. • Agitators used are dough hook and flat types. • Water is added onto aggregates in the mixer bowl and then they are mixed for about one minute, while aggregate mixing is in progress, foamed bitumen is sprayed into the bowl and directly mixed with the wet aggregates for another one minute.	
Storage loose foamed materials	Introduce the loose materials into plastic bag, place inside closed tins and store in the cool room (temperature 5^{o}C). Materials are stored for 1-3 days before compaction.	

Table 5.1 b - Specimen preparation (Compaction, curing and water sensitivity)

C. Compaction method	
Moulding	• Prepare a number of 100 mm gyratory moulds. • Prepare wet loose material, mass of one sample= 1200 g. • For materials mixed using flat agitator, 756 g of foamed materials, 312g of 20mm aggregate, 180g of 14mm aggregate and 10.5g of water are mixed together. Pour 1200g of the resulting mixture into a mould for each specimen.
Compaction	Materials are compacted using Gyratory compactor, applying either density or gyration number setting. Force 800 kPa, angle = 2.0º, density = 2300 kg/m3. Force 600 kPa, angle = 1.25º, gyration number = 200.
Specimen size	Diameter 100mm, height 65-67mm
D. Curing process	
Moulded curing	Keep the specimens in the compaction mould for 1 day at ambient temperature. Expose the top of the specimens to allow curing.
Demoulding	Extract the specimens from the mould carefully
Oven curing	Cure the specimens in the oven at 40°C for 3 days. Prior to testing, the cured specimens are conditioned at the required test temperature in an environmental conditioning cabinet.
Wet condition	For those specimens evaluated in wet condition after dry ITSM testing, specimens are soaked in a water bath at 25°C for 24 hours.
E. Water sensitivity	
Water soaking	• After dry ITSM testing, soak the cured specimens in the water bath at 40°C for 68 to 72 hours. • Bring the soaked specimens to the test temperature i.e. at 20°C for at least 2 hours.

Note: the specimens are cured at 40°C for 3 days to simulate approximately 6 months of field curing (Lee & Kim, 2003).

5.3.3 Determine binder content for tested specimens

This study selected an optimum binder content and used it for all experiments. One application of binder content is necessary in order to conduct a comparison between specimens generated using different foamed bitumen properties such as expansion ratio and half-life, bitumen temperature and bitumen grade.

The selected binder content was determined based upon a mixture design process using bitumen Pen 70/100 and 20 mm graded crushed limestone aggregate. Specimens were produced at three different binder contents i.e. at 2%, 3.5% and 5% by mass of aggregates. Five specimens were generated for each binder content. Foamed bitumen was produced at foaming water content of 2% and bitumen temperature of 150°C. All materials were mixed using the dough hook agitator and compacted using the gyratory compactor with an applied force of 600 kPa, an angle of 1.25° and a target density of 2300 kg/m^3. All compacted specimens were cured at 40°C for 3 days before testing. These cured specimens had a dry density of about 2250 kg/m^3 (close to MDD). All specimens were assessed for stiffness using the ITSM test in both dry and wet conditions. After the dry ITSM tests, all specimens were soaked in a water bath at 25°C for 24 hours before the wet ITSM test. Figure 5.3 shows the test results. It can be seen that optimum binder contents are observed for both the dry and wet tests and that the wet ITSM shows 0.5% higher optimum than the dry ITSM. This study selected the higher value of binder content i.e. 4%.

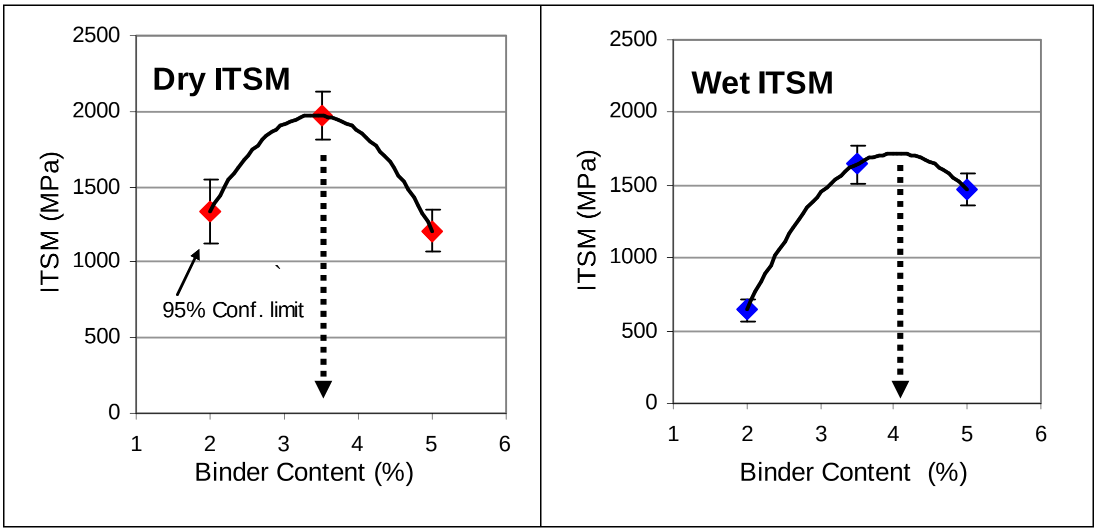


Figure 5.3 - Determining binder content based on the dry and wet ITSM testing

5.4 Compaction characteristics

In this study, all tested specimens were compacted using a gyratory compactor. In this compaction method, the mould is clamped at both ends and rotated on an axis eccentric to the vertical with an angle θ. The compaction angle can be varied e.g. 0°, 1.25°, 2.0° and 2.25°. While the mould is rotated, a static compressive vertical load (range between 600 – 800 kPa) is applied to the material. This generates horizontal shear stresses within the material. The height of the specimen is monitored during compaction using a deformation transducer and the applied load is measured using a load cell. Since the mass of material is known, the specimen density can be recorded during compaction and hence the compactibility of the material can be analysed in terms of the number of gyrations recorded during compaction. The speed of gyration is 30 revolutions per minute. A view of the gyratory compactor can be seen in Figure 5.4.

The compaction characteristics need to be studied since the mixture density affects the mixture stiffness. It could be that foamed bitumen properties affect the mixture performance at all stages during specimen preparation including at compaction stage. It is then interesting to know how large the effect of binder properties on the compactability of a mixture is.

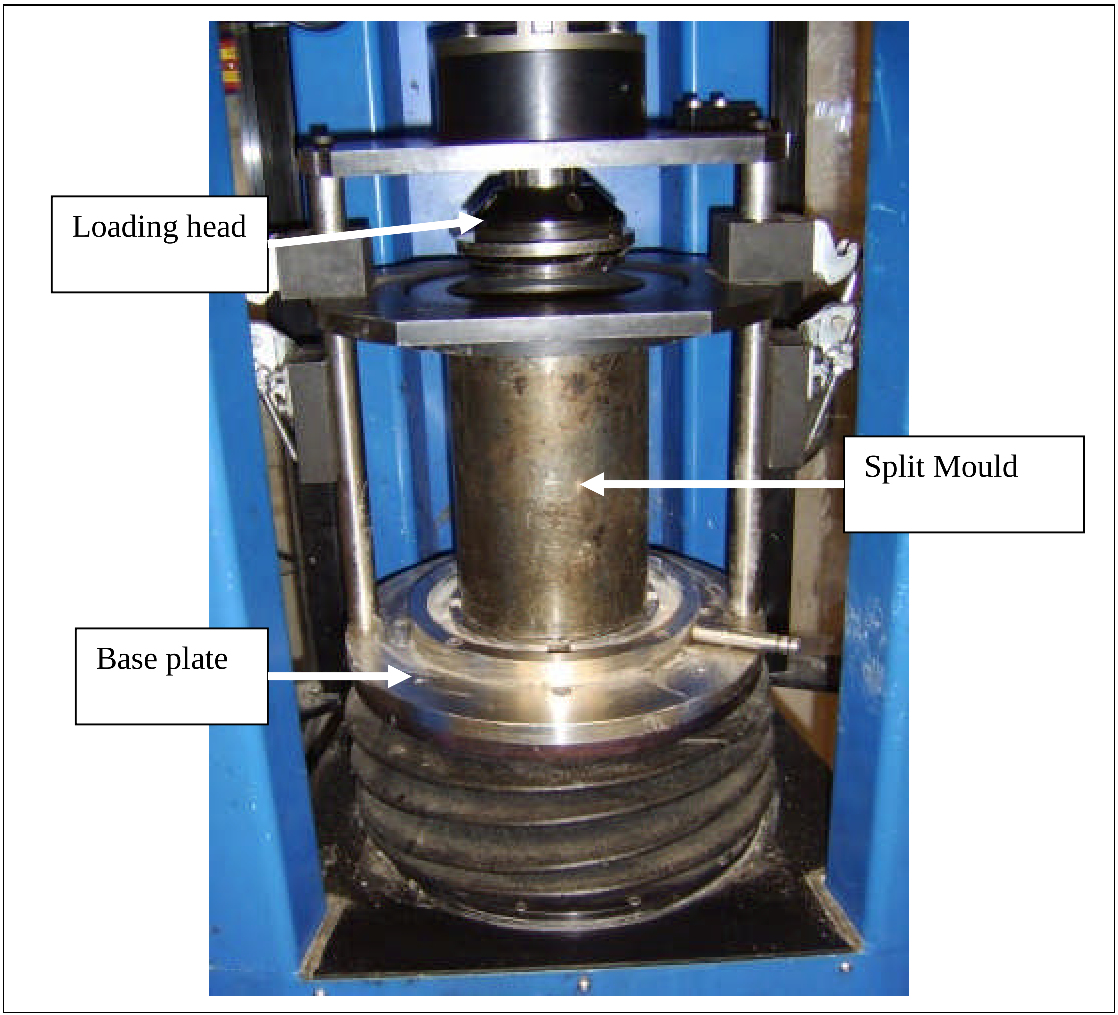
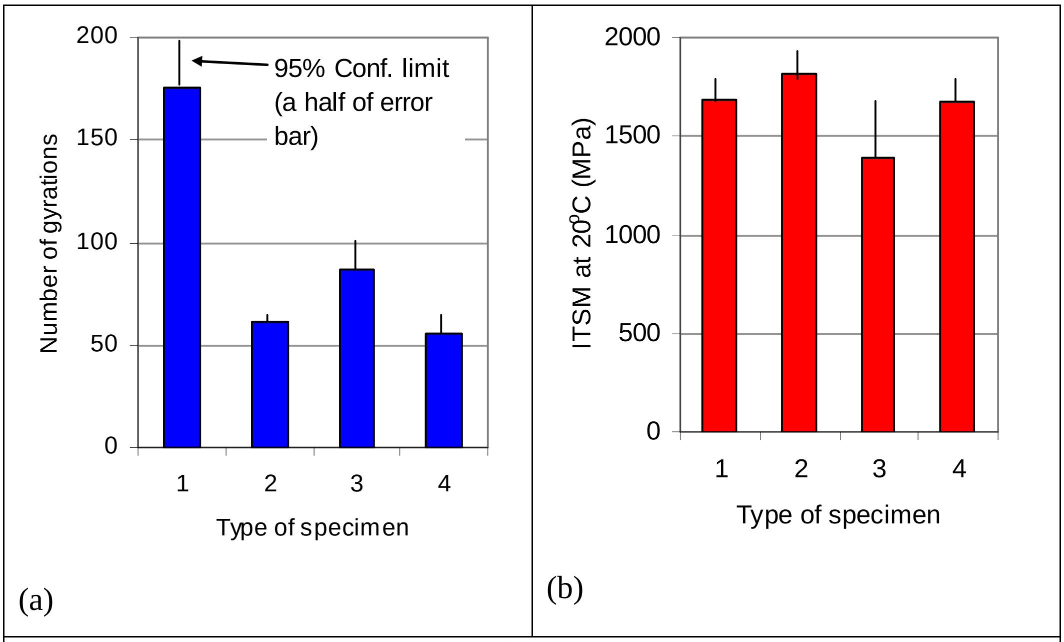


Figure 5.4 - Gyratory compactor

5.4.1 Effect of compaction mode on mixture compactability and stiffness

The effect of compaction angle and pressure on the mixture compactability and stiffness has been investigated. As shown in Figure 5.5, the standard Superpave protocol, which applies a ram pressure of 600 kPa and a compaction angle of 1.25 degrees, was compared with a pressure of 800 kPa and compaction angle of 2.0 degrees. It can be seen that the change of compaction angle from 1.25^{o} to 2.0^{o} at a pressure of 600 kPa or the increase of a pressure from 600 kPa to 800 kPa at an angle of 1.25^{o} gave a significant reduction in the number of gyrations to achieve the target density of 2242 kg/m^3. But this was not the case for the change of compaction angle at 800 kPa. Interestingly, as shown in Figure 5.5b, the different modes of compaction did not significantly affect stiffness values, as long as the final densities were comparable. However, this is not necessarily the case for permanent deformation.



1 – Compacted at 600 kPa, angle 1.25^{o}	3 – Compacted at 800 kPa, angle 1.25^{o}
2 – Compacted at 600 kPa, angle 2.0^{o}	4 – Compacted at 800 kPa, angle 2.0^{o}

Each result is an average of two specimens. Specimens were produced using 20 mm graded limestone aggregates and 4% bitumen (% of aggregate mass). Foamed bitumen was generated at FWC 2% and temperature of 150^{o}C. Materials were mixed using dough hook agitator. All specimens were compacted to obtain a target density of 2242 kg/m^3.

Figure 5.5 - Effect of compaction mode on the mixture stiffness.

5.4.2 Effect of number of gyrations on the mixture density and stiffness

Figure 5.6 shows the effect of number of gyrations on the mixture density and stiffness. The specimens were compacted using a pressure of 600 kPa and angle of 1.25°. The wet density of specimens was recorded from the gyratory compactor, whereas the dry density was determined after the specimens had been cured at 40°C for 3 days. The dry densities are about 65-240 kg/m^3 lower than the wet densities due to moisture loss during the curing process. As shown in the figure, the data reveal that scatter, expressed as the R^2 value of fitting curve (polynomial trend line), is very low, especially for wet density and ITSM data. Based upon these data, it is likely that the optimum number of gyrations will fall in the range 150 to 250. This study selected 200 gyrations or a target wet density of 2300 kg/m^3 for specimen preparation. Additionally, it seems the dry densities and ITSM values are clearly linked (see Appendix A: Figure A.5.1).

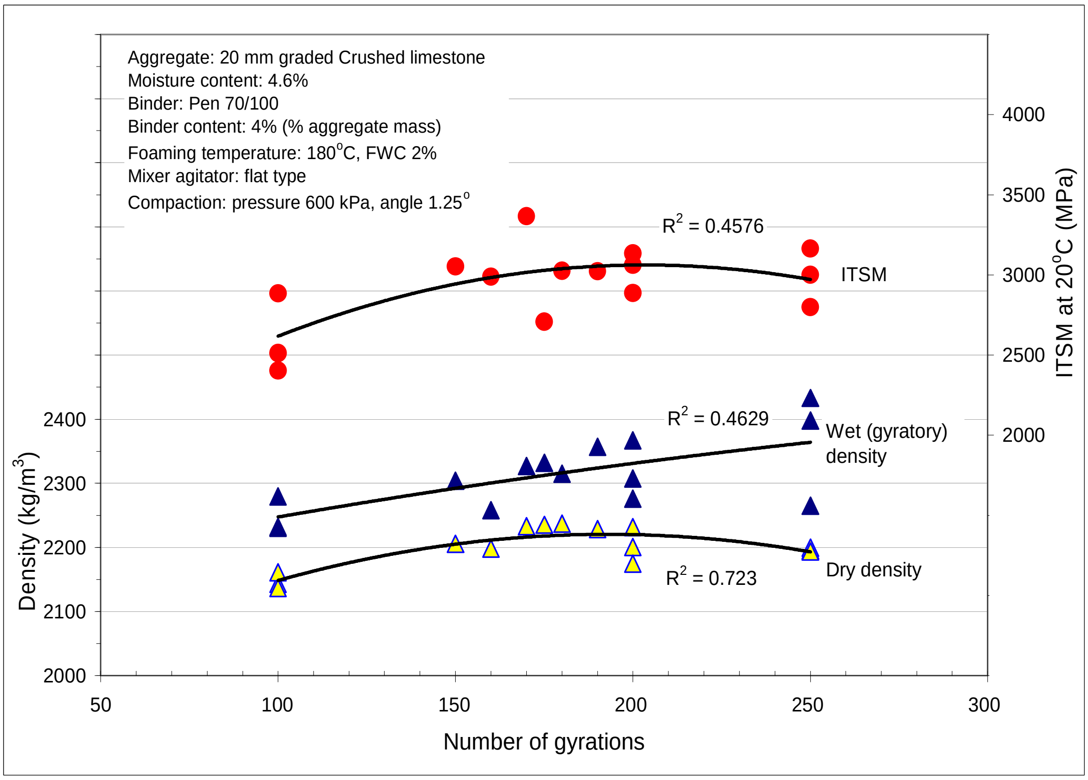


Figure 5.6 - Effect of number of gyrations on the mixture density and stiffness.

5.4.3 Effect of foamed bitumen properties on the mixture compactability

The compactability of a mixture in gyratory compaction can probably be evaluated based upon either the final wet density/ gyration number needed to achieve the target gyration number/ density or the rate of density increase over the second half of the compaction process. Figure 5.7 shows these two methods. The mixture components including aggregate grading, particle arrangement, water and binder, work together to affect the final wet density/ gyration number needed. However the lubricants (binder and water) might be expected to affect mixture compactability over the second half of the compaction process when the particles are relatively stable. The term 'wet density' is used to indicate that the specimens are still in a wet condition before being cured at 40^{o}C for 3 days.

Figure 5.8 shows compactability of the specimens at different FWC values, produced using bitumen Pen 70/100 and mixed using the dough hook agitator. These specimens were compacted using a pressure of 800 kPa at an angle of 2.25^o to achieve a target density of 2300 kg/m^3. The number of gyrations needed to achieve this target density may indicate its compactability. The test data in Figure 5.9 to Figure 5.12 are derived from specimens mixed using a flat agitator and compacted using a pressure of 600 kPa, an angle of 1.25^o and a 200 gyrations. These specimens were generated using three binder types i.e. Pen 50/70, Pen 70/100 and Pen 160/220, and produced at different FWC values (Figure 5.9 and Figure 5.10) or tested at different temperatures (Figure 5.11 and Figure 5.12). These figures show correlation curves between the bitumen grades, the foaming water content (FWC) or the bitumen temperature and the mixture compactability (gyration number/ final wet density or density change over the last 100 gyrations).

The results can be summarised as follows:

- Generally, the softer the bitumen the better the compactability. Interestingly, the compactability of the specimens produced using bitumen Pen 50/70 is far lower than others. The poor density of these specimens may indicate that the bitumen is not only harder but also more poorly distributed in the mixture (bitumen droplets are too large) and hence less workable.

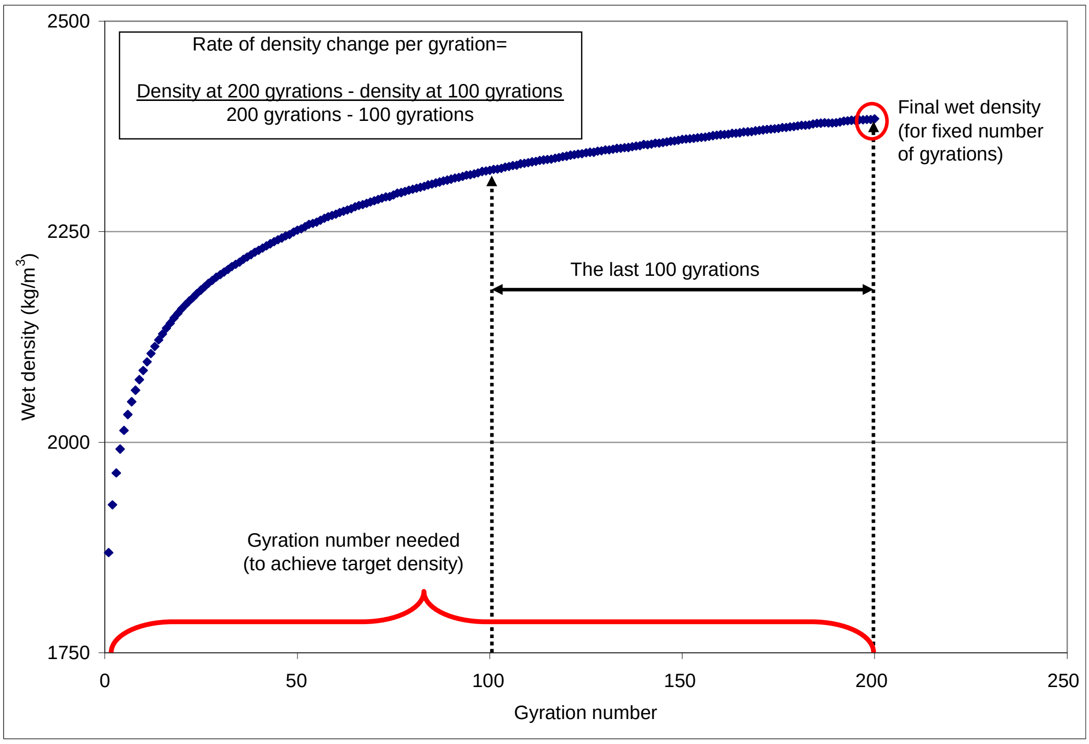


Figure 5.7 - Methods to evaluate a mixture compactability

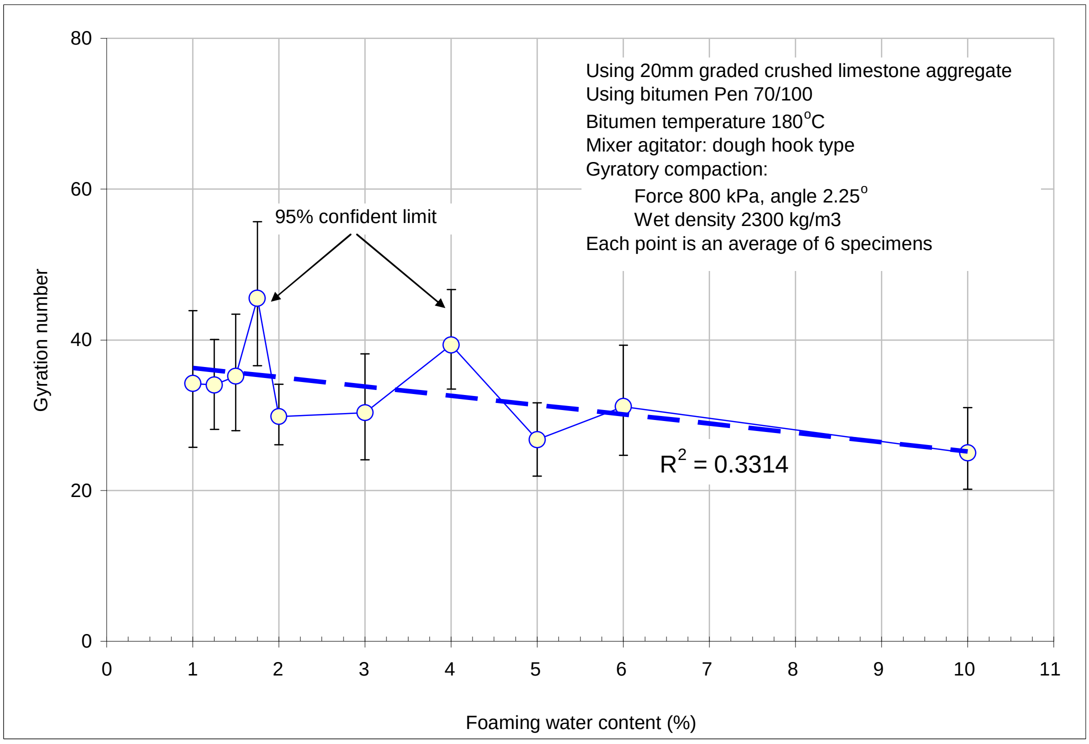


Figure 5.8 - Effect of applied foaming water content on the required number of gyrations.

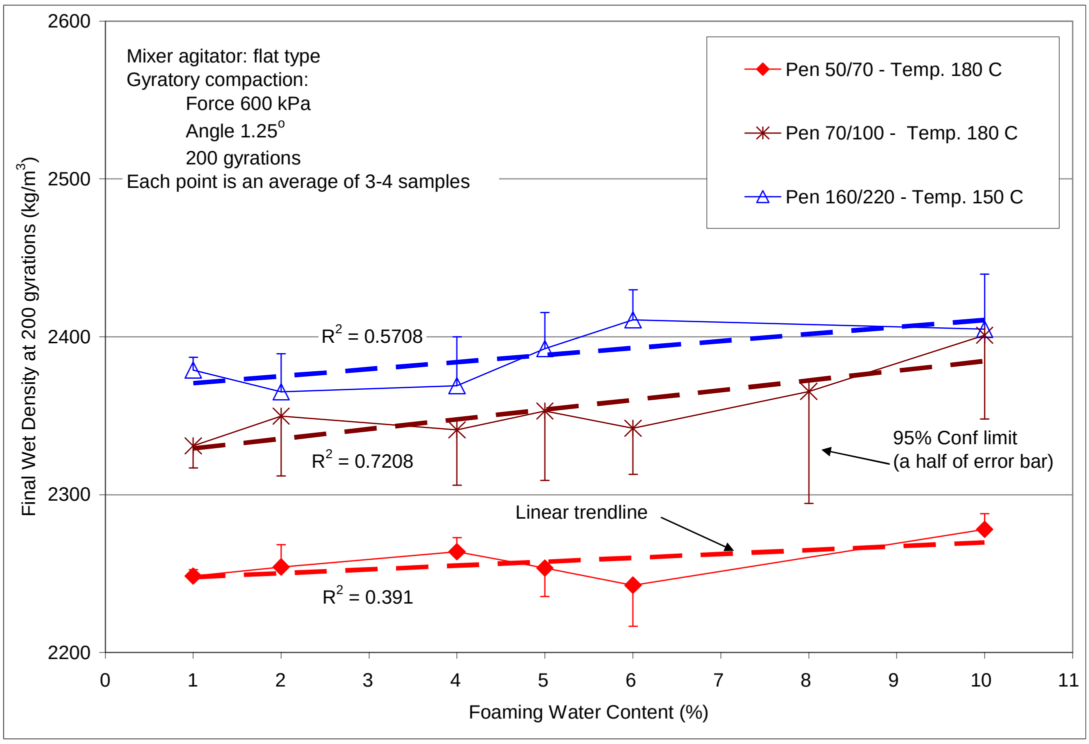


Figure 5.9 - Effect of applied foaming water content on the mixture wet density.

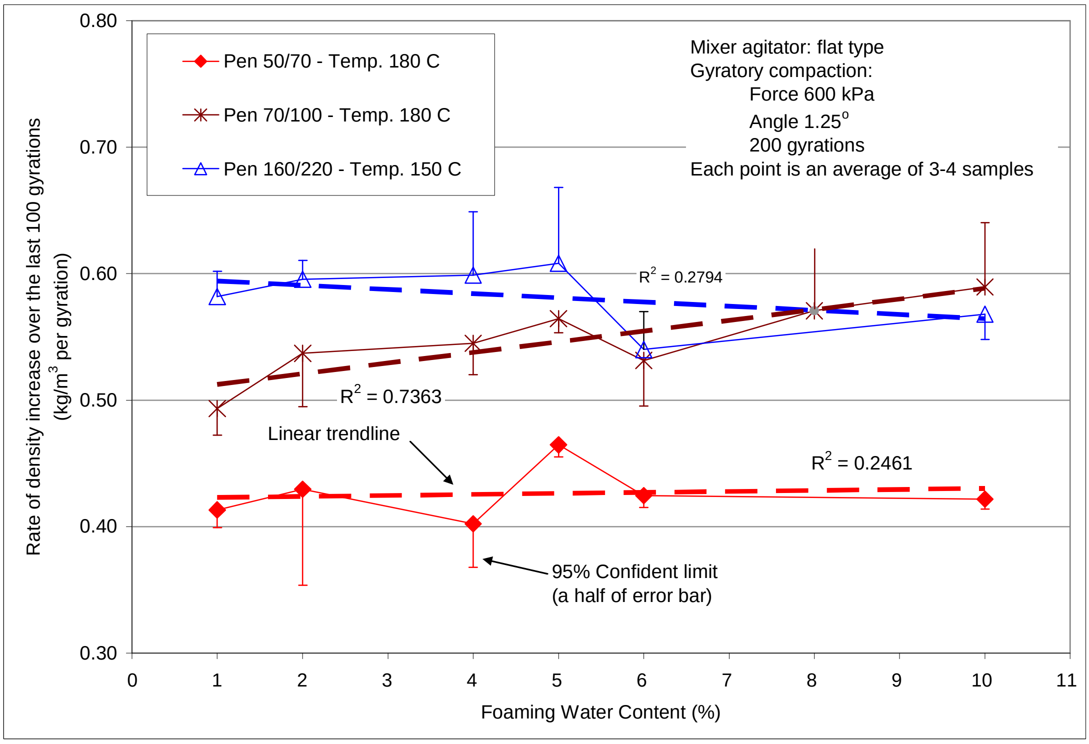


Figure 5.10 - Effect of applied foaming water content on the rate of density increase during compaction process.

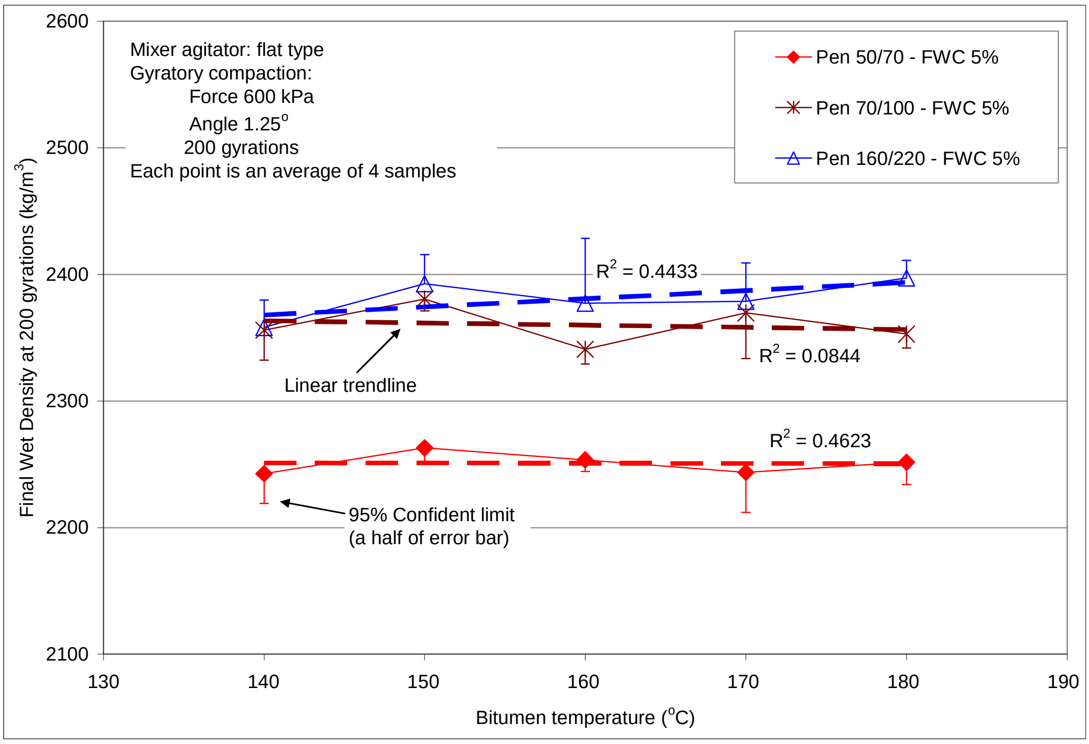


Figure 5.11 - Effect of bitumen temperature on the mixture wet density.

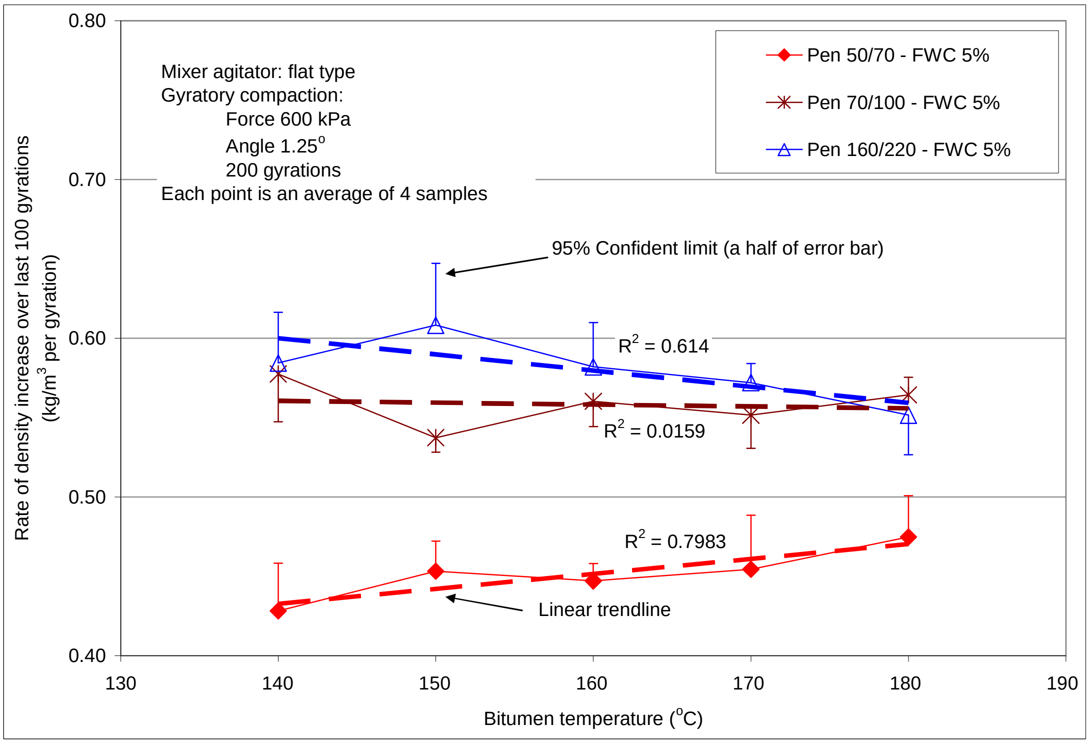


Figure 5.12 - Effect of bitumen temperature on the rate of density increase during compaction process.

- The effect of the FWC on either the gyration number/ final wet density or rate of density change over the last 100 gyrations appears unclear. As shown in Figure 5.9 and Figure 5.10, the data reveals no significant trends and the effect of FWC on the mixture compactability can not be observed. It appears that FWC is not important in its effect on the compactability of a mixture compared to other factors such as the aggregate particles and water since the differences in amount of water in the foam are very small, i.e. about 0.04 (at FWC of 1%) to 0.4% (at FWC of 10%) of the aggregate mass. A linear trend line for each case was also developed in order to analyse the general correlation. It can be observed that, based on the linear trend line, the compactability of specimens increases slightly with the applied FWC. The exception is the trend of specimens using bitumen Pen 160/220 as shown in Figure 5.10, for which the higher the FWC the lower the density rate (less compactability).
- Correlation of bitumen temperature against mixture compactability for all bitumen grades was not observed clearly (see Figure 5.11 and Figure 5.12). However, when it was evaluated based on density rate as shown in Figure 5.12, it appears that for bitumen Pen 50/70 the mixture compactability slightly increases with bitumen temperature, but for bitumen Pen 160/220, the trend is reversed.

5.5 Indirect Tensile Stiffness Modulus (ITSM)

5.5.1 Characteristics of ITSM values for foamed asphalt materials

The effect of applied horizontal deformation and test temperature on the ITSM values of foamed asphalt specimens has been investigated. A comparison to those of hot mix asphalt (HMA) was included. This investigation is required in order to understand the characteristics of mixture stiffness for foamed asphalt materials under the ITSM testing. Comparing the results to the ITSM values of HMA specimens may be useful since HMA's ITSM values have been widely understood.

Figure 5.13 shows the effect of applied horizontal deformation on the ITSM values. An aged specimen was used due to its strength being relatively high and this was expected to reduce the damage during testing. Detailed data from the specimens can be read in the figure. It was noted that increasing horizontal deformation was always accompanied by an increasing horizontal stress value. Therefore the result can be used to assess the stress effect on mixture stiffness. The test was conducted over two days in order to evaluate any healing that occurred after an overnight rest. As shown in the figure, the specimen was tested with an applied horizontal deformation from 2 µm to 25 µm, directly returning to 5 µm. It can be seen that an increasing horizontal deformation significantly affected the stiffness of the specimen. On the first day, the ITSM value decreased about 1800 MPa, which means that the mean stiffness reduction is about 350 MPa for each increase in horizontal deformation of 1 µm. However, in fact, the stiffness reductions decreased for higher deformations. On the second day, the rate of stiffness reduction was only about 100 MPa per 1 µm. It was observed that some healing occurred since there was no significant reduction of ITSM value from 7 µm to 9 µm after an over night storage. It was clear that after the series of tests with deformation up to 25 µm, the specimen was damaged since when it was re-tested with a deformation of 5 µm, its ITSM value was far lower than previously. Based upon this investigation, it can be stated that the applied stress significantly affects the stiffness of a foamed asphalt specimen and high stress potentially damages the specimen. This can be understood since foamed asphalt is not a fully bonded material since the binder does not coat all aggregate surfaces. This type of structure will tend to have a stress dependent behaviour and the uncoated surfaces in the mixture form the equivalent of cracks, which redistribute stress onto surrounding coated surfaces.

Figure 5.14 presents the effect of applied horizontal deformation on the ITSM values of three different mixture types i.e. a well mixed specimen (material mixed using the flat agitator), a poorly mixed specimen (material mixed using the dough hook agitator) and a HMA specimen. These three specimens contained approximately 4% of bitumen (% by aggregate mass) and the well mixed specimen had slightly higher density than the others. The aged foamed asphalt specimens exhibited higher ITSM values than the fresh specimens (at early age). Detailed data on the specimens can be read in the figure. All specimens were tested with applied horizontal deformations from 2 µm to 13 µm as shown in the figure.

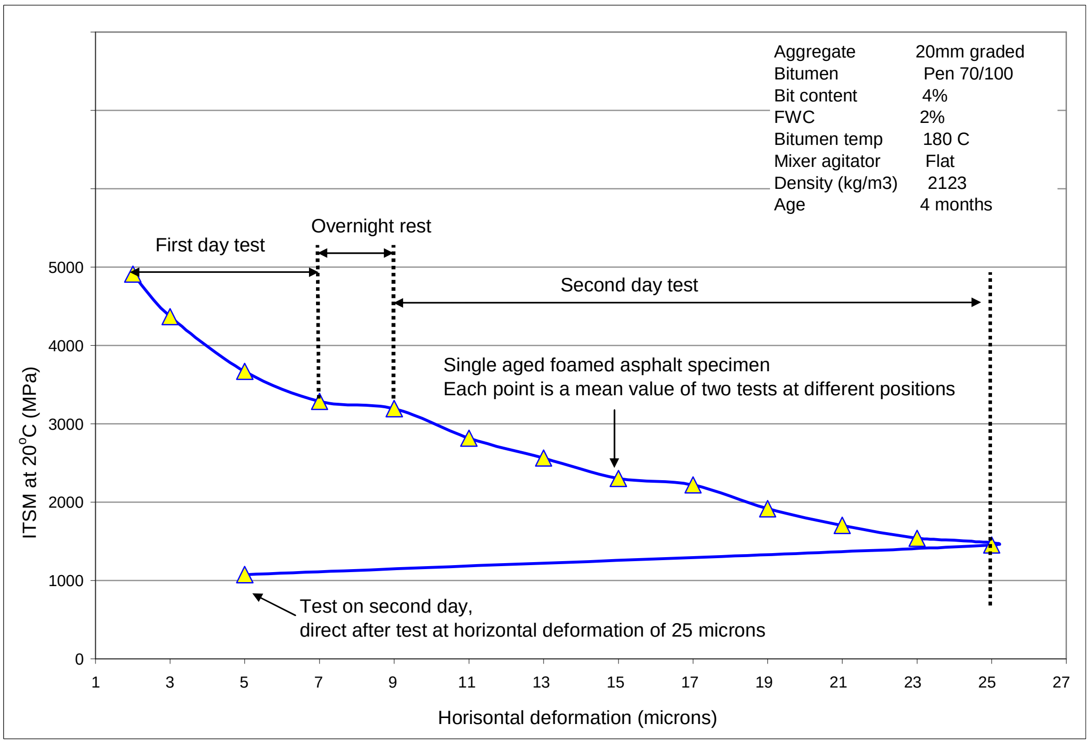


Figure 5.13 - Effect of horizontal deformation on the ITSM value of foamed asphalt specimen

The results show that the rate of ITSM reduction of the well mixed specimen was higher than that of the poorly mixed specimen due to its ITSM value being far higher than the ITSM value of the poorly mixed specimen. However, the rate of ITSM reduction of both these specimens was higher than that of the HMA specimen. It can therefore be deduced that the ITSM value of the foamed asphalt specimens is more sensitive to applied horizontal deformation (stress) than the HMA specimen. The foamed asphalt structure, which is not fully bonded, causes the mixture to be more stress dependent than fully bonded HMA materials.

Figure 5.15 shows the effect of test temperature on stiffness values of aged foamed asphalt and fresh hot mix asphalt specimens. It can be seen that these two specimens have a comparable ITSM value at a test temperature of 20^{o}C. When the specimens were tested at 40^{o}C, their stiffness reduced significantly to values lower than 1000 MPa. However, when they were tested at 5^{o}C, the ITSM value of the HMA specimen was far higher than that of the foamed asphalt specimen. This can be understood since in HMA specimens, which have a thin bitumen film, the binder hardening will be greater than in foamed asphalt specimens which have thick bitumen film. This means that foamed asphalt material is less sensitive to temperature than hot mix asphalt.

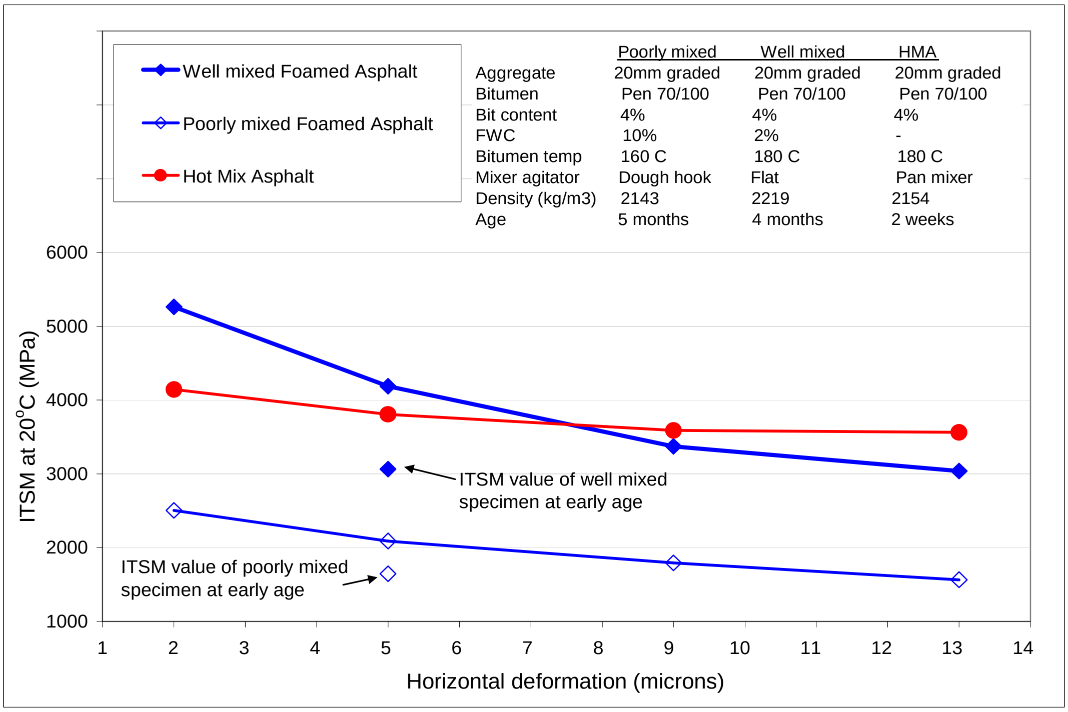


Figure 5.14 - Comparison of ITSM values between well and poorly mixed foamed asphalt specimens and a hot mixed asphalt specimen plotted against horizontal deformation.

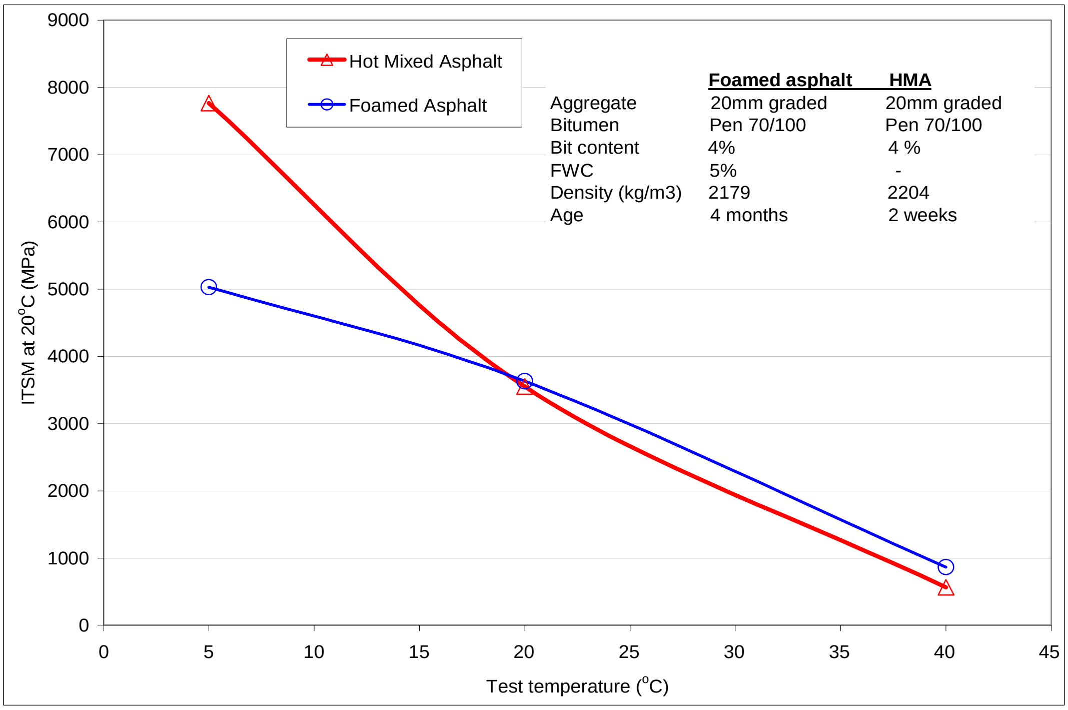


Figure 5.15 - Effect of test temperature on foamed asphalt and hot mix asphalt specimens.

5.5.2 Effect of foaming water content (FWC)

5.5.2.1 Using different mixer agitators

As shown in Figure 5.16, the effect of FWC on mix stiffness was investigated using two types of specimen i.e. dough hook mixed specimens and flat agitator mixed specimens. The dough hook mixed specimens were compacted at a pressure of 800 kPa and angle of 2.25^o to achieve a wet density target of 2300 kg/m^3, whereas the flat agitator mixed specimens were compacted at a pressure of 600 kPa and angle of 1.25^o for 200 gyrations. The dry densities (after curing) of flat agitator mixed specimens were slightly lower than the densities of dough hook mixed specimens. It was found that the mixing method is a very important variable in foamed asphalt performance. The mixing technique controls the efficiency of the expanded bitumen distribution across the aggregate phase.

The results show that specimens mixed using a dough hook exhibited poorer binder distribution (can be seen clearly as shown in Figure 3.14) and hence lower stiffness values. Across the full range of FWC investigated (i.e. up to 10%), the effect of FWC on mix stiffness was not clear. The appearance of minor fluctuations in stiffness values was attributed to small variations in mix moisture content and density values. The linear trend-line for the poorly mixed specimens (dashed line) has a slight negative slope with increasing FWC. However, when specimens were mixed using the flat agitator, their cured ITSM values increased significantly and the effect of FWC was clearly evident. In this case optimum performance was clearly obtained with specimens prepared at 5% foaming water.

As described in Chapter 3 (Section 3.3.4.2), the flat agitator performs better but it causes degradation of aggregates. It was observed that the gradation of specimens mixed using the flat agitator was finer by about 3% at the smallest sieve size and 9% at the largest sieve size. For subsequent mixes, it was therefore decided to mix firstly foamed bitumen and 10mm graded aggregate; aggregate larger than 10mm (i.e. 14mm and 20mm) was added prior to the compaction process. This should not significantly affect the mixture performance since foamed bitumen only coats effectively aggregates with a maximum size of 6.3mm (see Section 6.2.2). This procedure was applied for all remaining investigations in this study.

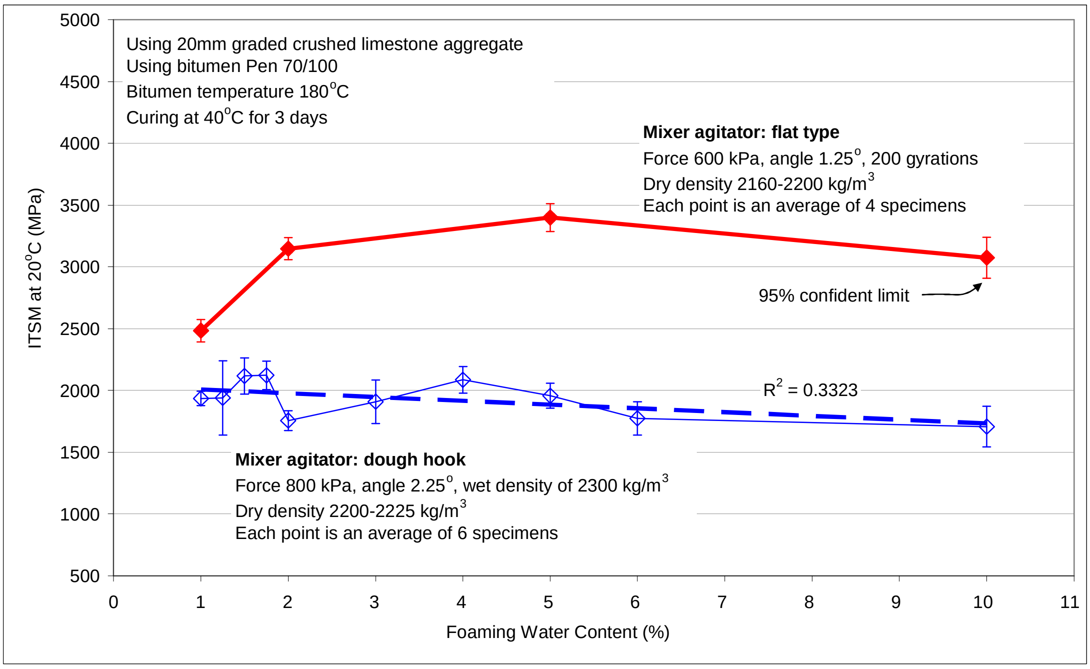


Figure 5.16 - Effect of foaming water content on the ITSM values of specimens mixed with different mixer agitators.

5.5.2.2 Using bitumen Pen 70/100

Figure 5.17 demonstrates the effect of foaming water content (FWC) on the ITSM values for FB 70/100 at 180°C (foamed bitumen produced using bitumen Pen 70/100 at a temperature of 180°C). As recommended previously, the specimens were prepared using the flat agitator, in which only aggregates with maximum size of 10mm were mixed with foamed bitumen and aggregate with size of 14mm and 20mm was added later. It can be seen that the average ITSM values at FWC of 1, 2, 5 and 10% are close to the average ITSM values produced by the well mixed specimens shown in Figure 5.16. This indicates that the revised specimen preparation method (aggregate >10mm added after initial mixing) can be used with confidence. As stated previously, the optimum performance was obtained with specimens prepared at 5% foaming water, although, as indicated by the 95% confident limit bars, the ITSM values at FWC of 2%-4% and of 6%-10% were no different.

The effect of FWC on the ITSM value was also evaluated at different test temperatures as shown in Figure 5.18. The samples used are the well mixed specimens as presented in Figure 5.16. Following curing at 40°C for 3 days and conditioning at the test temperature for at least 2 hours, the specimens were tested firstly at a standard test temperature of 20°C (the results shown in Figure 5.16). The specimens were then conditioned at a temperature of 5°C for at least 2 hours while the test cabinet was re-set to the same temperature i.e. 5°C (this always takes a minimum of 4 hours). The ITSM evaluation at 5°C was normally conducted one day after testing at 20°C. On the same day, the specimens were then tested at 40°C. ITSM tests at different test temperatures were required in order to evaluate effect of the applied FWC on the temperature susceptibility of foamed asphalt materials. As shown in Figure 5.18, at an FWC of 5%, the ITSM values generally change slightly more or are more sensitive to the temperature than at other FWC values. This indicates that at this FWC the bitumen component of the foam is more efficiently dispersed throughout the aggregate skeleton and can hence contribute more to the mixture performance compared to other FWC values.

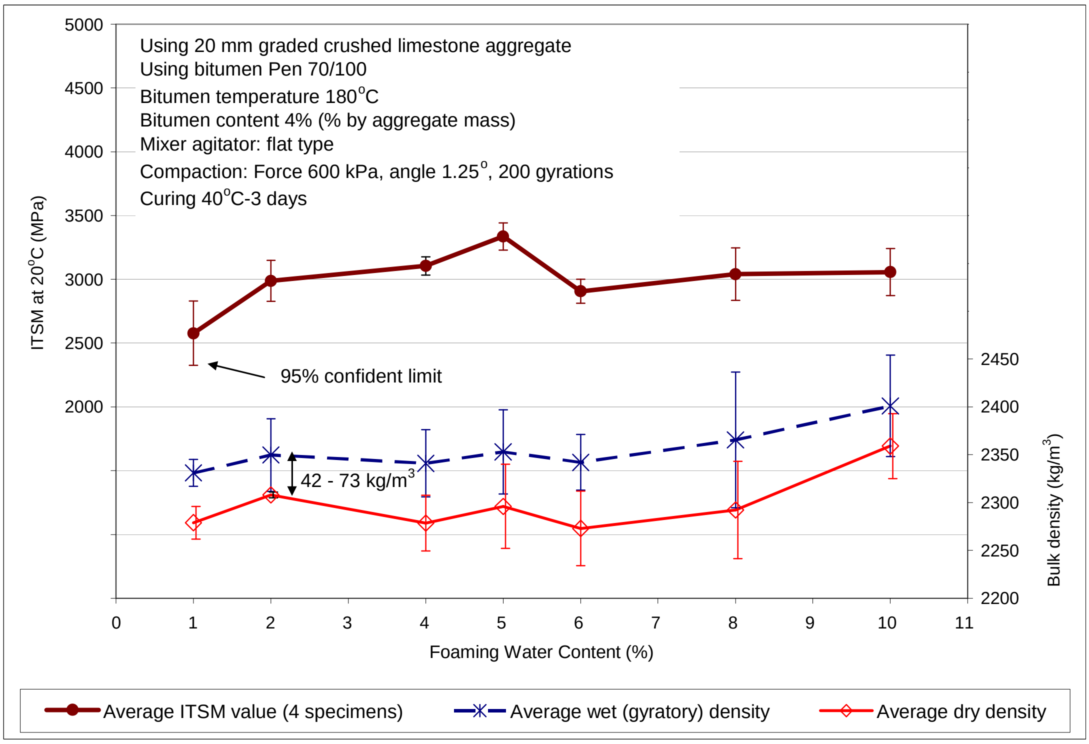


Figure 5.17 - Effect of foaming water content on the ITSM values of well mixed specimens produced using bitumen Pen 70/100.

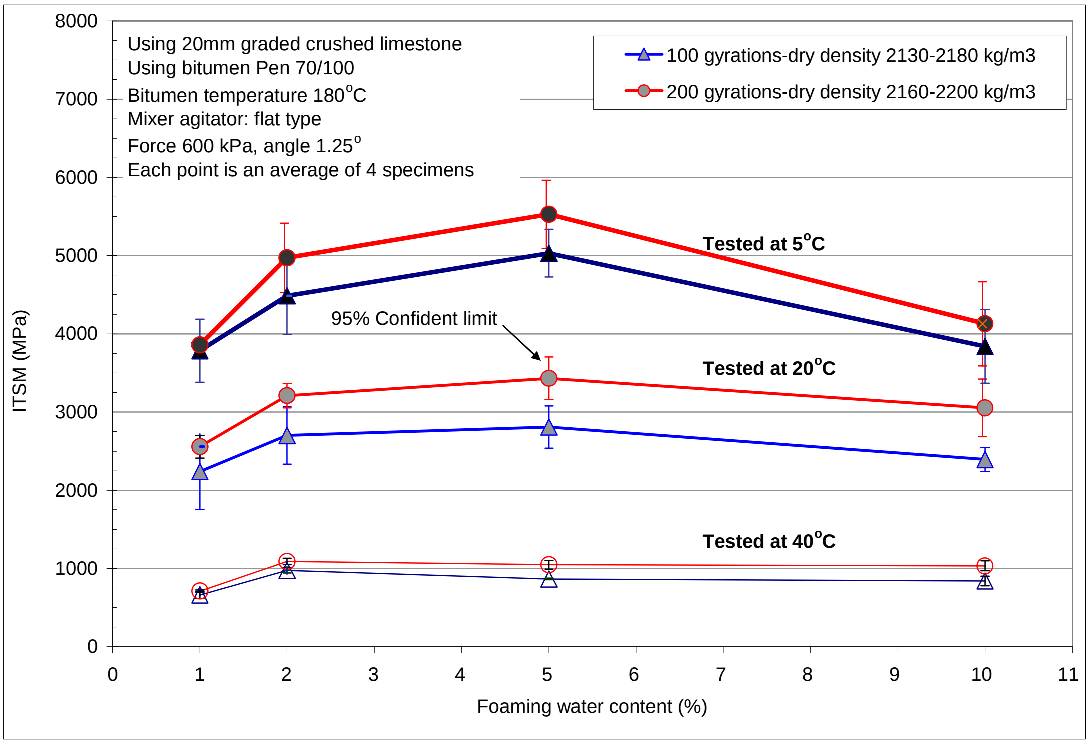


Figure 5.18 - Effect of test temperature on the ITSM values for well mixed specimens produced using bitumen Pen 70/100, compacted at 100 and 200 gyrations (Force 600 kPa and angle 1.25°).

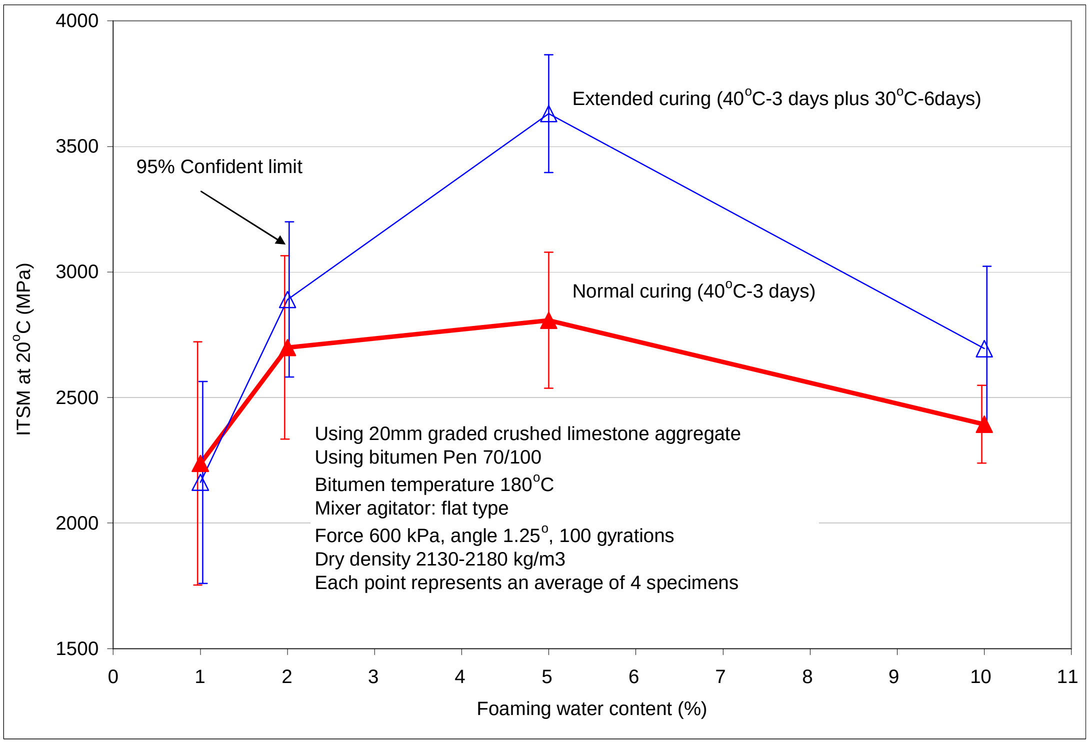


Figure 5.19 - Effect of curing regime on the ITSM values of well mixed specimens produced using bitumen Pen 70/100.

One of the key differences between foamed cold mix asphalts and hot mix asphalts is in the rate at which they achieve their "peak stiffness" (not taking ageing into consideration) following laying and compaction in the field. Unlike hot mix asphalts, foamed asphalts demand a substantial curing time (to allow loss of water from the compacted mix) during which the mixes gradually gain stiffness. Therefore, a typical foamed asphalt laboratory mixture design procedure normally includes an accelerated curing regime. In this study, the specimens were cured at 40^{o}C for 3 days. The curing temperature of 40^{o}C was selected to be lower than the softening point of the bitumen to ensure minimal damage to the specimens and to reduce binder ageing. By carefully weighing the specimens before and after curing, it was found in this study that following 3 days of curing at 40^{o}C, the 100mm diameter specimens were not totally dry, with around 0.2 to 0.4% of moisture remaining trapped within the specimens. It was subsequently decided to subject selected specimens to additional curing at 30^{o}C for a period of 6 days. The temperature of 30^{o}C was considered as normal in hot climatic conditions. Figure 5.19 shows the results of the effects of curing regime on the ITSM values, this time using only 100 gyrations. The results indicate a clearly defined optimum FWC. Interestingly, the extended curing regime seems to increase the ITSM at the optimum FWC significantly more than the modest increases at other FWC values.

5.5.2.3 Using bitumen Pen 50/70

Figure 5.20 to Figure 5.22 demonstrate the effect of foaming water content (FWC) on the ITSM values for FB 50/70 at 180^{o}C. This temperature was used since it was considered that for harder bitumen, the higher temperature will give better results. Detailed data for all test specimens can be read in the figures. The specimens were evaluated at test temperatures of 20^{o}C and 5^{o}C. When all specimens had been tested at 5^{o}C, the specimens were then soaked in a water bath at a temperature of 40^{o}C for 68 to 72 hours. This conditioning procedure was in accordance with BS EN 12697-12: 2003, which is a standard test for assessing water sensitivity of materials using the indirect tensile strength (ITS) test. However, this study applied the ITSM test for assessing water sensitivity of specimens produced at different FWC values. The results are intended to give an indication of the effect of FWC on the mixture stiffness after the soaking treatment. As shown in Figure 5.20, both wet density and dry density are also included in order to give complete information related to the ITSM values.

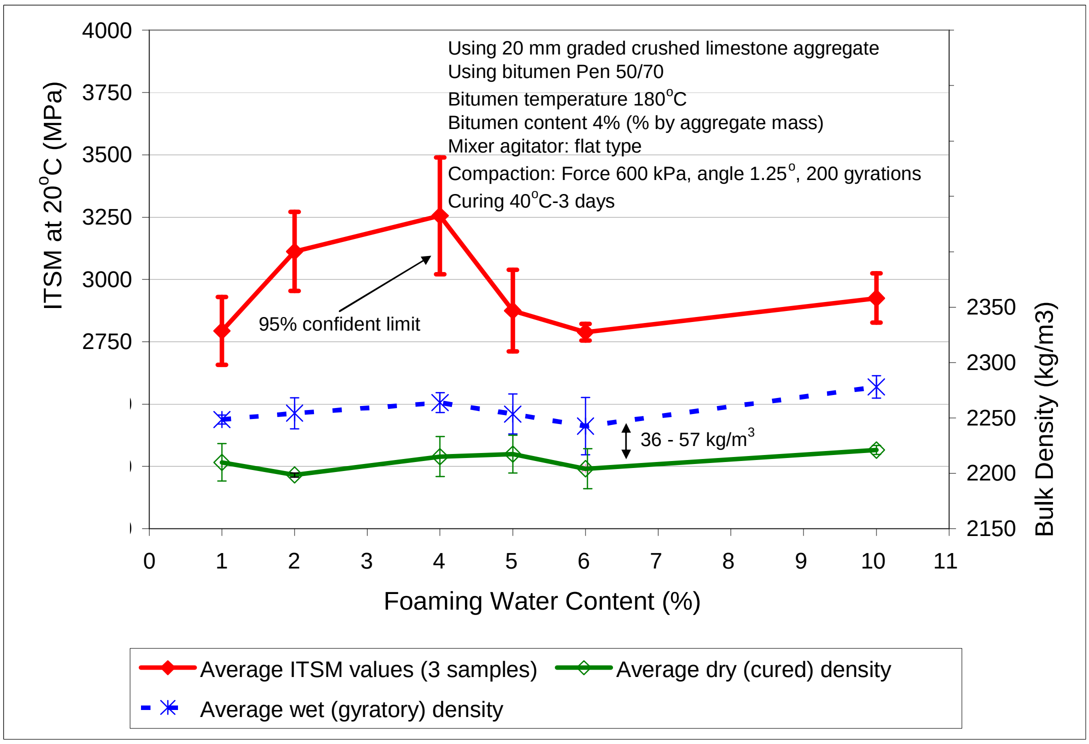


Figure 5.20 - Effect of foaming water content on the ITSM values for specimens generated using bitumen Pen 50/70

It can be seen that the dry density values are about 36 to 57 kg/m^3 lower than that wet density values due to loss of water during the curing process (at 40^{o}C for 3 days). However, several large stones on the surface of the specimen were also sometimes lost due to lack of bonding. In general, the trend of dry density is parallel to that of wet density across the FWC values.

In line with the test result using bitumen Pen 70/100, it is clearly evident that FWC has an effect on the ITSM values at a test temperature of 20^{o}C (see Figure 5.20). This time the optimum performance was apparently obtained at a FWC of 4%. The stiffness values drop dramatically at 5% and 6% foaming water. Based on the 95% confidence limits, the specimens at FWC of 2%-4% perform better than at other FWC applications. At a test temperature of 5^{o}C, the effect of FWC is more clearly evident than at 20^{o}C, as shown in Figure 5.21. This time the optimum performance is obtained at a FWC of 2% and the ITSM values at 2%-5% foaming water are higher than at other FWC applications.

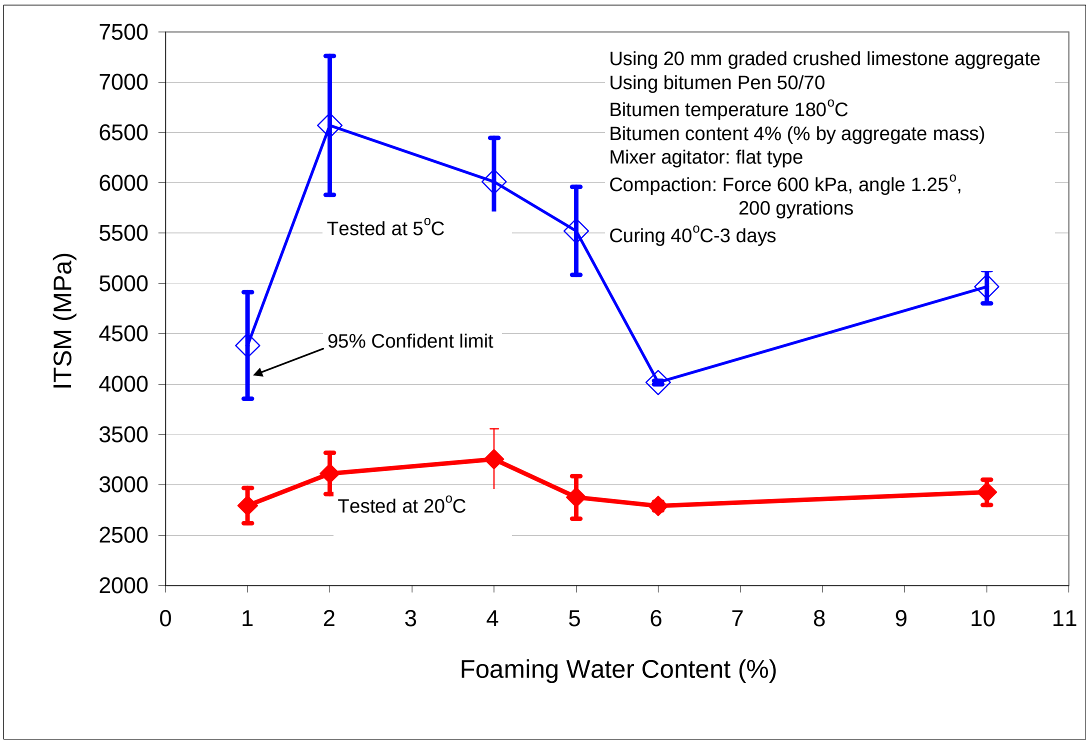


Figure 5.21 - Effect of test temperature on the ITSM values for specimens generated using bitumen Pen 50/70

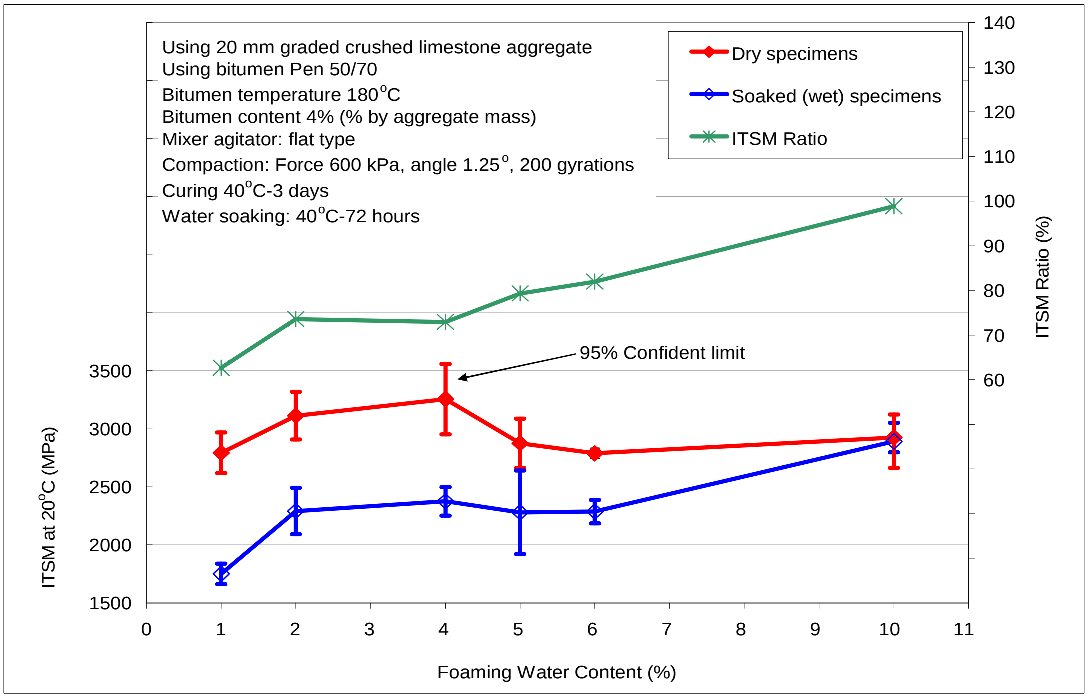


Figure 5.22 - Effect of water soaking on the ITSM values for specimens generated using bitumen Pen 50/70

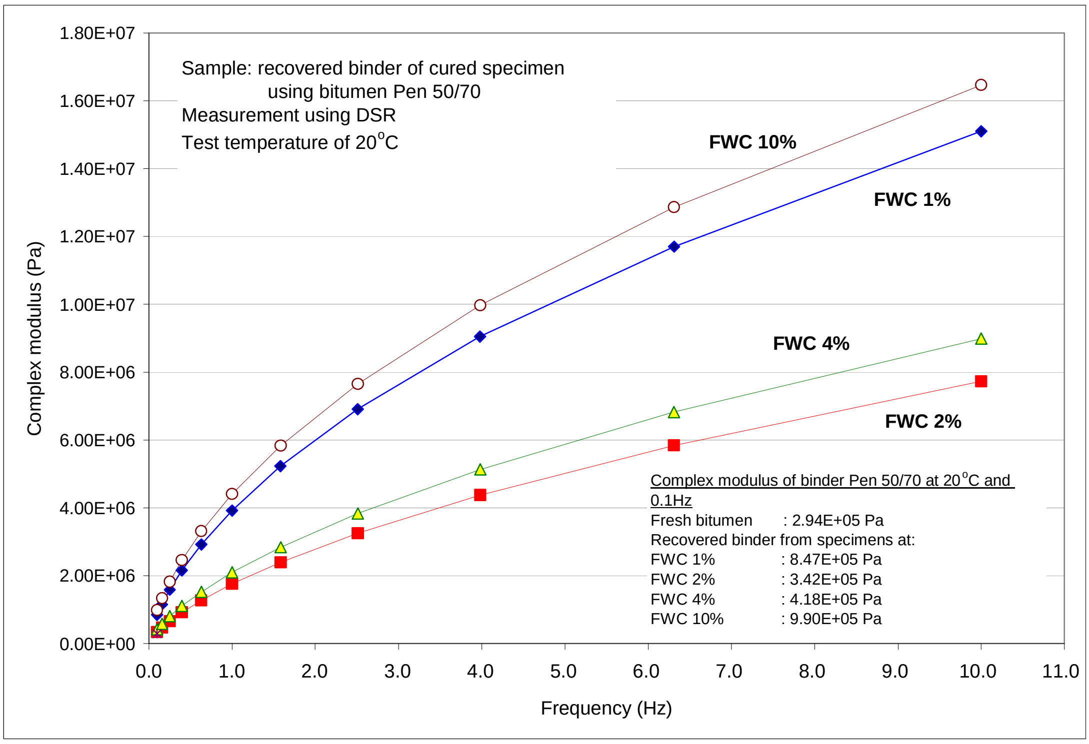


Figure 5.23 - Complex modulus of binder at various frequencies measured using DSR of recovered binder of cured specimens produced using bitumen 50/70 at various FWC values.

As shown in Figure 5.22, only the specimens with FWC higher than 5% have acceptable resistance to water damage (ITSM ratio > 80%). Interestingly, a higher FWC gives a higher ITSM ratio. The ITSM ratio was calculated from the wet ITSM divided the dry ITSM value. The specimens at a FWC of 10% demonstrated no stiffness reduction after water soaking. It should be noted that water soaking at a temperature of 40^{o}C may affect both specimen damage (ITSM value decrease due to water infiltration) and ageing (ITSM value increase due to bitumen stiffness increase). The recovered binders of these specimens were therefore investigated using the DSR (Dynamic Shear Rheometer). The results are presented in Figure 5.23. It can be seen that the binder stiffness of specimens produced at FWC of 1% and 10% is higher than those at FWC of 2% and 4%. The complex moduli of the recovered binders were found to be higher than the complex modulus of fresh binder measured at a frequency of 0.1 Hz (see notes in the Figure 5.23). It means that the treatment of the specimens caused binder ageing. Therefore the high wet ITSM values at a FWC of 10% are probably due to binder ageing whereas the lower wet ITSM values at a FWC of 1% might be due to specimen damage.

5.5.2.4 Using bitumen Pen 160/ 200

The effect of foaming water content (FWC) on the ITSM values using FB 160/220 at 150°C can be seen in Figure 5.24 to Figure 5.26. Unlike specimens using Pen 50/70 and Pen 70/100, no clear optimum performance was observed. At a test temperature of 20°C, specimens with FWC up to 4% exhibited slightly higher stiffness than specimens with FWC 5% to 10%. However at a test temperature of 5°C, the ITSM values at FWCs of 4% and 10% were apparently higher than others. Overall, based upon the 95% confident limits, the effect of FWC on ITSM values is not significant.

As shown in Figure 5.26, after water soaking at 40°C for 72 hours, all specimens exhibited higher ITSM values and hence their ITSM ratios were more than 100%. As was evident for specimens using bitumen Pen 50/70, the slightly increased ITSM values imply the binder ageing.

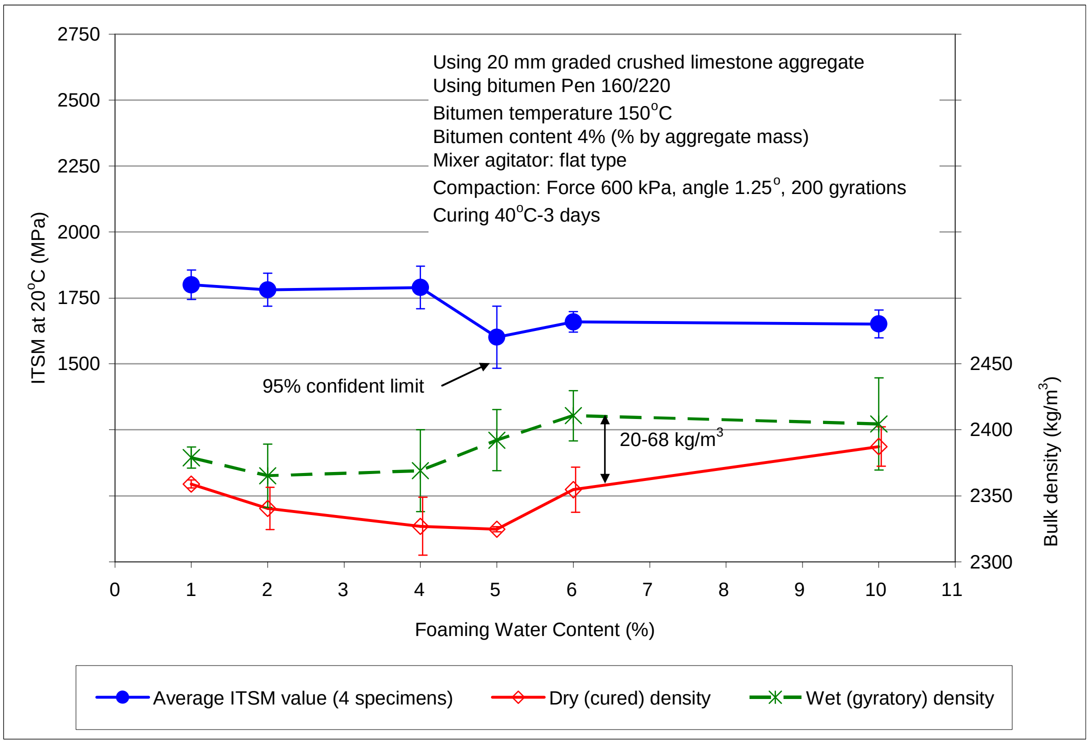


Figure 5.24 - Effect of foaming water content on the ITSM values for specimens generated using bitumen Pen 160/220

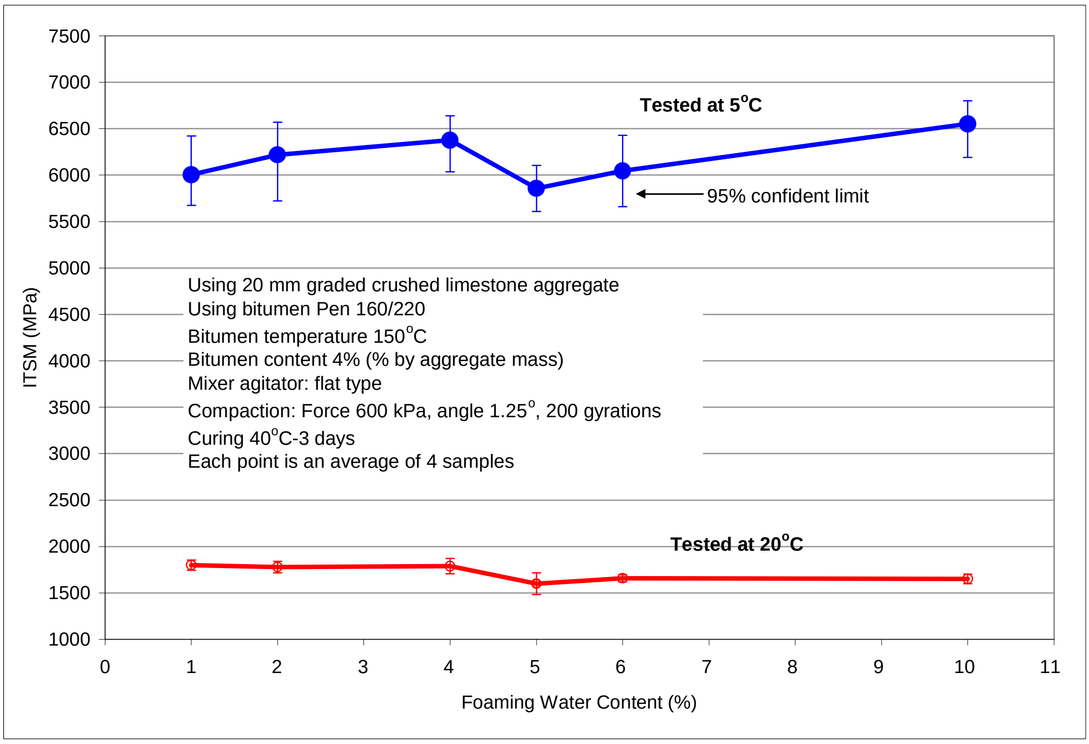


Figure 5.25 - Effect of test temperature on the ITSM values for specimens generated using bitumen Pen 160/220.

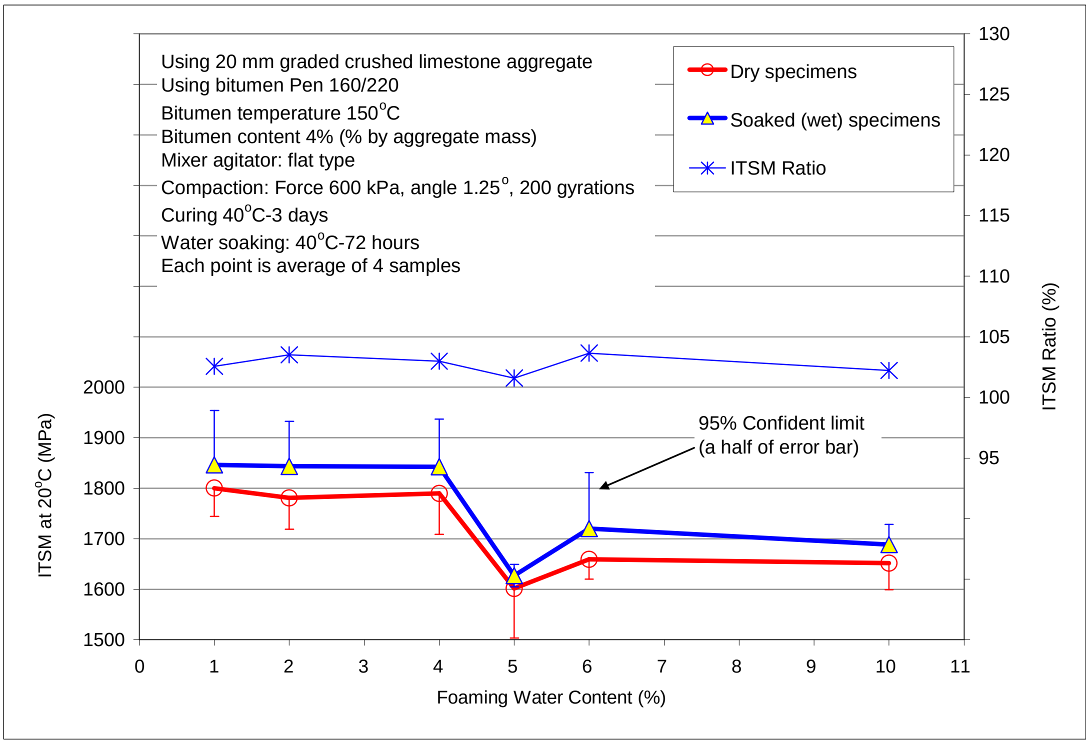


Figure 5.26 - Effect of water soaking on the ITSM values for specimens generated using bitumen 160/220.

5.5.3 Effect of Bitumen Temperature

Figure 5.27 shows the ITSM values of specimens produced using three different types of binder in which the foamed bitumen was generated at different temperatures. All specimens were produced at a FWC of 5%. The results reveal that the effect of bitumen temperature between 140^{o}C and 180^{o}C is not particularly significant for mixture properties. Based upon the 95% confident limit, the ITSM values of specimens produced at different bitumen temperatures for one binder type are relatively similar.

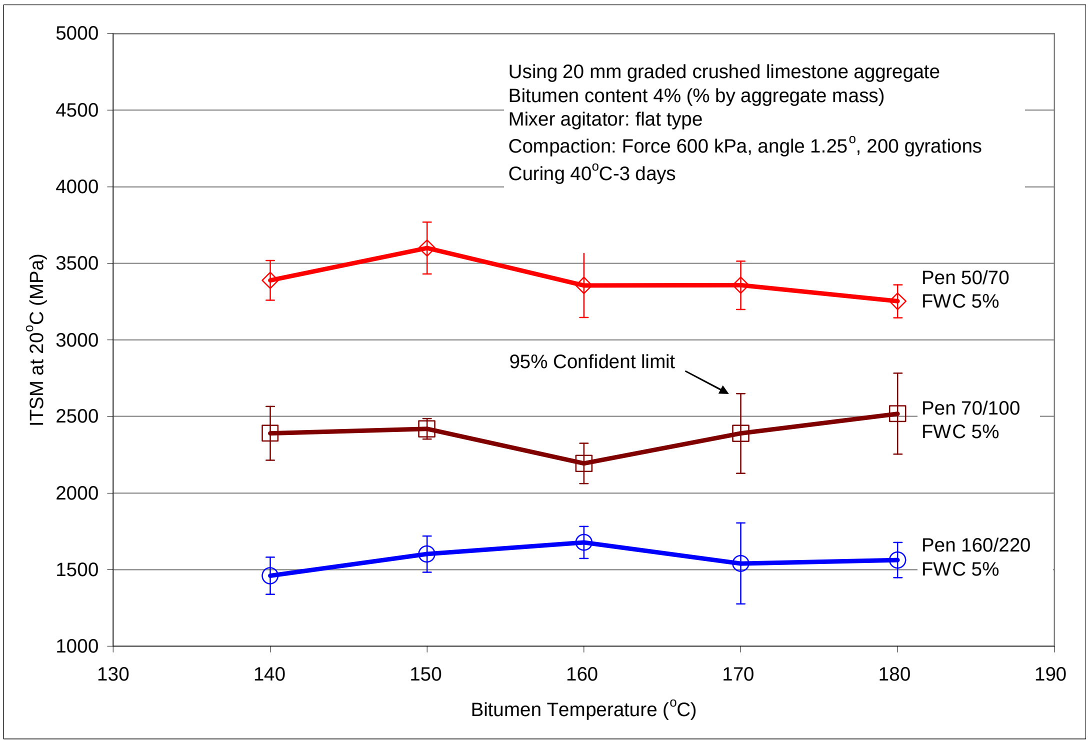


Figure 5.27 - Effect of bitumen temperature on the ITSM values for specimens produced using bitumen Pen 50/70, Pen 70/100 and Pen 160/220.

5.5.4 Evaluation of test data variability

Variability (or repeatability) of ITSM test data at a test temperature of 20^{o}C for specimens produced using three binder types at all selected FWC values was evaluated using standard deviation (SD), 95% confident limit (CL) and coefficient of variation (CV). SD is a measure of how values are dispersed from the mean value, 95% CL is an interval estimate of population data with an error level of 5%, whereas CV expresses the degree of data variation. The results of the evaluation are shown in Table 5.2 to Table 5.4 for specimens produced from the same batch (3-4 specimens) and in Table 5.5 for specimens produced from two different batches (7-8 specimens). The 10% of mean value is also shown to indicate the tolerance limit in the ITSM test (BS DD 213: 1993) between two test orientations specified. It can be seen that for specimens from the same batch almost all parameter values are less than the tolerance limits, i.e. SD and 95% CL are less than the 10% of mean value and CV is less than 10%. The only exception is found at a FWC of 1% for specimens produced using FB 70/100, where all parameters are greater than the tolerance requirement. However, if the variability is evaluated using test data from different batches, only for specimens FB 50/70 (FWC 5%, 180^{o}C) are the parameters lower than the tolerance limits. This may imply the variation of foamed bitumen properties between batches.

Table 5.2 - Variability evaluation of ITSM test data from specimens produced from the same batch specimens (using FB 50/70)

FWC (%)	1	2	4	5	6	10
mean value (MPa)	2794	3112	3256	2875	2788	2925
Standard deviation (MPa)	155	181	267	187	38	113
95% Confident limit (MPa)	175	205	302	212	44	128
Coefficient of variation (%)	5.5	5.8	8.2	6.5	1.4	3.9
10% of mean value (MPa)	279	311	326	288	279	293

Table 5.3 - Variability evaluation of ITSM test data from specimens produced from the same batch specimens (using FB 70/100)

FWC (%)	1	2	4	5	6	8	10
mean value (MPa)	2578	2988	3106	3336	2906	3041	3056
Standard deviation (MPa)	288	182	81	114	104	234	211
95% Confident limit (MPa)	282	206	80	112	102	230	207
Coefficient of variation (%)	11.2	6.1	2.6	3.4	3.6	7.7	6.9
10% of mean value (MPa)	258	299	311	334	291	304	306

Table 5.4 - Variability evaluation of ITSM test data from specimens produced from the same batch specimens (using FB 160/220)

FWC (%)	1	2	4	5	6	10
mean value (MPa)	1550	1531	1540	1351	1409	1402
Standard deviation (MPa)	57	64	83	120	39	54
95% Confident limit (MPa)	56	62	81	118	39	53
Coefficient of variation (%)	3.7	4.2	5.4	8.9	2.8	3.9
10% of mean value (MPa)	155	153	154	135	141	140

Table 5.5 - Variability evaluation of ITSM test data from specimens produced from two different batches (for three binder types)

Specimen type	FB 50/70 FWC 5% 180^{o}C	FB 70/100 FWC 5% 180^{o}C	FB 160/220 FWC 5% 150^{o}C
mean value (MPa)	3090	2831	1476
Standard deviation (MPa)	242	394	174
95% Confident limit (MPa)	179	292	120
Coefficient of variation (%)	8	14	12
10% of mean value (MPa)	309	283	148

5.6 Resistance to Permanent Deformation

The effect of foaming water content (FWC) on the resistance to permanent deformation has been investigated using the RLAT and the results are presented in Figure 5.28 to Figure 5.30. An example is presented in Figure 5.28, in which the characteristics of axial strain increase with load cycles during the test can be identified.

The procedure to prepare the specimens was similar to ITSM test specimen preparation. All specimens were firstly measured for stiffness (ITSM test). Before the RLAT, the top and bottom faces of the tested specimens were smoothed using sand paper. The sand paper was used since the specimen can not be trimmed by a saw. The weight and dimension (diameter and thickness) of each tested specimen were determined in accordance with section 5 of BS DD 226: 1996. The ends of the specimen were coated with a thin layer of silicone grease and black powder. The specimens were brought to the test temperature at least 2 hours before testing.

Two types of aggregate gradation were evaluated using different testing modes. The specimens generated using 20 mm graded aggregate were tested at a vertical stress of 200 kPa and a temperature of 30^{o}C (see Figure 5.29), whereas the specimens using a 10 mm graded aggregate were tested at 100 kPa and a temperature of 40^{o}C (see Figure 5.30). The latter test series was conducted in order to clarify the effect of FWC on the measured axial strain; this was not clear in the first test series.

The evaluation was based upon three parameter values, i.e. strain at 1800 pulses, strain at 3600 pulses and strain rate over the second 1800 load cycles as shown in Figure 5.28. The strain at 1800 pulse was included because several specimens failed (i.e. exceeded strain limit) before 3600 pulses had been completed. The parameter of strain rate over the second 1800 cycles was used in order to understand the effect of binder type on the characteristics of permanent deformation at this pulse range.

As shown in Figure 5.29, the effect of FWC on the permanent deformation of specimens generated using 20 mm graded aggregate is not clearly defined. The only point is that at FWC of 6% the materials exhibit poor resistance to permanent deformation. This is particularly clear when evaluation is based upon the strain rate over the second 1800 cycles. The strain curves at 1800 pulses and at 3600 pulses look parallel and these curves show a slightly different trend compared to the strain rate curve. The compaction and stiffness characteristics of these specimens can be seen in Figure 5.17. There is no clear correlation between density/stiffness and resistance to permanent deformation. Overall, it is likely that the effect of FWC is not important for resistance to permanent deformation of foamed asphalt material using 20 mm graded aggregate. It is probable that the effect of the aggregate skeleton and binder type is more important than the effect of foam properties (with different FWC). Testing modes (stress level and test temperature) may also affect the test results causing the effect of foam properties not to be observed.

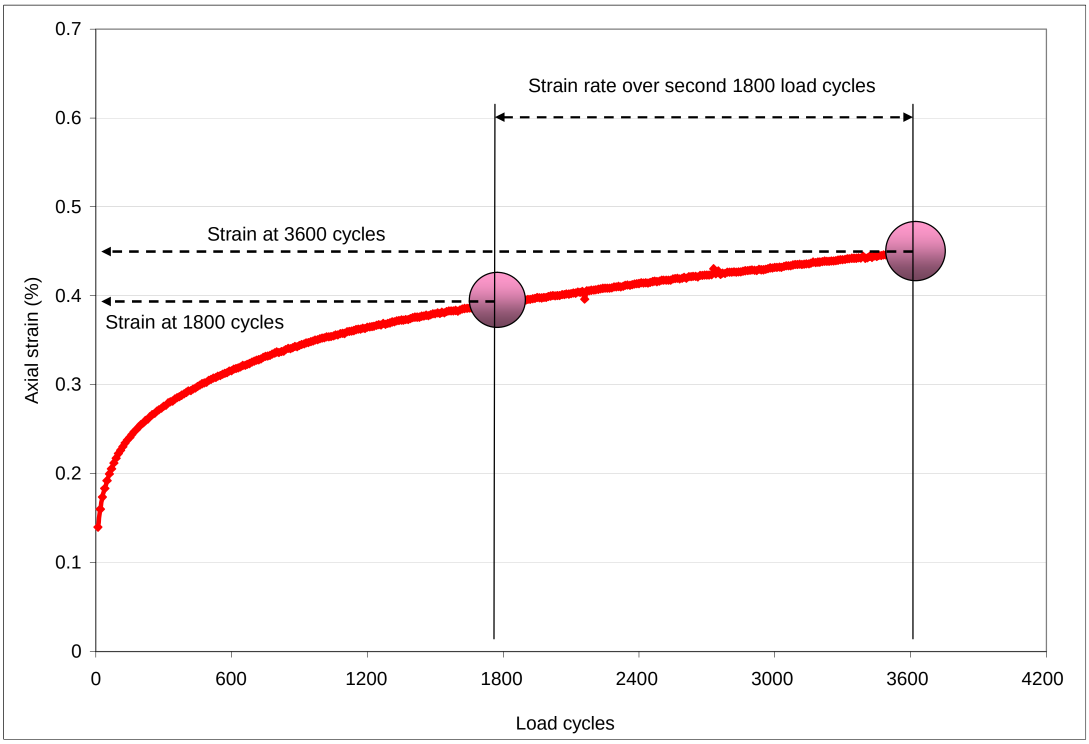


Figure 5.28 - Parameters used to evaluate RLAT results

It was decided to investigate the effect of binder in several ways, i.e. reduce the nominal size of aggregate (from size of 20 mm to 10 mm), increase the binder content (from 4% to 6.8%) and reduce the stress level (from 200 kPa to 100 kPa). The test temperature was also increased from 30°C to 40°C in order to achieve failure during testing.

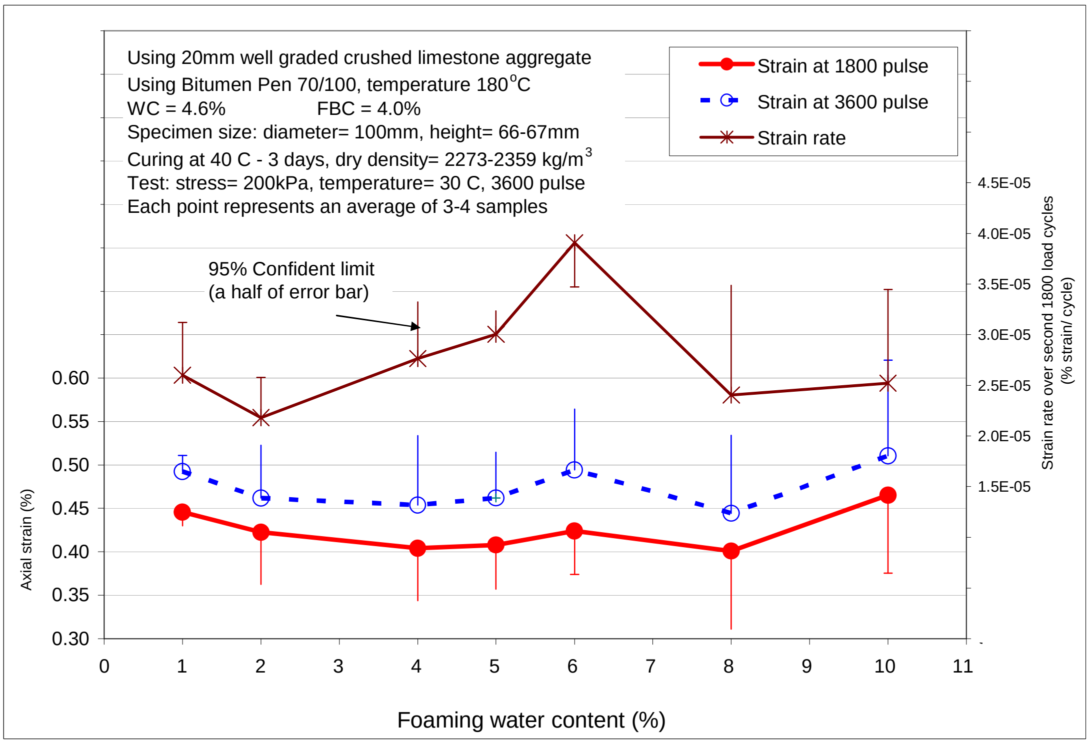


Figure 5.29 - Results of RLAT of specimens using 20 mm graded limestone aggregate with various FWC

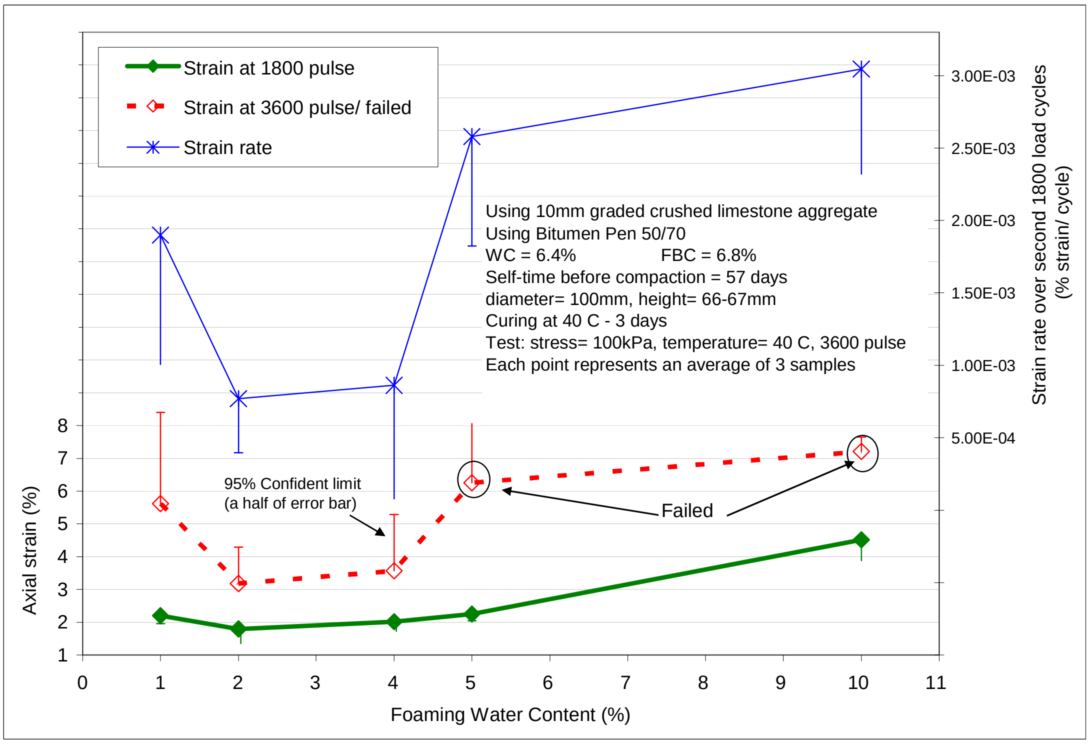


Figure 5.30 - Results of RLAT of specimens using 10 mm graded limestone aggregate with various FWC.

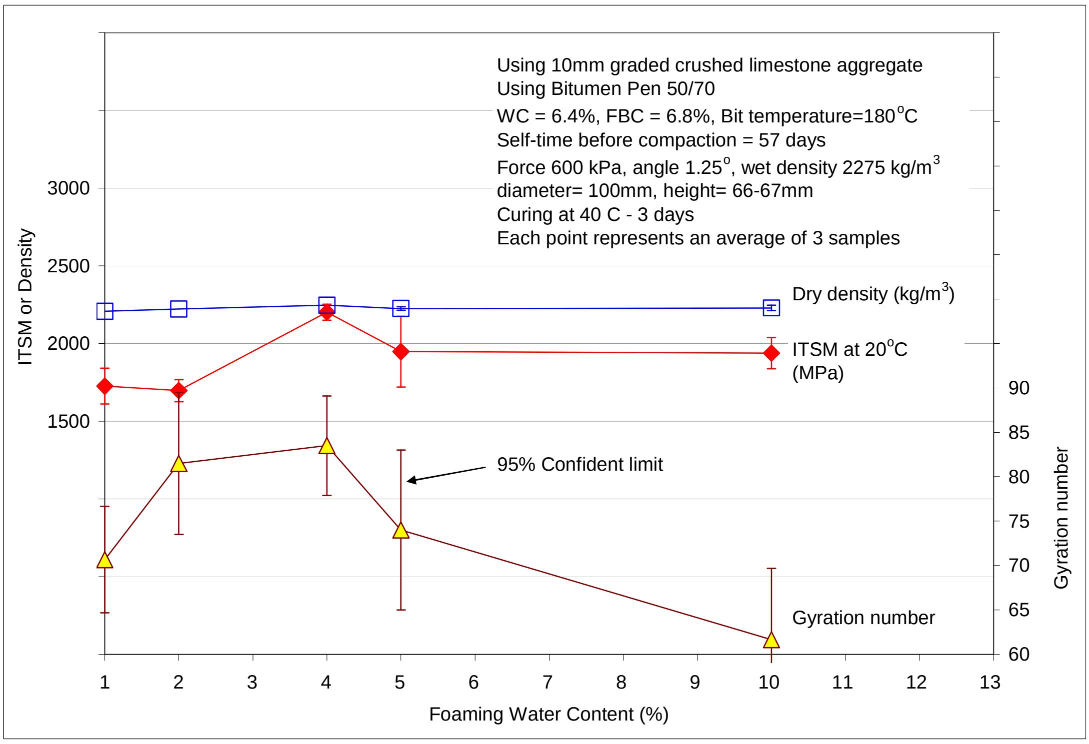


Figure 5.31 - Compaction and stiffness characteristics of specimens using 10 mm graded limestone aggregate.

Figure 5.30 shows the test results for 10 mm graded aggregate specimens. The compaction and stiffness characteristics of these specimens are presented in Figure 5.31. It is noted that this material was stored almost 2 months before compaction took place. To avoid any differences between specimens due to the storage period, the material was compacted to achieve a target density of 2275 kg/m^3 and hence the dry densities were all comparable. It can be seen that at FWC of 2% and 4% the specimens needed more gyration than at other FWC values. The trend of required gyration number did not noticeably influence their ITSM values. However, it appears that the gyration numbers shown in Figure 5.31 and the characteristics of permanent deformation resistance shown in Figure 5.30 are clearly linked. The trend of axial strain at 3600 pulses (or when strain limit exceeded) and the strain rate over the second 1800 load cycles are very close to the trend of gyration number. More energy of compaction appears to have resulted in a better resistance to permanent deformation. It is likely that the effect of FWC on the resistance to permanent deformation of a foamed asphalt material is less important than the effect of compaction characteristics (density or energy) or other parameters, e.g. aggregate skeleton.

5.7 Resistance to Fatigue

The test procedure used in this study is as described in Section 5.2.5. It was decided to investigate three different specimen types, i.e. specimens produced at FWC of 1%, 5% and 10%. Ten or eleven specimens were produced for each type. The ITSM values of specimens at FWC 10% were determined prior to ITFT testing at various stress levels, i.e. from 100 kPa to 300 kPa at intervals of 50 kPa. Based on these tests, a correlation between stress level and ITSM value was developed. Due to the possibility of specimen damage when tested at high stress loading, it was therefore decided to test the specimens using low stress loading. Subsequently the specimens produced at FWC values of 1% and 5% were tested at a horizontal deformation of 5 microns (stress level around 100 kPa to 150 kPa). The correlation developed at FWC of 10% was then utilised to correct the ITSM values of FWC 1% and 5% specimens. It is noted that both ITSM and ITFT were performed at a temperature of 20^{o}C. The results can be seen in Table 5.6 and Table 5.7, and also in Figure 5.32 to Figure 5.35.

Fatigue characteristics were identified using a plot of number of cycles (N) against N divided by vertical deformation (N/vd). These two values were recorded during testing. Basically, as described in Chapter 2, this plot was used by Read (1996) to define the initiation and propagation phases of fatigue. In this study, this plot was utilised to characterise the fatigue life of the material. The N value at which the N/vd reached its highest value was termed $N_{critical}$ (N_{cr}). The N value at which the specimen failed was termed $N_{failure}$ (N_f). Table 5.6, Figure 5.32 and Figure 5.33 show the fatigue characteristics of foamed asphalt materials. Increasing the applied stress level significantly reduces both N_{cr} and N_f as shown in Figure 5.32. It can also be seen in Figure 5.33 that the specimens with FWC of 5% performed better in fatigue at a stress level of 100 kPa than specimens with FWC of 1% and 10% since the stiffness at FWC of 5% is higher than both at FWC of 1% and 10% (see Figure 5.17 and Figure 5.18). However, although having lower stiffness, the specimens produced at FWC of 1% exhibited slightly better fatigue life than at FWC of 10%, although this may be due to scatter of ITFT data as commonly found by researchers. Table 5.6 indicates that for all cases N_{cr} was about 60% to 66% of N_f.

Figure 5.34 expresses the results in the form of a plot of N_f against the applied stress level. A power trend line was developed for each data set and hence the equation of a fatigue line and its R^2 value can be determined as shown in the figure. As an example, for FWC of 1%, the equation of the fatigue line is $y= 1239x^{-0.2636}$. This equation means that for monotonic loading (at N=1), the specimen should fail at a stress of 1239 kPa. The slope of the fatigue line is –0.2636, the negative sign meaning that the stress decreases with N_f. Greater slope or steeper fatigue line indicates that specimen performance is more sensitive to the applied stress. It can be seen that the fatigue lines for specimens produced at FWC of 1% and 5% are comparable, the line for FWC of 1% being slightly steeper than that for FWC of 5%, whereas the fatigue line for specimens at FWC of 10% appears far steeper than the other two. It was also found that data of the lower FWC give higher R^2 values.

Table 5.6 - The number of cycles to reach critical point ($N_{critical}$) and failure ($N_{failure}$) at various stress levels and foaming water content applications.

Specimen type and stress level (kPa)		$N_{failure}$	$N_{critical}$	% N_{cr}= $100*(N_{cr}/N_f)$
FWC 5%	100	18603	12020	65
	150	9565	5720	60
	200	3255	2060	63
	250	1172	770	66
	300	128	80	63
FWC 1%	100	10944	7070	65
FWC 10%	100	9660	6090	63

Table 5.7 - Fatigue characteristics of foamed asphalt materials produced at foaming water content of 1%, 5% and 10%.

Specimen type	Equation based on strain	Strain at 10^3 cycles	Strain at 10^4 cycles	Strain at 10^5 cycles
FWC 1%	$\varepsilon = 1590.5N^{-0.3118}$	185	90	44
FWC 5%	$\varepsilon = 1048.3N^{-0.2618}$	172	94	51
FWC 10%	$\varepsilon = 3881N^{-0.4183}$	216	82	31
	Equation based on Number of cycles	Cycles at 50 microstrain	Cycles at 100 microstrain	Cycles at 200 microstrain
FWC 1%	$N= 1.43E+10\ \varepsilon^{-3.15}$	63618	7169	807
FWC 5%	$N= 7.39E+10\ \varepsilon^{-3.50}$	83608	7390	653
FWC 10%	$N= 7.24E+10\ \varepsilon^{-2.15}$	16105	3629	818

When the results are plotted in the form of N vs strain, the fatigue line for FWC of 5% is clearly better than the two others as shown in Figure 5.35. It can be seen in

Table 5.7 that specimens produced at FWC of 5% will have better fatigue performance at strains lower than 200 microstrain, whereas for strains higher than 200 microstrain the specimens produced at FWC of 10% performed better than the others. A comparison between foamed asphalt (FA) and hot-mix asphalt (HMA) is given in Figure 5.36, in which the bitumen volume of HMA (20mm DBM) was slightly higher than that of FA (FWC 5%). It can be seen that the fatigue life of foamed asphalt is very short in comparison with HMA. At 200 microstrain, the fatigue life of FA is about 650 cycles whereas that of HMA is about 30000 cycles. This can be understood since HMA is a fully bonded material, with less void content and hence more resistance to fatigue.

Overall, the specimens produced at FWC of 10% exhibit the poorest fatigue performance, but these specimens were tested at relatively high stress to determine their ITSM value prior fatigue testing. It may be that their performance would not be significantly poor than that of specimens at FWC of 5%, if their condition was fresh at ITF testing.

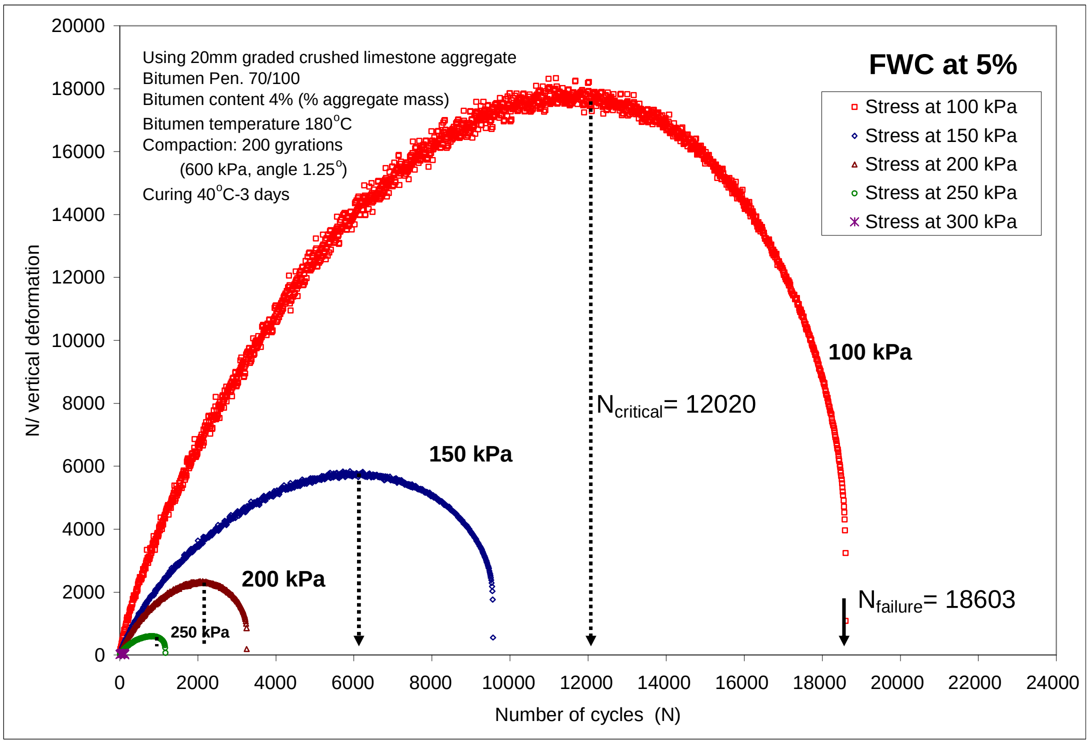


Figure 5.32 - Fatigue characteristics of foamed asphalt materials at different stress levels (specimens produced at FWC of 5%)

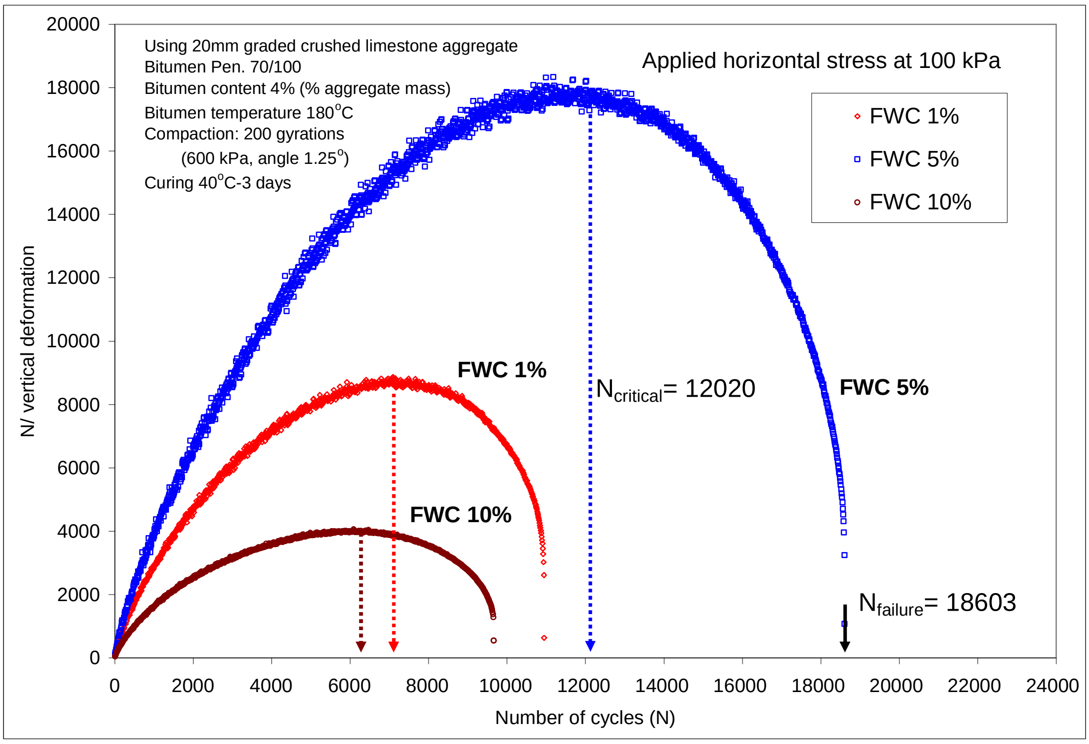


Figure 5.33 - Effect of foaming water content on the fatigue characteristics at a stress level of 100 kPa

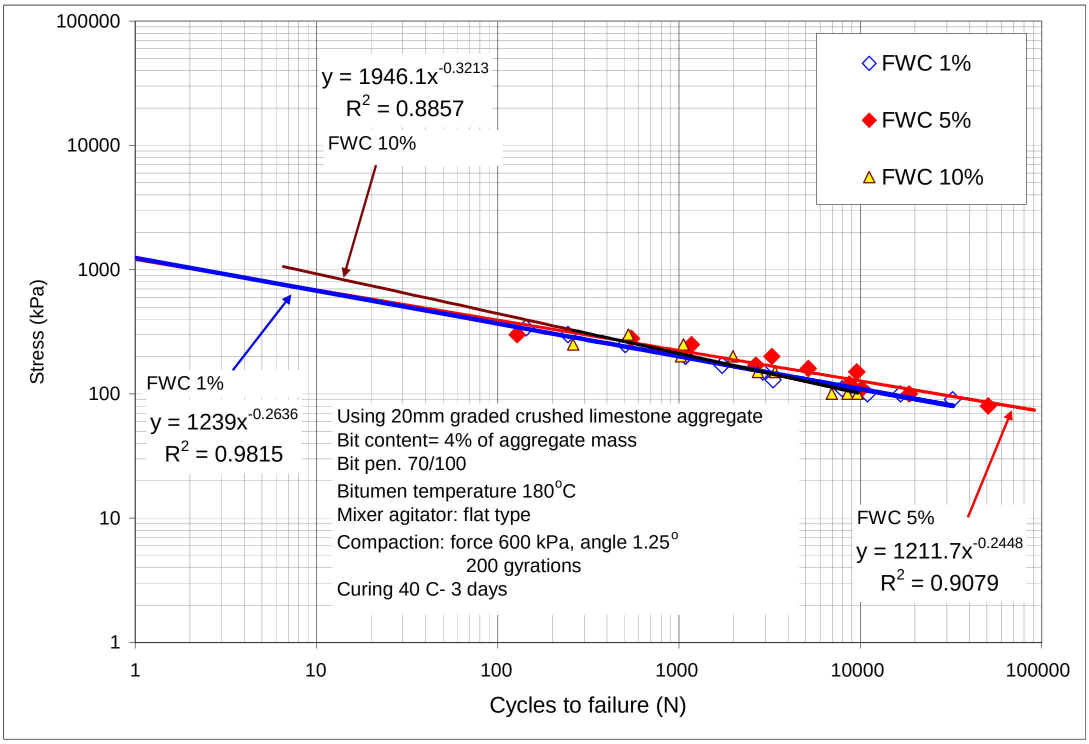


Figure 5.34 - Fatigue characteristics of foamed asphalt materials based on stress for specimens produced at FWC of 1%, 5% and 10%.

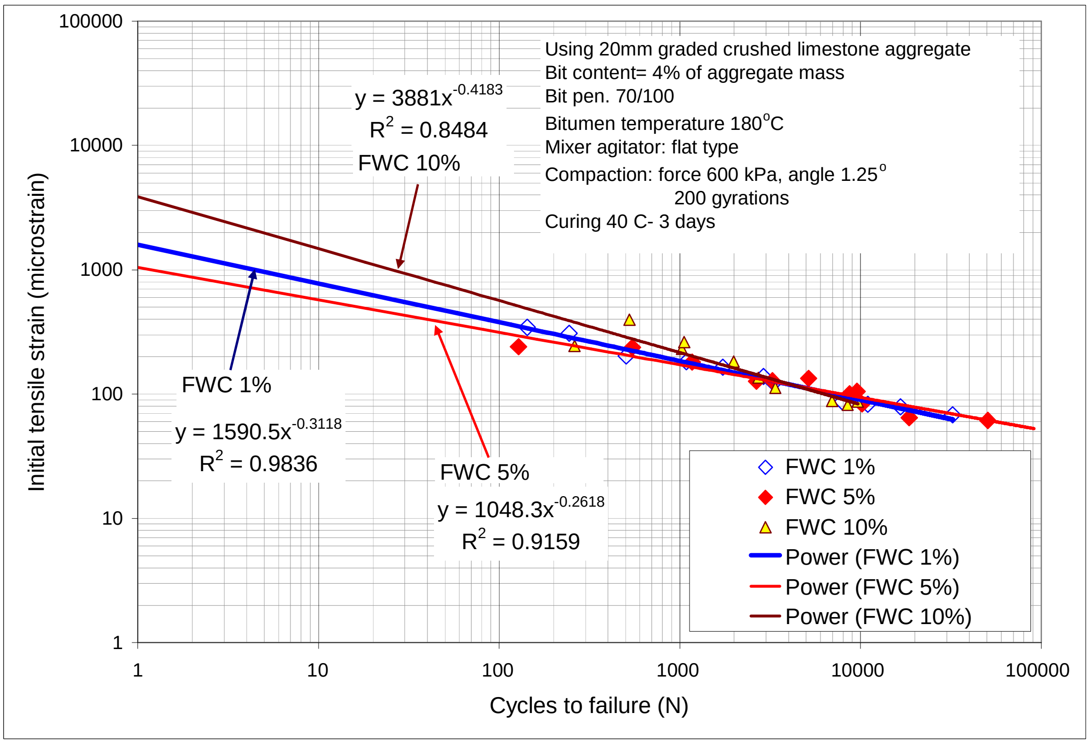


Figure 5.35 - Fatigue characteristics of foamed asphalt materials based on strain for specimens produced at FWC of 1%, 5% and 10%.

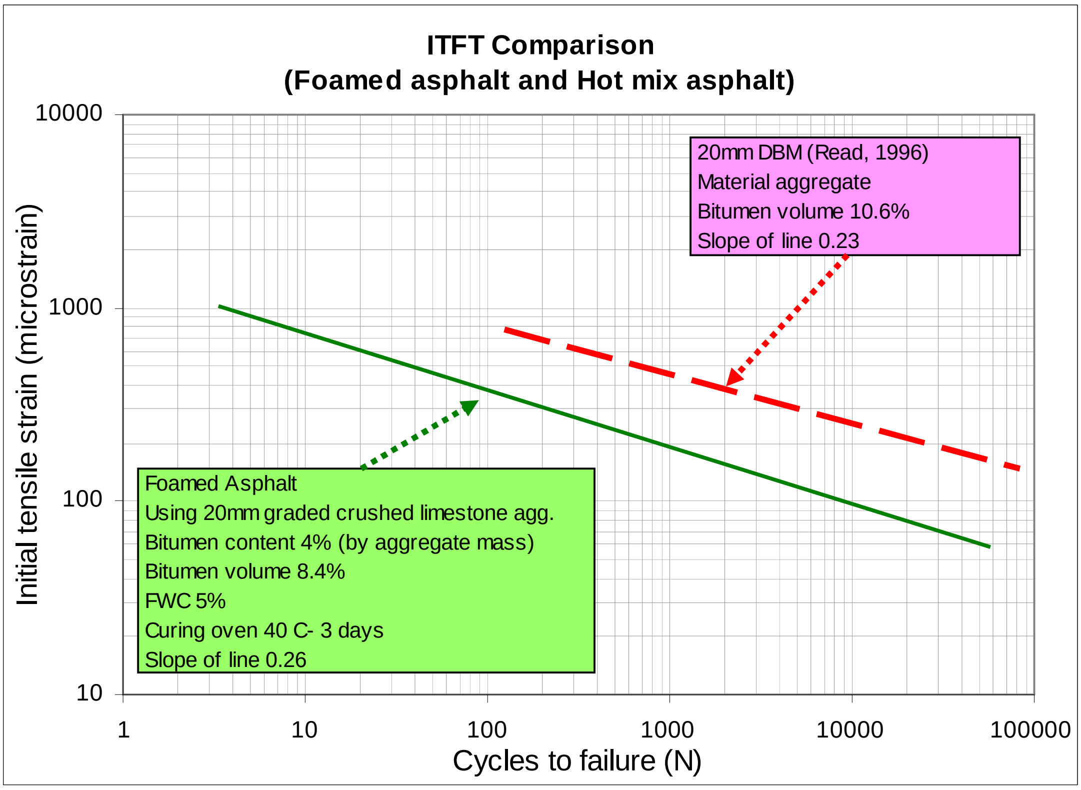


Figure 5.36 - Comparison of fatigue characteristics between foamed asphalt and hot mix asphalt (20mm DBM)

5.8 Discussion and Conclusions

The advantage of using the gyratory compactor is that of the compaction characteristics of materials can be studied. Therefore differences in mixture compactability can also be evaluated easily. A summary of the compactability investigation is as follows:

- The compaction modes did not significantly affect the stiffness values as long as the final densities were comparable. However, this was not necessarily the case for permanent deformation for which the number of gyrations applied in the compaction process will affect to the particle arrangement and hence affect the vertical strain in the RLAT.
- At a selected compaction mode, the wet (gyratory) density tends to increase with the number of gyrations, but this is not the case for dry density which tends to decrease at high gyration numbers. Additionally, it seems the ITSM values and dry densities are clearly linked.

The effect of foam properties on the mixture compactability has been investigated using three types of binder, varied according to FWC application and bitumen temperature. Essentially, the data reveals no significant trends and the effect of FWC and bitumen temperature can not be clearly observed (see Figure 5.8 to Figure 5.12). It may be that these factors are not as important as other factors such as moisture content or aggregate particles. When the correlation was evaluated using a linear trend line, the mixture compactability tended to increase with the FWC; with the bitumen temperature it tends to increase for low penetration binder but tends to decrease for high penetration binder. The important point is that the binder types affected mixture compactability significantly, the mixture produced using binder of Pen 50/70 giving poor density. It means foamed asphalt materials using softer binder tend to give better compactability performance than those using harder binder since softer binder acts more lubricant during compaction due to having lower viscosity.

Due to foamed asphalt not being a fully bound material and its binder not being as continuous as an HMA’s binder, it is therefore interesting to understand its tensile stiffness characteristics. The trends of horizontal deformation/ stress and test temperature effects on the ITSM value were found to be similar to those of HMA, the ITSM values decreasing with those two parameters. However, the ITSM values of foamed asphalt were found to be more sensitive to applied horizontal deformation and less temperature susceptible than those of HMA (see Figure 5.14 and Figure 5.15). These facts may indicate that the ITSM test is suitable to evaluate the stiffness of foamed asphalt materials. It is supposed that binder distribution in the mixture controls the stiffness value of foamed asphalt materials. Necessarily, well mixed specimens will tend to be more sensitive than poorly mixed specimens in terms of the effect of horizontal deformation and test temperature. This is seen in that the ITSM value reduction for the well mixed specimens was higher than for the poorly mixed specimens as shown in Figure 5.14, due to their ITSM values being so different.

As the focus of this study, the correlation between foam properties and mixture performance has been explored deeply. It was found that foam properties and mixing quality are two important aspects, influencing each other, in developing mixture performance. Foam property variation was created by using different FWC values and bitumen temperatures, whereas mixing variation was created by different mixer agitators. The results can be explained as follows:

- The flat agitator performed better at mixing than the dough hook and it aided material properties and enhanced the ITSM value (see Figure 5.16). Binder in the well mixed specimens was better distributed and hence more continuous than in the poorly mixed specimens. In the compacted specimen, the presence of this binder reduces the strain under loading and hence improves the ITSM value.
- Specimens mixed at different FWC values using the dough hook exhibited no significant differences in ITSM (see Figure 5.16); minor fluctuations might be representative of small variations in mixture density and moisture content. It can be seen that, since the dough hook gives poor mixing, different foam properties do not have any effect on the binder distribution in the mixture.
- Interestingly, when specimens were mixed using the flat agitator their cured ITSM values increased dramatically and the effect of FWC was clearly evident (see Figure 5.16). However the effect of FWC was not clearly observed for

specimens produced using bitumen Pen 160/220. It is supposed that mixtures produced using soft bitumen are easy to mix and result in very good mixing for all cases of FWC applications and hence there is no difference between specimens performance at different FWC values.

- Optimum performance was obtained at a FWC of 5% for specimens prepared using FB 70/100 at 180^{o}C (and 4% for FB 50/70 at 180^{o}C). This became even more evident when the full range of testing temperatures (see Figure 5.18 and Figure 5.21) and curing conditions (Figure 5.19) were considered. It may be that the optimum performance is obtained with the best binder distribution in the mixture, in which state the mixture is more sensitive to testing temperature and curing condition. The case of specimens produced using bitumen Pen 160/220 is very interesting. As shown in Figure 5.37, at a test temperature of 20^{o}C the ITSM values of these specimens were lower than the ITSM values of specimens produced using bitumen Pen 70/100, but at a test temperature of 5^{o}C the result was the reverse. This may be explained as follows. Since bitumen Pen 160/220 is softer than bitumen Pen 70/100, specimens using Pen 160/220 are better mixed than specimens using bitumen Pen 70/100. Therefore, it is likely that at a test temperature of 5^{o}C binder distribution is more important than binder stiffness in developing the ITSM value.
- The performance of foamed asphalt materials under water soaking has been investigated. Two types of specimen, produced using bitumen Pen 50/70 and Pen 160/220, were evaluated in terms of their ITSM values before and after water soaking at 40^{o}C for 3 days. The test results as shown in Figure 5.22 and Figure 5.26 are very interesting. It was found that the resistance to water damage of all Pen 160/ 220's specimens was acceptable since the ITSM ratios were all higher than 80%. For Pen 50/70, the result was acceptable only on the specimens with FWC higher than 5%. The fact that the ITSM ratio increased with FWC might be due to binder ageing. It is known that the binder was ageing, since the recovered binder moduli were higher than those of the fresh binder (see Figure 5.23). It should be noted that binder ageing was not caused by the foaming process but it occurred in the compacted specimens. Since the binder is not continuously distributed in the mixture, the void content becomes higher and the

bitumen film becomes thicker. High void content will make the mixture less durable due to oxidation but a thick binder film will reduce binder hardening (making it more durable). So, durability of foamed asphalt mixtures is an interesting topic requiring further research.

- At a FWC of 5%, bitumen foaming temperature was found to be less significant than FWC in developing stiffness. This can be understood since foamed bitumen properties do not change significantly with bitumen temperature.

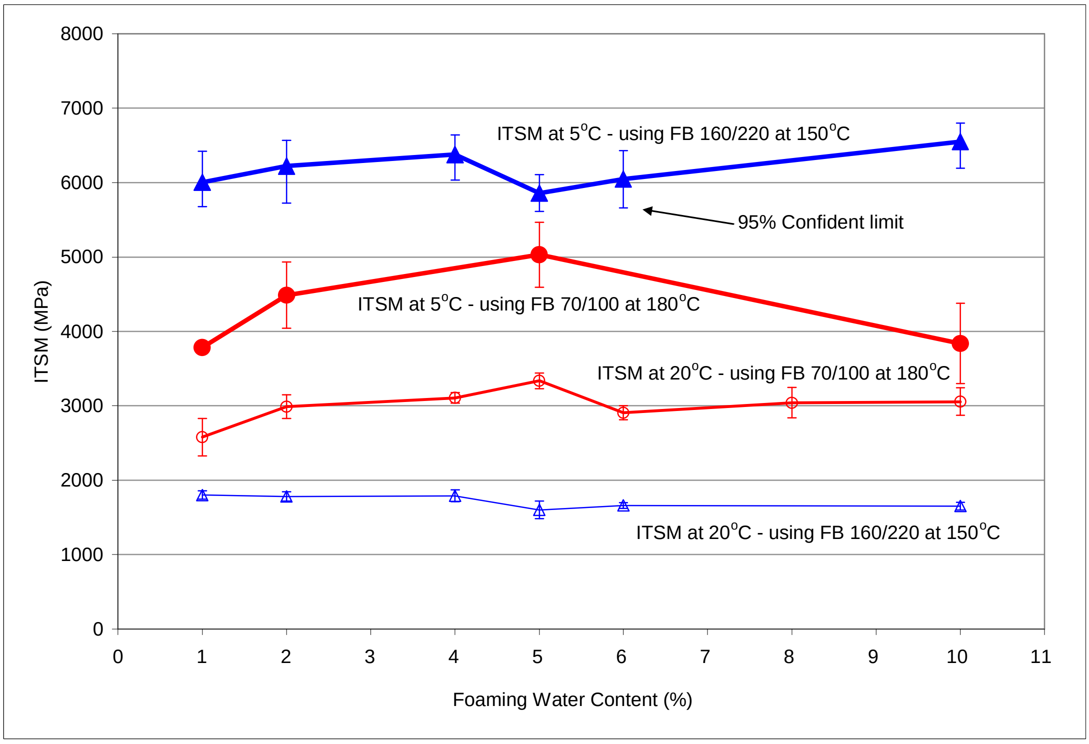


Figure 5.37 - Comparison of ITSM values between specimens produced using bitumen Pen 70/100 and Pen 160/ 220 at test temperatures of 5°C and 20°C

As indicated in the PTF scale testing (described in Chapter 3), foamed asphalt materials tend to fail in rutting, which is affected by the binder type, mixture proportion and the presence of cement. In this chapter, cylindrical specimens of foamed asphalt materials, produced at various FWC values, were evaluated for their resistance to permanent deformation under the RLAT. Two types of mixture were used, i.e. specimens using 20 mm and 10 mm graded aggregates, which resulted in strains at 1800 pulse about 0.4-0.5 % and 2-5% respectively. However these two mixture types were tested at different stress and temperature, and hence these strains can not be compared directly. Actually, the 10 mm graded specimens were selected in order to explore the correlation between foam properties and resistance to permanent deformation, since the relationship was not clear using 20 mm graded specimens. However, it was found that for 10 mm graded specimens, their resistance to permanent deformation was closely related to compaction energy.

Specimens generated at FWC of 1%, 5% and 10% were produced in order to investigate fatigue characteristics of foamed asphalt materials. The fatigue life was found to be very short in comparison with hot-mix asphalt materials (e.g. 20mm DBM), about 600-800 cycles at 200 microstrain. The $N_{critical}$ value (number of cycles (N) at the peak value of N/vertical deformation) was generally about 60%-66% of $N_{failure}$. This value is about 90% for HMA mixtures (Read, 1996). It means, following Read's (1996) idea, that foamed asphalt materials will initiate cracking faster than HMA, but have a longer crack propagation life. The fatigue life was significantly affected by applied stress level and clearly linked with the stiffness value. Based on the developed fatigue lines, the specimens produced at FWC of 5% were found to have the lowest slope (best performance) and the specimens produced at FWC of 10% were the steepest (worst performance). However their fatigue lives at 200 microstrain were comparable. It can be therefore deduced that in the field the fatigue performance of foamed asphalt materials with various FWC values is not significantly different.

In conclusion, foamed bitumen properties (with different FWC) have a moderate effect on foamed asphalt performance. This effect can be clearly evident based on stiffness evaluation, but not on fatigue and permanent deformation assessments. It is noted that the effect can only be clearly defined using well mixed specimens in which the binder is well distributed in the mixture. It can be concluded that at FWC of 1% the mixture will perform poorest. It is then recommended to use FWC of 2% to 4%, that statistically results in better performance than other FWC applications. It can thus be argued that peak performance from a foamed asphalt (assessed for example using stiffness criteria) can only be guaranteed when the mixture is manufactured and compacted at optimum FWC.

6 EXPLORATION OF THE EFFECT OF FOAMED BITUMEN CHARACTERISTICS ON MIX PROPERTIES

6.1 Introduction

Chapter 6 presents a discussion exploring the link between foamed bitumen characteristics and mixture properties. This chapter forms a bridge between Chapter 4 and 5. The role of binder distribution in the mixture is discussed initially as this aspect is key to understanding foamed asphalt properties. A detail discussion about the relationship between foam and mixture properties is presented in Section 6.3 and the chapter ends with a discussion of how foamed asphalt mixture (FAM) develop its stiffness, since the formation of bitumen-aggregate bonds is different from that of other common bituminous mixtures.

6.2 The Role of Binder Distribution in Foamed Asphalt Mixtures

The role of binder distribution is an important key to understanding foamed asphalt properties. Binder distribution, a result of the mixing process, is a bridge linking foamed bitumen characteristics and cold-mix properties. This study will therefore describe a mechanism of binder distribution. The term ‘binder distribution’ is preferred rather than ‘aggregate coating’ to describe the specific formation of aggregate-bitumen mixture in FAMs, which is quite different compared to that of hot-mix asphalt (HMA).

6.2.1 Appearance of binder distribution

In this section, the formation of binder distribution in various trial mixtures is studied in order to understand and describe a mechanism of binder distribution for FAMs.

Firstly, a clear difference is observed between foamed bitumen and hot bitumen when they are sprayed onto cold wet aggregates. The result mixture of foamed bitumen spraying is a well distributed binder across the aggregate phase, whereas hot bitumen spraying results in an uncombined asphalt mixture spotted with large aggregate-bitumen globules. The photographs of these two trial-mixtures can be seen in Appendix A: Figure A.6.1 and Figure A.6.2. These results can be explained by referring to the theory of foam as discussed in Chapter 4. In the foaming process, bitumen molecules diffuse from the bulk phase to the interface (Barinov, 1990). These molecules act as surfactants which are composed of a hydrophilic 'head' and a hydrophobic 'tail' (Breward, 1999). The hydrophilic 'head' prefers to be present in liquid phase whereas the hydrophobic tails prefer to stay at the air interface. When foam is sprayed onto wet aggregates, the hydrophilic heads will encourage distribution onto wet aggregate surfaces, whereas hot liquid bitumen can not distribute onto wet aggregate surfaces effectively since most of its molecules prefer to stay in the bulk phase.

Secondly, the differences in binder distribution in FAM at various mixing times have been well identified. As shown in Appendix A: Figure A.6.3 to Figure A.6.6, it is clearly observed that during mixing process foam does not fully coat and seal the aggregates, as in the case of HMA, but it distributes across the aggregate phase as combined bitumen-fine aggregate particles. Foam forms large soft mastic globules with a size of 2-3 cm at the beginning of the mixing process (e.g. at 2 seconds). These globules appear broken into smaller pieces at 5 seconds and then distribute in the mixture (at 10 seconds) and form a more homogenous foamed asphalt mixture at the end of mixing (60 seconds). It is noted that the binder mainly distributes on the fine aggregate particles and forms combined bitumen-fines clusters.

Thirdly, the formation of binder distribution has been observed at the beginning of the mixing process at various FWC values. The effect of foam volume (ERm) on the binder distribution at the beginning of mixing also needs to be understood. Unfortunately, the differences due to ERm are not clear visually (see Appendix A: Figure A.6.7 to Figure A.6.9). This means that the effect of ERm on the binder distribution does not appear to be as significant as the effect of mixing time.

Finally, the formation of binder distribution when foamed bitumen is sprayed into single size aggregate has also been observed. Initially, the aim of this observation was to assess the maximum aggregate size that would be coated by foam; it was observed that binder distribution across the aggregate phase depends upon the aggregate size (see Appendix A: Figure A.6.10 to Figure A.6.13). If foam was sprayed onto filler at high content (~ 10%), the resulting mixture appears as a uniform non-cohesive black powder. But if the binder content is low, the binder disappears and is trapped amongst the filler particles. If foam is sprayed onto larger aggregate sizes, the binder distribution becomes less homogenous.

6.2.2 Assessment of binder distribution

Binder distribution has been assessed using the particle size distribution test according to BS 812-103.1: 1985. The test procedure is as follows:

- Following mixing, the loose foamed asphalt mixture is air dried for at least 3 days,
- Foam material is then separated using dry sieving into fractions in the range from maximum size (passing 20mm) down to minimum size (passing 0.075mm).
- Each fraction is then washed using sieve no. 1.18mm and 0.075mm to allow the filler to flow out,
- The washed fractions are then oven dried at 60^{o}C before the soluble binder content test is conducted (BS EN 12697-1: 2000) to find the binder content and particle size distribution for each fraction.

The completed results can be found in Appendix B: Table B.6.1 to Table B.6.3, while the photographs of binder distribution for each fraction can be seen in Appendix A: Figure A.6.14.

Figure 6.1 shows the bitumen content and aggregate proportion for each fraction. The fractions are obtained by dry sieving; this means that fine aggregate particles are probably attached to the coarse fractions. It can be seen that the binder contents in the fine fractions (< 0.3mm) and coarse fractions (>6.3mm) are both low. Binder content is found to be high at medium fractions (0.3mm–6.3mm) and it is a maximum at 2.36mm. It should be noted that the absence of binder in the finest fractions does not mean that binder does not distribute onto fine aggregate, but that some fine particles are bonded together to form larger clusters. In the case of low binder content in coarse fractions, this means that binder is unable to distribute on them.

Figure 6.2 shows the distribution of coated and uncoated aggregates in terms of particle surface area. This result comes from wet sieving tests of all fractions, following the soluble binder content test. As an example (see Appendix B: Table B.6.2), for fraction #14 (passing sieve size 14mm) with an aggregate mass of 220.57 g, sieved down to sieve size of 0.075mm, this results in 214.8g quantities retained on sieve size 10mm (passing sieve size of 14mm) and other quantities are retained on the smaller sieve sizes. The aggregate mass from fraction #14 retained on sieve size 10mm is classified as uncoated, whereas all aggregate particles passing sieve size 10mm are classified as coated. Thus, the mass of coated and uncoated particles for each size can be determined as shown in Table B.6.2. By assuming the aggregate particles are spherical in shape and a density of 2680kg/m^3, the quantity of coated and uncoated particles can therefore be determined in terms of volume (mm^3), number of particles and particle surface area (mm^2) as shown in Table B.6.3. It can be seen that coated particles only comprise around 15% by mass or volume, but they are about 60% in terms of number of particles or 55% in terms of particle surface area. As shown in Figure 6.2, the smaller the particle size the higher the proportion coated by binder, and thus the filler size fraction is the best coated (around 80% of the coated particle surface area or 45% of the whole particle surface area). The maximum size of coated aggregate is 6.3mm. It should be noted (see Table B.6.3) that for particle sizes of 0.3mm and less, the particle surface area of coated aggregates is more than that of uncoated aggregates; for particle sizes of 0.6mm and above, the particle surface area of coated aggregates is less than that of uncoated aggregates; for particle size 10mm and above, all aggregates are uncoated. This means the binder content of large fractions i.e. fractions #10 and #14 (see Figure 6.1), is basically binder from accompanying fine aggregates.

Thus, in summary, following the mixing process the foam distributes onto fine aggregates forming larger particles of various sizes, for which the most binder rich is found to be at 2.36mm size for this sample (FWC of 4%). These binder rich 'particles' mainly consist of fine aggregates (largely filler) and the maximum coated aggregate size is 6.3mm.

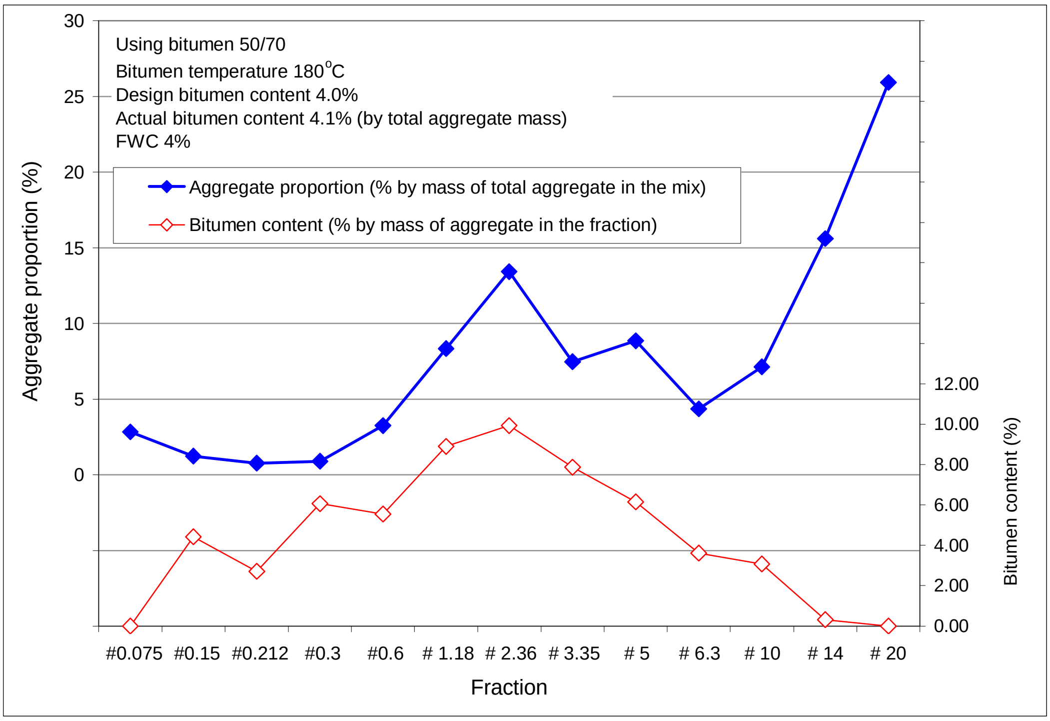


Figure 6.1 - Bitumen content and aggregate proportion for each fraction.

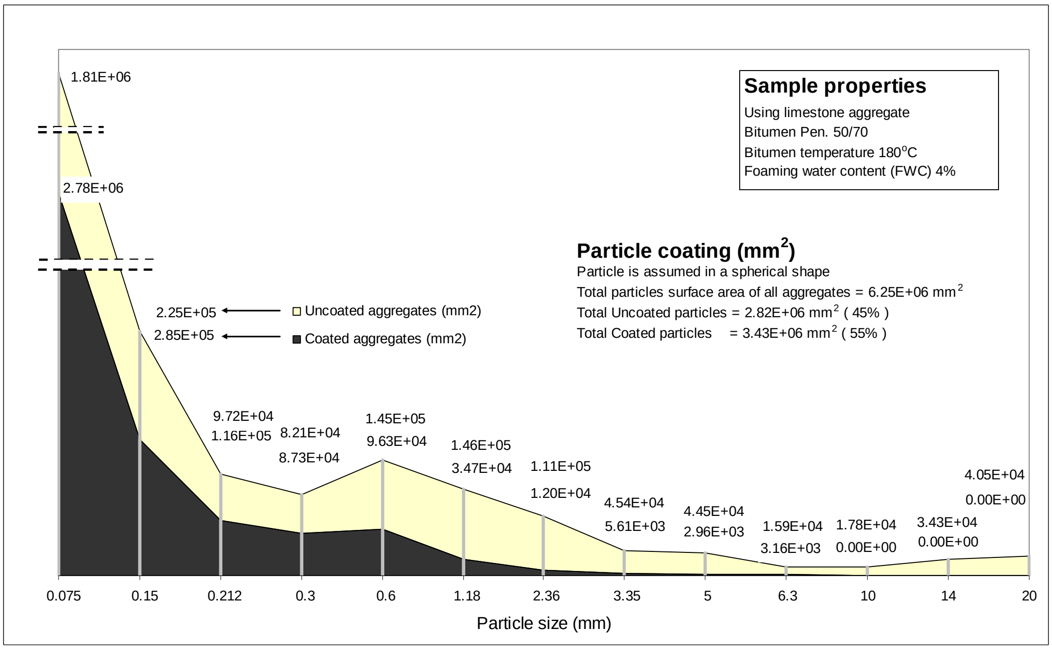


Figure 6.2 - Distribution of coated and uncoated particles in terms of particle surface area.

6.2.3 Effect of mixing protocol on Cold-Mix Properties

In section 6.2.1, the effect of mixing time on the binder distribution was observed visually. In this section the effect of mixing protocol is investigated by indirect tensile stiffness evaluation on cured cylindrical foamed asphalt specimens.

In this study, a 20 quart Hobart mixer was used to investigate the effect of mixing protocol. Three important aspects have been studied, i.e.:

- Mixing speed
- Mixer-agitator type
- Mixing time

Figure 6.3 demonstrates the effect of mixing speed on the cured specimen stiffness measured at 20^{o}C. Two mixing speeds have been investigated i.e. 107 rpm (termed low speed) and 365 rpm (termed high speed). It can be seen that mixing speed has a significant effect on mixture stiffness for both mixer-agitator types. The higher mixing speed results in higher mixture stiffness.

Figure 6.3 also shows the effect of mixer-agitator type used in the mixing process on mixture stiffness. In this case, the specimens produced using a flat agitator type resulted in higher mixture stiffness than with a spiral dough hook type, for both mixing speeds. The spiral dough hook agitator having a single spiral rod performs less well than the flat type which has a wide frame. The wide frame causes all particles to agitate simultaneously and allows the binder to distribute effectively across the aggregate surface.

All specimens shown in Figure 6.3 were mixed for one minute after foam spraying, whereas Figure 6.4 presents a comparison of mixture stiffnesses between specimens mixed for one minute and two minutes. Three types of specimen, i.e. produced at FWC of 1%, 5% and 10%, have been investigated, representing different foam properties in terms of ERm and HL characteristics. It can be seen that mixing time has a moderate effect on the mixture stiffness of specimens produced at FWC 1%, but not at FWC 5% and 10%. Mixing time appears more related to the foam’s half life rather than its expansion. A foam with longer half life has a better chance of improving its stiffness with longer mixing time.

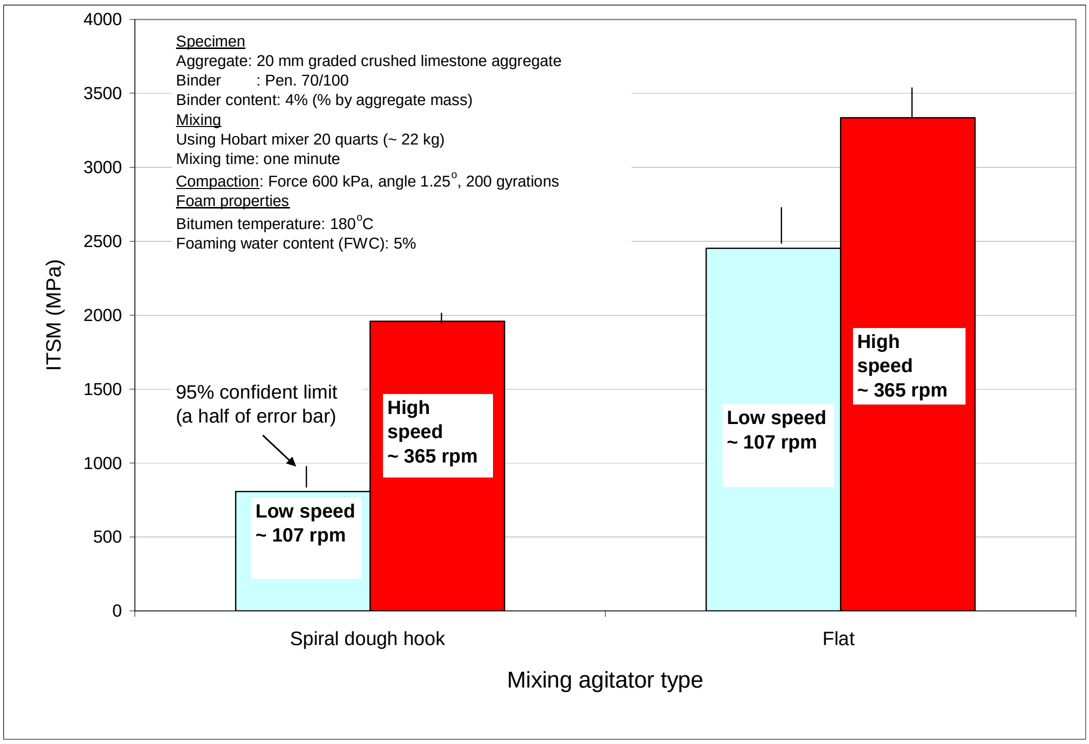


Figure 6.3 - Effect of mixer agitator type on stiffness of the mixture with low and high speed mixing

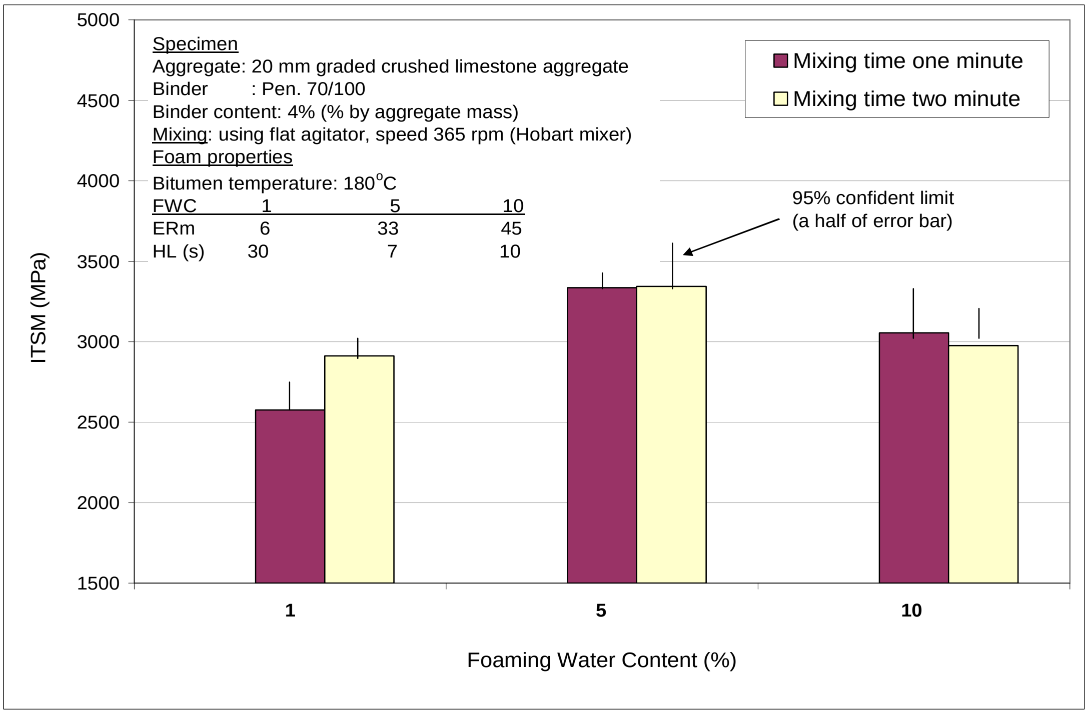


Figure 6.4 - Effect of mixing time on the mixture stiffness at different foam properties

6.2.4 Binder distribution mechanism

In foamed asphalt mixtures, the bitumen is sprayed onto wet aggregate in the form of foam at the start of the mixing process. It can be understood that both the foamed bitumen characteristics and the mixing protocol therefore become important aspects in foamed asphalt production. However, a foamed asphalt material produced using a good quality foam will not exhibit high performance if it is mixed using a poor mixer.

As discussed previously, it is clear that following the mixing process, the loose foamed asphalt material consists of two particle types, i.e. aggregate and combined bitumen-fines particles. About 15% of the aggregate mass was found to be coated by binder. The maximum coated particle size is 6.3mm and the maximum binder-fines particle size is found to be 14mm. In this circumstance, the number (and size) of binder-fines particles becomes crucial since it influences binder distribution. A greater number of binder-fines particles or smaller sized binder-fines particles results in better binder distribution, which stimulates the compaction process and hence potentially enhances the mixture properties. Thus, the mechanism of binder distribution needs to be well understood.

Basically, foam is created immediately following injection of pressurised air and water into the hot bitumen phase in the expansion chamber. The number of foam bubbles may grow rapidly under high pressure. When foam is sprayed out into a steel cylinder, the growth in bubble size can be seen clearly due to the internal pressure being higher than the surrounding pressure.

Therefore, it can be understood that the hot bitumen has changed, becoming a foam which is directly sprayed into the wet aggregate mixture. This means the key bitumen properties have changed from bulk properties to surface properties, allowing the foam to distribute onto the wet surface (interface between air and liquid). When foam is in contact with the moist aggregate, it has no chance to grow, as evidenced in the measuring cylinder due to its temperature immediately reducing to below 100^{o}C (see Figure 4.10). Steam gas inside the bubbles may return to the water phase.

However there are still large numbers of tiny voids trapped inside the bitumen as assessed in density tests of collapsed foam (see Figure 4.42). This void content is found to be higher for foam generated at higher FWC. The presence of these voids causes the collapsed foam to remain soft and workable. It is supposed that during mixing, the binder properties are proportional to those of foam when investigated in the measuring cylinder (in terms of ERm, HL or flow behaviour).

As discussed in section 6.2.3, the mixing speed and agitator type have a significant effect on mixture stiffness since the mixer helps the foam to spread onto the wet aggregate surface. Mixer speed has to be high enough to cope with the spraying rate of the foam. As a simple analogy, the work of the mixer can be imagined as a conveyor. If aggregates are put onto a conveyor and sprayed with foam, a higher speed of conveyor and a larger volume of foam will result in more aggregate surface area being sprayed. Because foams collapse soon after contact with moist aggregates the mixer speed and foam volume over the contact period are very important. During this period, the agitator is used to agitate the aggregate particles so that they can be sprayed by foam as effectively as possible.

In fact, foam prefers to distribute onto a wet surface (air-liquid interface) due to the presence of amphiphilic surfactant molecules on the bubble lamellae. Foam therefore prefers to distribute on the water phase, which is mainly present amongst fine particles due to suction effects. When foam contacts the moist fine particles, it collapses immediately due to rapid temperature reduction and it therefore forms mastic globules. These mastic globules are still warm and soft with a temperature of between 90^{o}C and 61^{o}C (see Figure 4.10). This condition allows the mixer agitator to cut ‘the warm mastic globules’ to form smaller particles and then distribute them throughout the aggregate phase to obtain a more homogeneous mixture. It can be understood that the cutting process will mainly depend on mixing speed, agitator type and the workability of the mastic globules. It is supposed that the workability of these globules is mainly affected by binder properties such as viscosity, that depends on the rate at which the globules cool down during the mixing process. The rate of cooling down of foam may be quicker at higher FWC applications. The larger foam volume results in greater foam surface area and hence the bitumen films of the bubbles are thinner. This implies that foams with larger volume will cool down quicker when their surface contacts the wet aggregate.

The process of cutting the warm mastic globules by mixer results in a more homogeneous mixture. When mixing is applied far longer, it is then logical that the mastic particles become smaller and hence the mixture will be much more homogeneous. However, a longer mixing time will not affect the binder distribution in the mixture once the mastic particles have returned to a cool condition. It is noted that the higher expansion foam (achieved at higher FWC) will cool down faster than lower expansion foam.

It is clear that during the mixing process the properties of the binder-fines particles are important. These properties represent the ability of the binder to be broken by mixer and to distribute across the aggregate phase. Unfortunately, this study has not investigated binder particle-fines properties directly. However, binder-fines particle properties are expected to be proportionally linked to foamed bitumen characteristics, so it is still reasonable to link foamed bitumen characteristics and mixture properties.

6.3 Correlation between foamed bitumen characteristics and mixture properties

6.3.1 Relationship between ERm/HL/FI and ITSM values

As discussed in Chapter 5, the effect of foamed bitumen properties (with different FWC values) on mix performance can only be clearly defined using well mixed specimens, based on stiffness evaluation. In this section, the relationship between parameters of foamed bitumen characteristics such as the maximum expansion ratio (ERm) and half-life (HL), and mixture properties is analysed. The foam index (FI) value, as another important parameter derived from the foam decay curve, is also included. As discussed in Chapter 4, these parameters are dependent on foaming water content (FWC), binder type and bitumen temperature, and the effect of bitumen temperature was found to be less than that of the two others.

Figure 6.5 shows the relationship between ERm and ITSM values at test temperatures of 5°C and 20°C for Pen 50/70 and Pen 70/100 binders. This figure is generated from Figure 4.15 (Chapter 4) and Figure 5.17, Figure 5.18 and Figure 5.21 (Chapter 5).The test temperature of 40°C and bitumen Pen 160/220 are excluded in this figure since they show insignificant effects from ERm on ITSM values (see Figure 5.18 and Figure 5.25 in Chapter 5). The effect of binder properties on the ITSM value reduces with increasing test temperature, and hence, as shown in Figure 6.5, the effect of ERm at a test temperature of 5°C appears more clearly defined than at 20°C. It seems that binder distribution is important for stiffness value at low temperature. As shown in Figure 5.15, foamed asphalt and HMA specimens with equal composition and density exhibit comparable ITSM values at test temperatures of 20°C and 40°C, but at 5°C foamed asphalt specimens have lower stiffness, indicating the lack of binder distribution compared to HMA specimen.

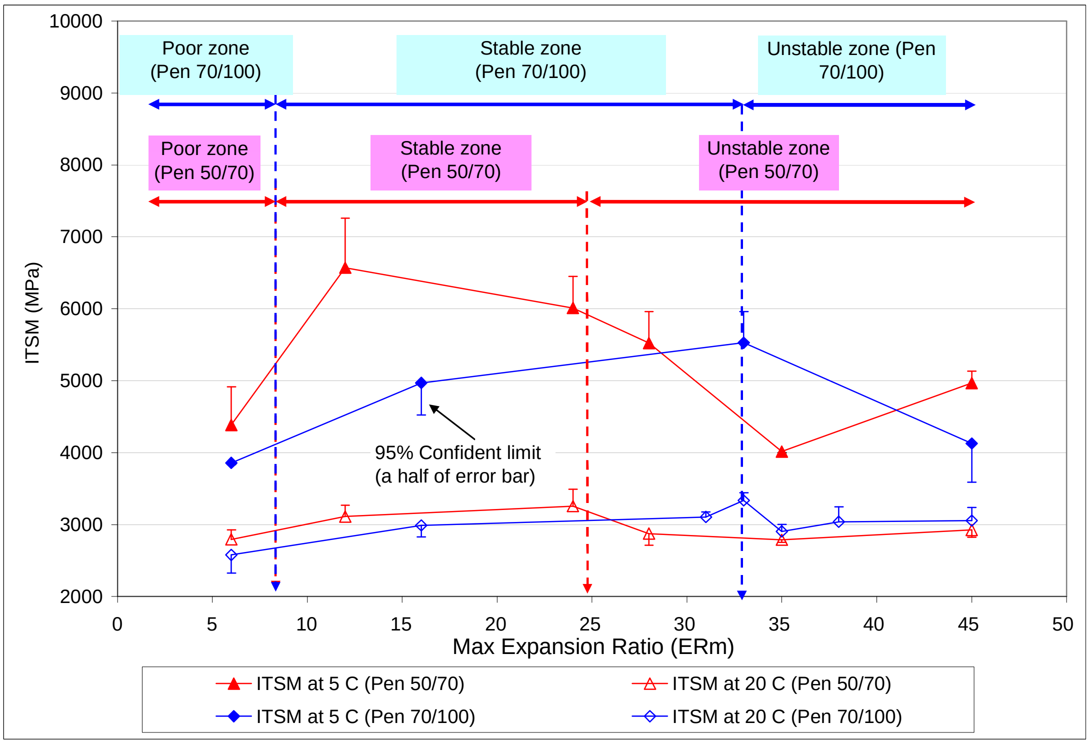


Figure 6.5 - Relationship between maximum expansion ratio (ERm) and cured ITSM values at test temperatures of 5°C and 20°C for bitumen Pen 50/70 and Pen 70/100.

Based upon Figure 6.5, three important zones can be observed as follows:

- A poor zone, in which the stiffness values are the lowest,
- A stable zone, in which the stiffness values are more consistent and higher than in other zones,
- An unstable zone, in which the stiffness values are variable.

As shown in Figure 6.5, at a particular ERm value, the stiffness values for both binder types are different and hence their zone ranges vary. ITSM values at an ERm of around 6 are usually poor for both binders. Binder Pen 70/100 has a wider stable zone than Pen 50/70, indicating that binder grade has a significant effect on mixture stiffness. For the case of a very soft binder (e.g. bitumen Pen 160/220), it was found that ERm had no significant effect on the value of the resultant ITSM.

Figure 6.6 shows the ITSM values of foamed asphalt cured specimens at various half-life (HL) values. It can be seen that at a very short HL (about 5 seconds) the ITSM values can still be high but at longer HL values the ITSM values are generally lower. It is supposed that the effect of HL is only supplementary to the ERm effect in that, at a particular ERm value, a longer HL will give a better mixture performance. The longer HL values are also found to give a benefit in terms of ITSM values if the foam materials are mixed for a longer time as discussed in section 6.2.3. Therefore the concept of foam index (FI) should logically be considered, since it represents both ERm and HL values. As discussed in section 4.6.2 (Chapter 4), the effect of ERm is greater than that of HL in developing the FI value. As shown in Figure 6.7, the FI value required to achieve the maximum ITSM with binder Pen 70/100 is higher than with binder Pen 50/70. This is in line with the ERm effect, for which the optimum ERm for bitumen Pen 70/100 is higher than for bitumen Pen 50/70 (see Figure 6.5).

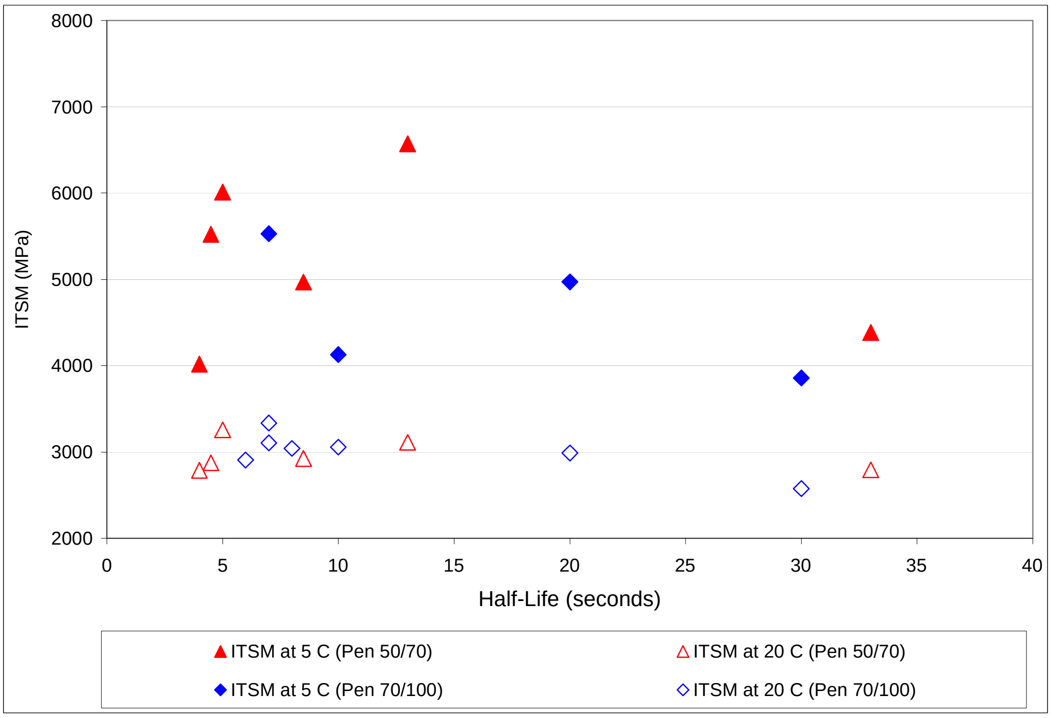


Figure 6.6 - ITSM values at test temperatures of 5°C and 20°C for bitumen Pen 50/70 and Pen 70/100 at various half-life (HL) values.

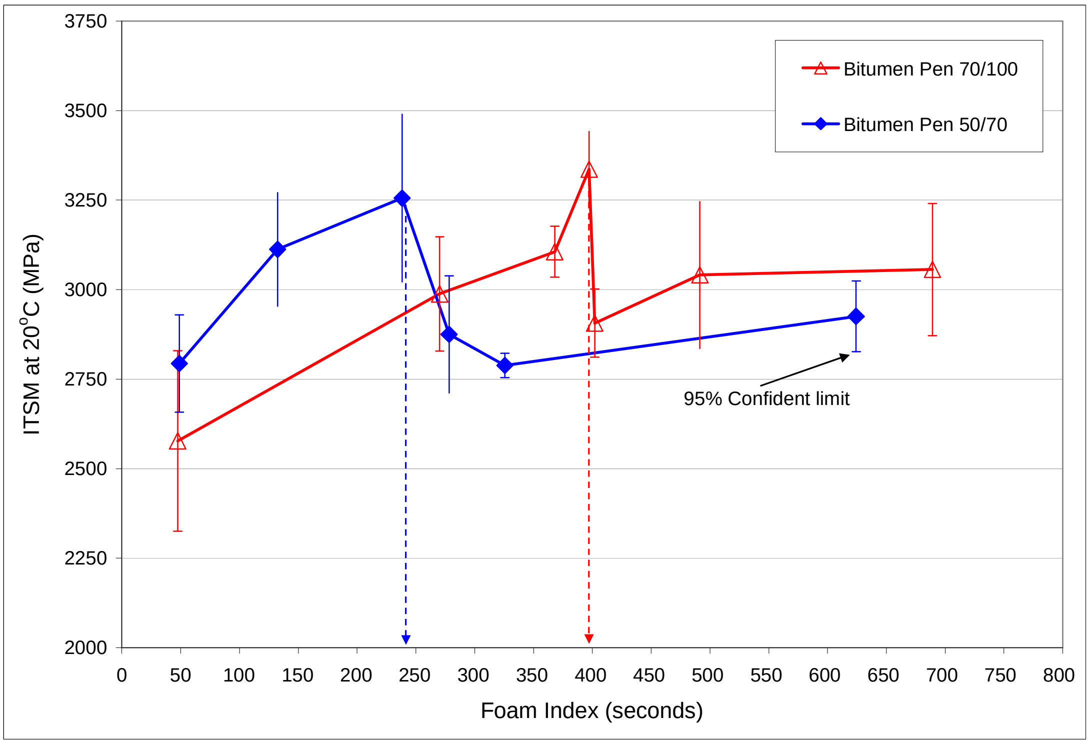


Figure 6.7 - Relationship between Foam Index (FI) and cured ITSM values at test temperatures of 20°C for bitumen Pen 50/70 and Pen 70/100.

6.3.2 Combination effect of ERm and apparent viscosity of foam on the mixture performance

Referring to the binder distribution mechanism, foam with higher volume potentially allows the binder to be distributed across a greater aggregates area. It is therefore logical to say that a higher ERm or FI will result in better binder distribution and hence will improve the stiffness. However, this prediction is not valid in fact, as shown in Figure 6.5 and Figure 6.7. The beneficial effect of ERm or FI on mixture stiffness appears to be present only up to a certain ERm or FI value, and especially in the unstable zone their effects become unclear. This indicates that neither ERm nor FI can be seen as the single major parameter. It has been remarked that foam viscosity should be potentially considered as another important parameter. There was some evidence during the mixing process that the Hobart mixer appeared to have slightly more difficulty in mixing foamed material generated using higher FWC. This implies that at such a foam volume (or ERm) the foam is too stiff to be mixed effectively, resulting in lack of distribution of binder across the aggregate surface. Moreover, the finding that specimens produced using binder Pen 160/220 mix better than other specimens produced using harder binder, indicates that the viscosity significantly affects the mixing performance (see section 5.8 of Chapter 5). It is noted that foam viscosity is assumed to be affected by binder viscosity (see Eq. 4.12 in Chapter 4). Foam viscosity is also found to cause specimens to have different ITSM values even where they have the same ERm or FI values (see Figure 6.5 and Figure 6.7). The effect of foam viscosity caused by binder grade is clearly observed in the resultant mixture stiffnesses since the difference of binder grade is reflected in a large difference in viscosity. However, this is not the case for different foam viscosities generated by different gas contents of foam for one type binder.

Referring to Figure 4.38 (Chapter 4), it can be seen that the estimated apparent viscosity (using bitumen Pen 70/100) increases slightly with increasing ERm values up to around 30, and then increases dramatically from an ERm of around 35, and finally remains constant up to an ERm value of 45. With reference to the Kraynik theory (see Eq. 4.12), foam viscosity affected by binder viscosity is found to be controlled by foam temperature. A reduction of temperature from around 120^{o}C to

100^{o}C (a function of the amount of water added) results a dramatic change of binder viscosity (depending on the binder type). It should be noted that at a FWC of 7% to 10%, the resultant foam temperature is constant, i.e. 98^{o}C, and hence the foam viscosity also remains stable in this range (between ERm of 36 to 45) as shown in Figure 4.38. It is noted that the gas content has a multiplying effect on the apparent foam viscosity; it means that as gas content approaches 100%, foam viscosity is almost double that of binder viscosity. This correlation between ERm and apparent foam viscosity will vary with various bitumen temperatures and binder types.

6.3.3 Limits of ERm value for each zone

ITSM values in the unstable zone are variable. This zone is indicated by a decreasing ITSM value soon after achieving maximum value (at FWC of 5% or ERm of 28 for bitumen Pen 50/70 and FWC of 6% or ERm of 35 for bitumen Pen 70/100) and then it gradually increases up to a FWC of 10% or ERm around 45 (ITSM at 20^{o}C). This variation of ITSM values is strongly predicted to have a close correlation with foam viscosity as discussed above. When the ITSM value drops after its maximum value, this is caused by a dramatic increase in foam viscosity. When the ITSM values gradually increase again up to a FWC of 10% (or ERm around 45), it is caused by the foam viscosity being stable in this range and hence giving an opportunity for the ERm value to enhance the stiffness. The drop in mixture performance is also found in the results of RLAT testing as shown in Figure 5.29 (using FB 70/100 at 180^{o}C). The axial strain rate over the second 1800 cycles is found to be highest at a FWC of 6%, indicating the worst resistance to permanent deformation where the foam viscosity increases sharply. Additionally it should be noted that at high ERm values (gas content > 96%), foam is also found to be unstable (see Figure 4.2), and this may be a factor affecting mixture performance in the unstable zone.

In the stable zone, the ITSM values are relatively constant and higher than in other zones. The highest ITSM value is usually found in this zone, indicating the optimum foam properties. In this zone, the foam viscosity is far lower than in unstable zone, and it gradually increases with increasing ERm values. It is therefore reasonable that the ITSM values in this zone are mainly affected by the ERm value. It should be noted that for specimens using lower bitumen grade (e.g. Pen 50/70), the optimum properties are potentially achieved at a lower ERm since foam viscosity is higher than with softer binders. It is also noticeable that, in the stable zone range, the foam is dry and in a stable condition (see Figure 4.2).

In the poor zone, the ITSM values are usually lowest and this occurs at a FWC of 1% (or ERm around 6) or less. However, as shown in Figure 5.17 and Figure 5.21 in Chapter 5, increasing the FWC from 1% to 2% results in a significant increase in ITSM values. This reflects a doubling of the ERm value from FWC of 1% to 2%, a much greater rate of increase than is seen at higher FWC (see section 4.5.1). Referring on the observation that hot bitumen (i.e. FWC of 0%) could not be mixed with wet aggregates effectively (see section 6.2.1), this indicates that at an ERm value between FWC of 0% and 1%, foamed bitumen starts to be able to distribute onto wet aggregate surfaces effectively. This should relate to the onset of bubble motion in wet foam (see Figure 4.2) which is found at around 52% foam quality or equal to ERm of around 2-3. Jenkins (1999) proposed a minimum ERm value of 4 in terms of FI calculation. It can be deduced that the poor zone of ERm relates to a wet foam structure (foam quality between 52% and 87%). The presence of liquid hot bitumen in the wet foam structure may reduce the ability of foam to distribute onto wet aggregate surfaces. It is therefore deduced that the poor zone is in the range of ERm about 3 and 8 (about 52% and 87% gas content).

6.3.4 Effect of foamed bitumen properties on the resistance to water damage

As discussed in Chapter 5, two points have been identified, i.e. specimen damage and ageing, in interpreting the result of water sensitivity test on the foamed asphalt material. It can be understood that water will infiltrate more easily into specimens with poorer binder distribution. On the other hand, foam with higher ERm will have more potential to undergo ageing after coating the aggregates. It is noted that the binder was found not to have aged due to the foaming process (see Table 4.6), but ageing was found to have occurred in samples of recovered binder from cured specimens (see Figure 5.23). The fact that the complex modulus of the recovered binder at FWC of 1% is higher than at FWC of 2% and 4% as shown in Figure 5.23

is not fully understood. It may be that ageing occurs more readily due to water being able to infiltrate more easily.

In general, the ITSM ratio of specimens using bitumen Pen 50/70 is found to be lower than that of specimens using Pen 160/220 (see Figure 5.22 and 5.26). This is because binder distribution in the specimens using Pen 160/220 is better than in specimens using bitumen Pen 50/70. Binder ageing in FAMs is probably less occurring since the binder is not fully distributed (form thicker bitumen film). The internal stripping is probably more likely occurring.

The effect of FWC (or ERm) on the water sensitivity is clearly evident in the specimens using bitumen Pen 50/70. At a FWC of 1%, the ITSM ratio is lowest due to specimen damage. The ITSM ratio exhibits an increase with the FWC, possibly indicating that binder ageing occurs more readily at higher ERm or FWC values.

6.4 Theory Consideration of Stiffness for Foamed Asphalt Mixture

Figure 6.8 illustrates the binder distribution across the aggregate phase in a mixture. The binder is in the form of combined particles of bitumen and fine aggregate as described previously. A more homogenous mixture implies better binder distribution, i.e. smaller size of binder-fines particle, distributed uniformly across the various aggregate sizes. With better binder distribution there will be higher mixture stiffness.

In a compacted mixture, the formation of combined binder-fines and uncoated particles leads to some aggregate particles contacting others without binder as shown in Figure 6.8. Referring to Thom and Airey (2006) (see section 2.2.3), the absence of binder at the point of contact between aggregate particles will result an inter-particle movement within the aggregate and hence give a low modulus. If the position of binder is between contact areas of aggregate particles as shown in the figure, the binder will resist inter-particle motion and hence potentially increase stiffness. It is a logical assumption that a good binder distribution will give a chance for binder-fines particles to locate in useful positions.

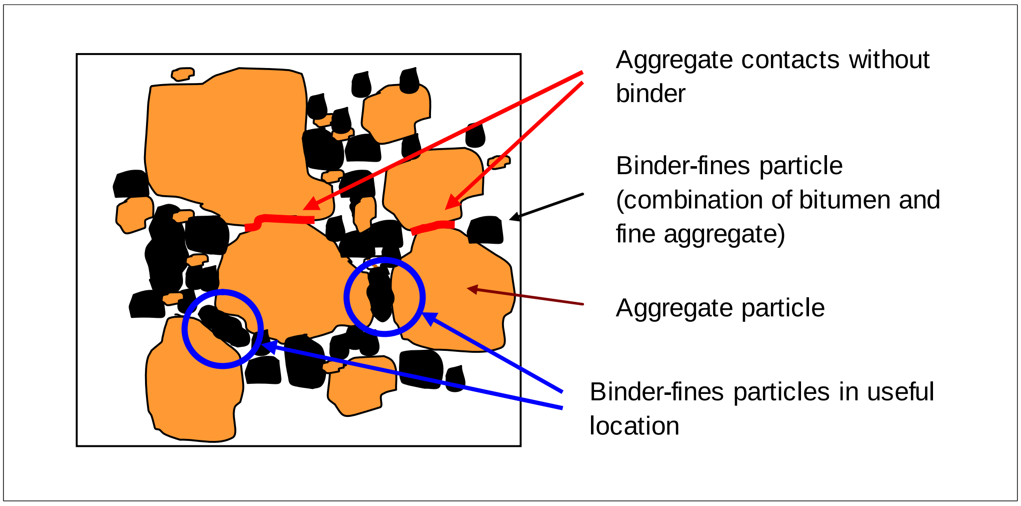


Figure 6.8 - Binder distribution in foamed asphalt mixture

The effect of binder distribution on mixture stiffness may also be analysed based on Brown and Brunton (1986). In this approach, mixture stiffness can be predicted analytically from bitumen stiffness and the VMA (voids in mixed aggregate) value (see Figure 2.9). It can be seen that if the value of VMA decreases the mixture stiffness will increase. VMA is defined by bitumen volume (Vb) plus void content (Vv). The total volume of the mixture is Vb+Vv+Va, in which Va is the volume of aggregate. Both Vb and Va are parameters that depend upon mass proportion and specific gravity, neither of which will change with binder distribution. Thus, only void volume (Vv) will be affected by binder distribution variation. This leaves two possibilities why mixture stiffness might be affected by binder distribution: better binder distribution results in better density and hence reduces void content and therefore increases stiffness – following Brown and Brunton (1986); alternatively, better void distribution across the mixture may have a direct effect even without any change in Vv.

6.5 Conclusions

The relationship between foamed bitumen properties and mixture performance has been well explored in this chapter. It is important to note that foam properties are not the major influence on mixture performance. Mixing protocol and binder type are found to have a more dominant effect than foam properties. Consequently, the effect of foam properties is only clearly defined in well mixed specimens, based on stiffness evaluation. The effect of foam properties is also found to vary with binder type. Additionally, bitumen temperature has been observed to be a minor parameter in affecting mixture performance.

The key issue in understanding foamed asphalt properties is that of binder distribution. A uniform mixture in terms of binder distribution will achieve high performance. Binder distribution is a result of mixing process. It is therefore the mixing process that is the most important factor in foamed asphalt production (in combination with good foam properties). The mechanism of binder distribution, which has to be well understood in order to study the effect of foam characteristics, has been fully described. It is found that the binder formation is that of a combination of bitumen and fine aggregate, forming particles that distribute across the aggregate phase. The properties of these binder-fines particles become crucial during mixing process. The workability of binder-fines particles influences the uniformity of the binder distribution. Although this study has not directly investigated this binder formation, it is reasonable to assume that binder-fines particle properties are proportional to those of the foamed bitumen. The fact that the ITSM over various FWC values are closely related to foamed bitumen properties such as ERm, apparent viscosity of foam and foam structure (wet or dry foam; stable or unstable foam) strongly supports that assumption.

The ITSM over various FWC values was found to be affected by a combination of ERm and apparent viscosity of foam. The trend of ITSM values has been identified using those two parameters and three zones of ERm values are proposed, namely a poor zone, a stable zone and an unstable zone. In fact these three zones can be linked to that of foam structure. The poor zone relates to wet foam structure, the stable zone to stable dry foam structure and the unstable zone to unstable dry foam structure. It is noted that the variation of ITSM values in the unstable zone is clearly affected by the apparent viscosity of foam. It should be understood that values of the estimated apparent viscosity may not have been rigorously determined, but their trend over the range of ERm values is useful in identifying the trend of ITSM over various ERm values.

The relationship between ERm and ITSM can be utilised to determine the optimum ERm or at least to understand the stable zone which results in optimum mixture performance. Generally, the dry side of the optimum ERm is controlled by the ERm value and the wet side is controlled by foam viscosity. The ERm limits between the three zones can also be approximated, i.e. in the poor zone it is between 3 and 8, in the stable zone it is between 8 and 25 (for binder Pen 50/70) or between 8 and 33 (for binder Pen 70/100), and beyond 25 (for binder Pen 50/70) or 33 (for binder Pen 70/100) is the unstable zone. However, these limits will vary for different binder types (a major effect) and bitumen temperatures (a minor effect), especially the limit between the stable and unstable zones. It is supposed that use of harder binder or lower bitumen temperature tends to produce a shorter stable zone. In this study, material using bitumen Pen 160/200 was found to produce no significant variation in ITSM over various ERm values and hence the zone categories could not be defined as clearly as for other grades.

The way that foamed asphalt develops its stiffness has been identified. Referring to Thom and Airey (2006), the absence of binder at certain points of contact between aggregate particles (due to poor binder distribution) will result in inter-particle movement within the aggregate and hence will give a low modulus. Conversely, if the binder is present, it will resist inter-particle motion and hence potentially increase stiffness. Alternatively, with reference to Brown and Brunton (1986), a better binder distribution may reduce the void volume and hence increase the stiffness.

7 PRACTICAL GUIDANCE TO PRODUCE AN OPTIMISED FOAMED ASPHALT MIXTURE (FAM)

7.1 Introduction

The pattern of the aggregate-binder structure of a foamed asphalt mixture (FAM) differs from that of a hot mix asphalt (HMA) mixture. HMA is a fully bonded material in which all aggregate surfaces are coated by binder. In FAM, not all aggregate particles are coated by binder.

This study has identified the coating mechanism of aggregate-binder in FAM. Foamed bitumen prefers to distribute in the water phase only, which is mainly present amongst fine aggregate particles. It is therefore reasonable that, following the mixing process, foam only coats fine particles and that they form composite binder-fines particles, a combination of binder and fine particles. It is found that these binder-fines particles contain around 15% of the total aggregate mass (for FB 50/70: FWC 4%-180^{o}C) and it is predicted that these particles will be at a temperature of 50^{o}C or higher during mixing process. These binder-fines particles distribute across the uncoated aggregate phase. The uniformity of binder-fines particle distribution was deduced to be affected by mixing protocol and binder properties. Following the compaction process, it is possible that the point of contact between two aggregate particles is accompanied by either the presence or the absence of binder-fines particles. The absence of binder will result an inter-particle movement within the aggregate and hence give a low stiffness modulus. Therefore the stiffness modulus of FAM is controlled by the uniformity of binder particle distribution.

It is clear that binder distribution plays a key role in achieving optimum FAM performance. A better binder distribution will result in a better mixture performance. Since the uniformity of binder distribution is a result of the mixing process, this step therefore becomes a very important one in FAM production. This study has successfully identified the critical effect of mixing protocol on the mixture stiffness.

A clear correlation between foamed bitumen properties (binder type, ERm, HL and viscosity) and mixture stiffness is also identified.

This chapter is aimed at proposing a practical guidance for selecting foamed bitumen properties in order to achieve an optimised FAM. The parameters considered are (a) mixing protocol, (b) binder type, (c) maximum expansion ratio (ERm) and (d) foaming water content (FWC) and bitumen temperature. These parameters are considered since they are easily measured/ defined and controlled during foam production. FWC and bitumen temperature are found to be main parameters in affecting foam viscosity, whereas foam viscosity and ERm combine to influence the FAM stiffness.

7.2 Main considerations to achieve an optimum foamed asphalt performance

7.2.1 Mixing protocol

Mixing protocol is found to be a major parameter affecting FAM performance. If a FAM has been designed properly but is not mixed using a good mixer, it can not be guaranteed to produce a uniform binder distribution. Three aspects have been identified that affect the mixing performance, i.e. mixing speed, agitator type and mixing time.

In the laboratory, a Hobart mixer designed to be mounted onto the foaming plant (Wirtgen WLB 10) is commonly used to mix FAM material. Pan mixers commonly used for laboratory mixing of HMA materials are not recommended due to their speed being too low. The two principles governing mixer suitability for FAM are (firstly) that the mixing speed should be fast enough to cope with the rate of foam spraying, enabling aggregate particles to distribute as thoroughly as possible while the binder is still workable, and (secondly) that the mixer agitator should enable as much aggregate agitation as possible to expose aggregate surfaces to foam spraying and to cut the binder-fine particles into suitably small size. A high-speed twin-shaft pugmill mixer (such as that developed by CSIR Transportek in South Africa), having a high speed and very efficient agitator, is probably the most suitable for FAM material.

In this study, using a Hobart mixer, it was found that applying higher mixing speed resulted in a better mixture performance, and hence it was recommended to apply the highest speed level, i.e. level 3 with a speed around 365 rpm. The type of mixer agitator was also found to affect the mixture performance significantly, for which a flat agitator performed better than a dough hook, aiding binder distribution and enhancing the mixture stiffness. It was found that the size and mass of aggregate used also influence the mixing quality. It is recommended to use a maximum aggregate size less than any gap in the structure of the agitator or between the agitator and the sides of the mixer bowl. The mass of aggregate for one batch should be limited so as to allow the mixer to work properly.

Mixing time is another important factor in the mixing protocol. In this study, material was mixed with foam for one minute, which is the same order of magnitude as the foam life. It was found that applying a longer mixing time (i.e. 2 minutes) can only improve mixture stiffness for foam with a high half-life value (e.g. at FWC of 1%-2%).

Mixer specification should also consider the type of binder used. FAM using a hard bitumen (low penetration) requires a high power/speed mixer. Logically, a poor mixer for FAM with low bitumen pen will produce poor binder distribution, and conversely, a FAM with very high bitumen pen may not require a mixing speed as high as with a low bitumen pen. If the mixer quality is inadequate or the binder viscosity is too low, the expansion of the foam is a less significant factor in its effect on binder distribution.

In the field (in in-situ cases), mixer power and capacity and depth of pavement to be recycled are of prime importance in producing a more homogenous binder distribution in the mixture. Clearly, a shallow depth of recycled pavement will reduce the required mixer power, and hence it is suggested that when using low bitumen pen and a thick recycled layer, the mixer power and capacity should be as high as possible. If the mixer power and capacity is limited, it is better to use a higher bitumen pen, thinner pavement layer, and possibly, lower ERm (in the range of stable values).

In summary, it is strongly recommended to use as good a mixer as possible in terms of its speed and agitator type for any purpose in FAM production. However, if an appropriate mixer is not available, the following suggestions may be useful:

- Use high bitumen pen (e.g. Pen 160/220),
- Apply high bitumen temperature (e.g. 160^{o}C-180^{o}C),
- Apply low FWC (e.g. 1%-1.5%) or low ERm (e.g. 5-10),
- Use longer mixing time (e.g. 1-2 minutes),
- Use reduced batching mass (to be judged considering the mixer capacity),
- Use small maximum aggregate size (e.g. 10mm),
- Apply to a thin layer of recycled pavement (field in-situ case only).

7.2.2 Binder type

The considerations in selecting binder type for FAM are basically similar to those for common bituminous materials, namely traffic load type (heavy or light traffic) and regional climate, for which a heavily trafficked road or a road in a hot region requires a harder binder. However, in FAM, binder type also significantly affects the distribution of binder-fines particles during the mixing process, which influences the mixture stiffness. Thus, the effect of binder type on binder distribution should be combined with its effect on mixture performance when considering binder type selection for FAM.

Binder type influences viscosity and stiffness modulus. Low bitumen pen will normally have high viscosity and high stiffness modulus. Thus, a FAM using low bitumen pen will be negatively affected during mixing since binder with high viscosity reduces workability, but there will be a positive effect on mixture stiffness due to the high stiffness of the bitumen. For a FAM using high bitumen pen, the situation will be reversed. Thus, when selecting binder type for FAM material two things should be considered, i.e. mixer capability and regional climate. A FAM using low bitumen pen should be mixed using a high speed mixer to ensure good binder distribution; if a suitable mixer is not available, it is better to use a higher bitumen pen. On the other hand, if a FAM is to be paved in a cold region, a softer binder will be better since binder distribution is more important in developing mixture stiffness at low temperature; however in a hot region, a harder binder is more suitable since binder modulus is more important in developing mixture stiffness at high temperature.

Since binder distribution in FAM is also affected by the maximum expansion ratio (ERm) of foam which is influenced by binder type, this means that binder type should also be considered in the mixing process in a second way. In order to produce high expansion foam, it is suggested that high temperature (160^{o}C to 180^{o}C) is applied for low bitumen pen (Pen 50/70) and low temperature (140^{o}C to 160^{o}C) for high bitumen pen (Pen 160/220). However the effect of bitumen temperature on the binder/foam viscosity should also be considered.

A summary of these considerations in selecting binder type to produce optimum mixture performance of FAM can be seen in Table 7.1. Bitumen Pen 70/100 is a moderate grade binder that can be used in almost any climatic condition. The most appropriate bitumen temperature for this bitumen type is not defined since the effect on ERm showed no clear trend.

Table 7.1 - Considerations in selecting binder type for FAM material

Binder type	Bitumen temperature	Mixer	Climate
Bitumen Pen 50/70	high temperature (160^{o}C to 180^{o}C)	Good mixer (highly recommended)	Hot region
Bitumen Pen 70/100	No clear trend	Good mixer (highly recommended)	Most climates
Bitumen Pen 160/220	low temperature (140^{o}C-160^{o}C)	If good mixer is not available, a lower speed mixer can be used with higher bitumen temperature	Cold region

7.2.3 Maximum Expansion Ratio (ERm)

Basically, both ERm and HL have an effect on FAM stiffness, but the effect of HL is only supplementary to the ERm effect. It means, as an example, that if two kinds of foams have similar ERm but different HL, the foam with higher HL will tend to produce a better mixture stiffness. Actually, the FI concept is more representative of foam volume than ERm since FI accommodates both ERm and HL. However, the effect of ERm and FI on the mixture stiffness is found to be approximately proportional, and hence the use of ERm is simpler for practical purpose. It was found in this study, also reported by many researchers, that measurement of HL is less valid than ERm. For these reasons, this guidance will be in terms of ERm.

With regard to foam structure, wet foam is found to be in the range between 52% and 87% foam quality, stable dry foam is in the range between 87% and 96%, and beyond 96% the foam becomes unstable. These categories are found to matching ERm zones in terms of their correlation with mixture stiffness (ITSM), i.e. a poor zone, a stable zone and an unstable zone. It is known that the ERm value always tends to increase with FWC, however the ITSM value tends to follow the ERm in the poor and stable zones only; it exhibits unpredictability in the unstable zone (at high FWC). It can therefore be deduced that, in the poor and stable zones, FAM performance is significantly affected by the ERm value.

Following the above description, the limits of the ERm zones that give good mixture performance can be estimated from the ranges of foam quality. Two limit points can be identified as follows:

- Minimum ERm value to produce an effective mixture,
- Minimum ERm value to produce a stable mixture performance.

The first minimum ERm value means that foamed bitumen starts to be able to mix effectively with cold wet aggregate, although the resulting mixture may not have stable performance. If the ERm is less than this value, the resulting mixture will be uncombined asphalt, as when hot bitumen is sprayed onto wet aggregate. This limit is estimated to represent the start of wet foam structure, i.e. at a gas content of around 52% which is equal to an ERm of around 2-3. This value should probably be increased to 5 to give a suitable safety factor for practical purposes. It is therefore recommended that a minimum ERm value of 5 (80% gas content) is used to produce an effective FAM. This value is normally close to that of a foamed bitumen produced at FWC of 1%, depending on binder type and bitumen temperature.

The second limit represents the minimum value of ERm at which foamed bitumen can distribute uniformly onto wet aggregate and hence it results in an optimum mixture performance. This limit is estimated to be at the start of the stable dry foam region, i.e. at a gas content of around 87.5%, which is equal to a ERm of about 8. Applying a safety factor, this study recommends an ERm value of 10 (gas content of 90%) as a minimum. It should be understood that this value may not be exact due to differences in mixing effectiveness. As an example, a foam with ERm of 10 may produce optimum mixture performance when thorough mixing is applied (e.g. good mixer, low batching mass or small maximum aggregate size); however if poorer mixing is applied, it is possible that use of an ERm of 8, for example, may produce a similar or even better mixing performance. This value (ERm of 10) can normally be obtained from a foamed bitumen produced at FWC less than 2% (typically around 1.5%), depending on binder type and bitumen temperature.

Table 7.2 provides a summary of recommended minimum ERm limit values to produce stable mixture performance using three binder types.

Table 7.2 - The minimum ERm limit to produce stable mixture performance

Binder type	Minimum limit	Note
Pen 50/70	ERm= 10	ERm of 5 is considered as the start of wet foam structure (80% gas content), whereas ERm of 10 is the start of the stable dry foam that comprises 90% gas content
Pen 70/100	ERm= 10	
Pen 160/220	ERm= 5	

7.2.4 Foaming water content (FWC) and bitumen temperature

As discussed previously, it was observed that the ERm effect was inconsistent in the unstable region and foam viscosity was found to be another parameter affecting the mixture stiffness in this region. It is supposed that in the unstable region foam viscosity increase significantly with FWC and counteracts the ERm effect. This is in line with the results of the foam flow rate investigation and supported by the theory of general foam, in which the apparent foam viscosity increases with gas content.

Referring to the Kraynik equation, it is known that foam viscosity also increases with increasing liquid bitumen viscosity and gas content (ERm). The bitumen viscosity can be estimated based on foam temperature, which can be predicted using the heat energy transfer equation for every FWC application. Therefore, with increasing FWC, a reduction of foam temperature and an increase of bitumen viscosity, ERm (and therefore foam viscosity) are expected. It should be understood that the estimated foam viscosity may not be very precisely determined, but the comparison between values and the change of this viscosity with FWC are very useful in explaining the effect of FWC on mixture stiffness.

It can be clearly seen that the point at which the ITSM value drops in the unstable zone is identified as the point where the foam viscosity increases dramatically. Beyond this point (at higher FWC) the mixture stiffness value will depend on the balance of ERm and foam viscosity. It was also found that some water remains in the foam at high FWC application rate (e.g. at > 7%, at a bitumen temperature of 180^{o}C). Therefore, it is not recommended to use foam in this zone due to the risk of remaining water and the unpredictability of mixture performance, despite the fact that the ITSM values are sometimes high.

The start of the unstable zone at the point where the foam viscosity increases dramatically can be used to identify the end of the stable zone. However, it is impractical to use foam viscosity as a limiting value due to the difficulty in defining it. Therefore, the FWC and bitumen temperature are selected to define the limit since both have a link to foam viscosity. Both these values are also easily defined in the laboratory or the field. Figure 7.1 shows data from specimens using FB 70/100 at 180^{o}C at various FWC, which show the ITSM value dropping at a FWC of 6% (termed the critical FWC). Consequently, the end of the stable zone will be at a FWC slightly lower than 6%. It can be seen that at a FWC of 5% the ITSM reaches its maximum value. Figure 7.1 also shows data from specimens using FB 50/70 at 180^{o}C and FB 160/220 at 150^{o}C, for which the maximum ITSM value is found at a FWC of 4% for FB 50/70 while for FB 160/220 the ITSM values at FWC ≤ 4% are all slightly higher than at FWC > 4%. It can be seen that with a bitumen temperature of 150^{o}C, the critical FWC is found to be lower than at 180^{o}C. However, this is also a function of the binder viscosity. The critical FWC of harder binder will tend to be lower than that of softer binder at a similar bitumen temperature.

A procedure to determine the critical FWC for each binder type and bitumen temperature has been developed. As shown in Figure 7.1, the bitumen viscosity at which the ITSM drops was identified for those three binder grades. It is found that the critical bitumen viscosity is around 1.5 Pa.s. It is noted that this viscosity is based on the foam temperature at 60 seconds in the scenario in which foam is sprayed into a steel cylinder (namely Scenario 2 in Chapter 4). This critical bitumen viscosity was used to determine the critical foam temperature for those three binder types as shown in Figure 7.2. The resulting critical foam temperatures for bitumen Pen 50/70, Pen 70/100 and Pen 160/220 are 116^{o}C, 106^{o}C and 98^{o}C, respectively. As described in Chapter 4, the foam temperature at 60 seconds for Scenario 2 at various FWC can be calculated (see Table A.4.2). The results can be seen in Figure 7.3, in which the temperatures decrease with FWC. The lowest foam temperature was found to be 97^{o}C for all bitumen temperatures. The line of foam temperature for each bitumen temperature can be developed theoretically as shown in this figure. The intersection between these lines and the critical foam temperature was defined as the critical FWC for the corresponding binder type and bitumen temperature (see Figure 7.3). For example, the critical FWCs for Pen 50/70 at 180^{o}C, Pen 160/220 at 150^{o}C and Pen 70/100 at 180^{o}C are found to be at 5%, 5% and 5.8%, respectively, which are close to the FWC values where the ITSM drops as shown in Figure 7.1.

If the critical FWC for each application of binder type and bitumen temperature has been determined, this identifies maximum FWC application rate. It should be lower than the critical value. Applying a safety factor, this study recommends a maximum FWC limit about 1% lower than the critical FWC. Table 7.3 shows the recommended maximum FWC limit for three binder types at bitumen temperatures of 140^{o}C to 180^{o}C. For bitumen Pen 50/70, bitumen temperatures of 140^{o}C and 150^{o}C are not recommended since their ERm is statistically lower than at high temperatures and their viscosity is also too high. Applying a temperature higher than 180^{o}C (e.g. at

190°C or 200°C) may be considered since it results in lower foam viscosity and is predicted to give higher ERm, as long as the ageing effect is not significant. For bitumen Pen 160/220, bitumen temperatures of 170°C and 180°C are also not recommended since their ERm is statistically lower than at low temperatures. This also avoids the ageing effect. The effect of FWC on mixture stiffness is actually not too significant for bitumen Pen 160/220. Finally, at a temperature of 140°C for bitumen Pen 70/100, the stable zone is found to be very narrow and hence it is not recommended to apply this temperature.

Table 7.3 - The maximum FWC limit to produce stable mixture performance

Binder type	Bitumen temperature					Notes
	140°C	150°C	160°C	170°C	180°C	
Pen 50/70	Not recommended		2.5%	3.1%	4%	Higher temperature e.g. 190°C and 200°C can be considered
Pen 70/100		2.5%	3.2%	4%	4.8%	At 140°C the stable zone is too narrow
Pen 160/220	3%	4%	4%	Not recommended		Effect of FWC is not too significant

Note: The maximum FWC limit is considered to be 1% lower than the critical FWC.

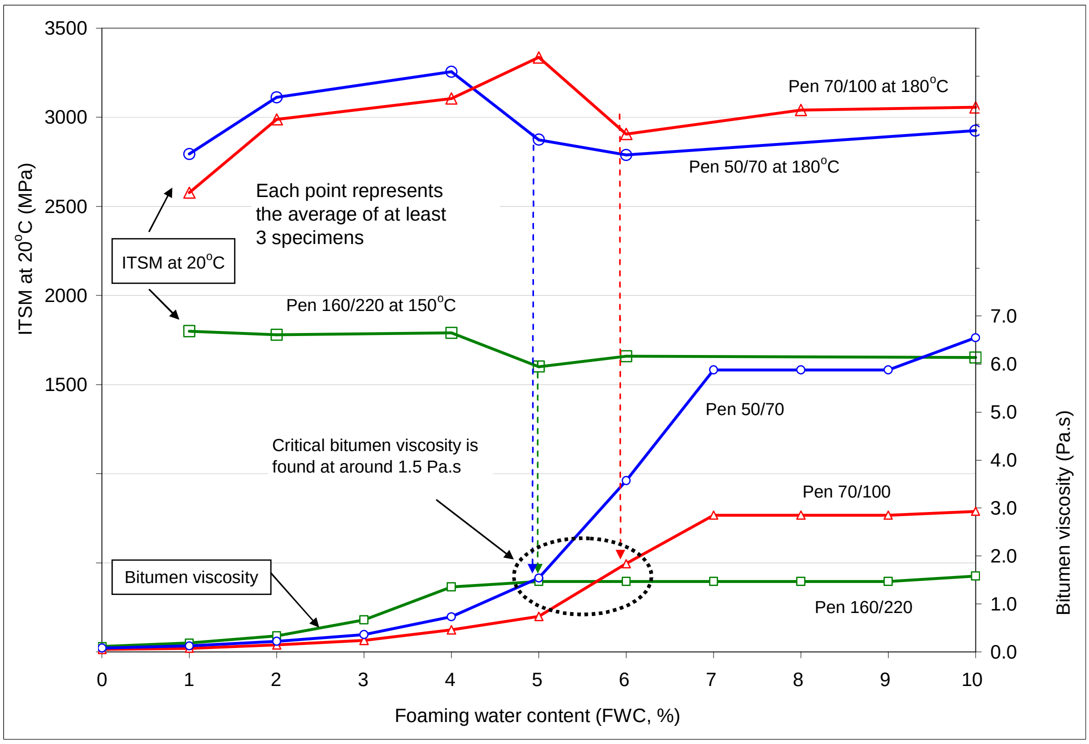


Figure 7.1 - Determination of the critical bitumen viscosity based on the drop in ITSM value (at 20°C) in the unstable zone for bitumen Pen 50/70, Pen 70/100 and Pen 160/220

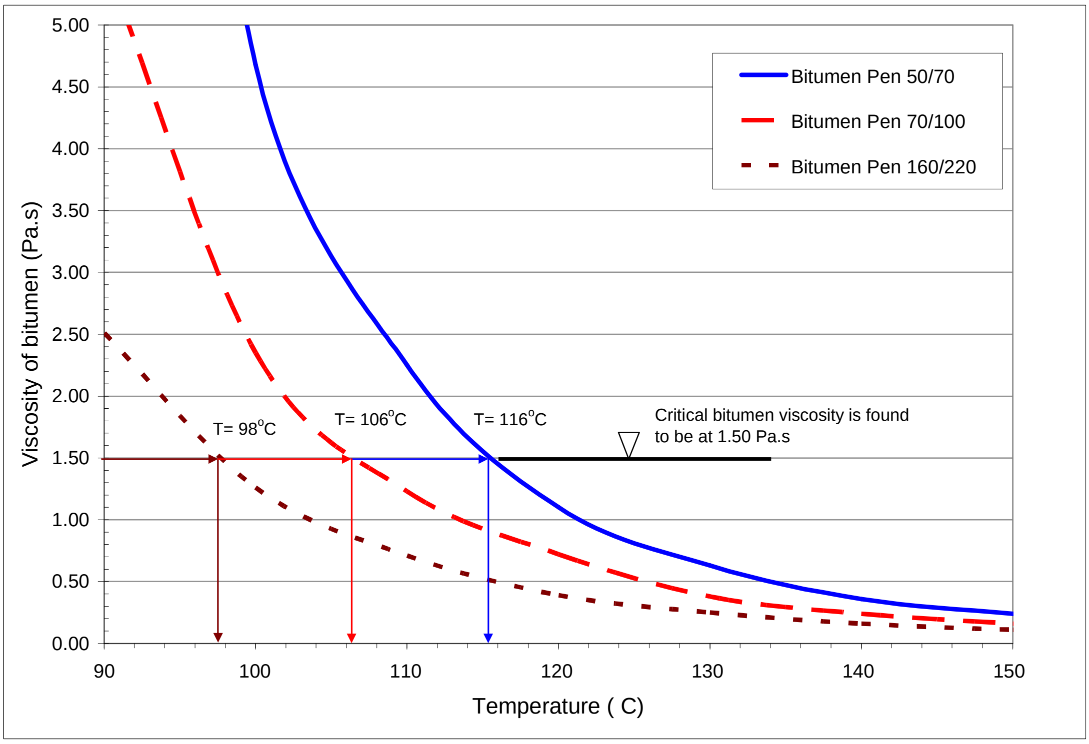


Figure 7.2 - Determination of the critical temperature based on the critical bitumen viscosity for bitumen Pen 50/70, Pen 70/100 and Pen 160/220.

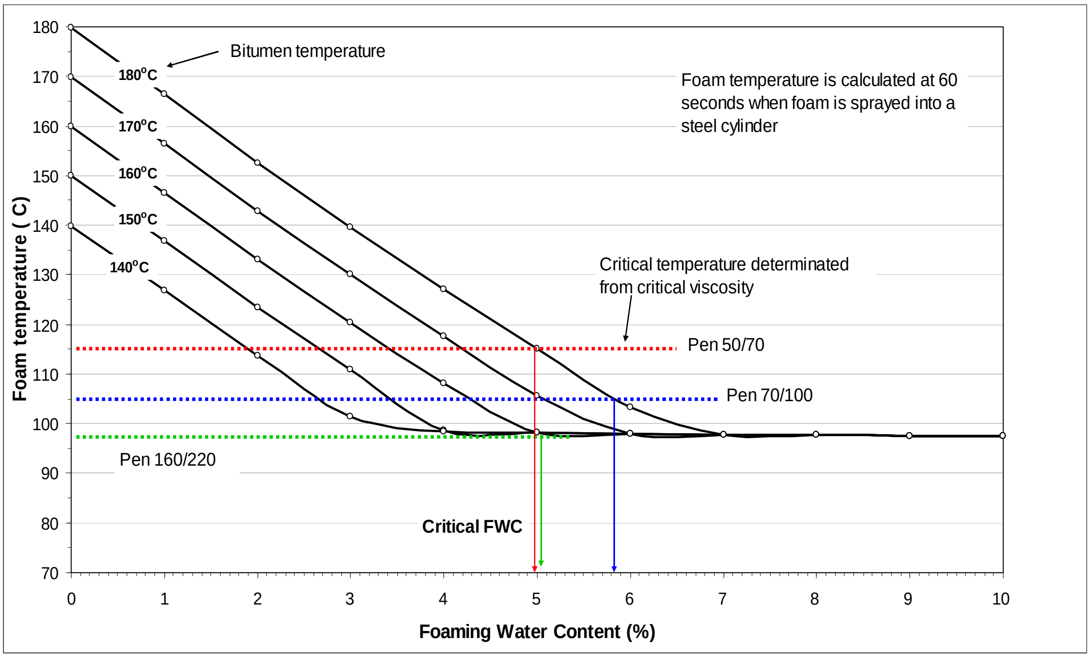


Figure 7.3 - Determination of the critical FWC based on the critical temperature for bitumen Pen 50/70, Pen 70/100 and Pen 160/220 at various bitumen temperatures.

7.2.5 Recommendation to achieve the best mixture performance

Table 7.2 and Table 7.3 provide the minimum and maximum application limits to produce stable FAM performance for three binder types. The stable range varies with bitumen temperature. The lower the bitumen temperature and bitumen pen the shorter the stable range. It is expected that a mixture generated in the stable range will produce stable mixture performance, within which the best performance is located. Since the stable zone is sometimes wide, it is necessary to indicate the location of best mixture performance.

It is recommended to apply a relatively low ERm (within the stable zone), i.e. 10 – 15 (or FWC around 1.5-2%), when low bitumen pen (Pen 50/70) is used due to its natural viscosity already being high. However, when bitumen Pen 70/100 is used, the use of a higher ERm within the stable zone is suggested. It is recommended to apply a value within 1% of the maximum FWC limit. For bitumen Pen 160/220, the stiffness value is affected only very slightly by ERm for any FWC application due to its viscosity being low, such that almost all FWC values can 'in practice' be used; however this study suggested a low ERm, i.e. between 5 and 10 (or FWC around 1% to 1.5%) since low FWC represents least risk (of the presence of the remaining water and high viscosity binder).

It is suggested to apply a higher ERm or FWC when a very effective mixer is used since mixer quality can enhance the ERm effect and counteract the binder viscosity effect. If foam viscosity is low (e.g. Pen 160/220), the mixing quality is less significant and hence it is better to apply a lower ERm as discussed above. Foam with higher HL or FI value at the same ERm is preferred since it will produce better mixture performance.

The procedure to define the application limits is one step toward selecting the most suitable foamed bitumen characteristics to produce optimum FAM performance and to unlock proper understanding of FAM performance, which is affected by many complex parameters. It should be noted that these suggestions are based on mixture obtained in a Hobart mixer using a flat agitator at a speed level 3 (365 rpm); if a better mixer is used, a higher FWC can be applied.

Table 7.4 - Recommendations to achieve the best performance of FAM

Binder type	Recommended range to achieve the best mixture performance	Note
Pen 50/70	ERm between 10 and 15 (or FWC around 1.5%-2%)	- With a very effective mixer, it is suggested to use a higher ERm or FWC. - FWC_{max} is maximum FWC limit (depend on bitumen temperature)
Pen 70/100	Between FWC_{max} and (FWC_{max}-1%)	
Pen 160/220	ERm= 5 - 10 (or FWC around 1% -1.5%)	Mixer speed and ERm value are not significant; low FWC represents least risk.

Note: Foam with higher HL or FI value at the same ERm is preferred.

7.3 Closure

For practical purposes, this study suggests the guidance for mixture design and FAM production presented in Table 7.5. The proposed guidance is based on the laboratory work and theoretical study conducted in this project. Since bituminous material performance is affected by many aspects such as material properties and laboratory test procedure, the limitations of the proposed guidance should therefore be stated as follows:

- Based on one source bitumen,
- Based on one type aggregate, i.e. crushed carboniferous limestone from Dene Quarry, Derbyshire, UK,
- Foam was produced using a Wirtgen WLB 10 foaming plant,
- Material was mixed using a Hobart 20 quart capacity mixer at a speed level 3 (365 rpm), a flat agitator and a batching mass of 4-6 kg,
- Based on a mixture design for which aggregate water content and bitumen content are 4.6% and 4% by aggregate mass respectively,
- Material was compacted using a gyratory compactor,
- Samples were oven cured at 40^{o}C for 3 days,
- Stiffness evaluations were conducted using Nottingham Asphalt Tester facilities.

Nevertheless, it is believed that the results of this study are likely to be generally applicable.

Table 7.5 - Practical guidance for foamed asphalt mixture (FAM)

Mixer consideration:
For all FAM production, it is highly recommended to use a high speed mixer and suitable agitator. If an appropriate mixer is not available, the following suggestions can be applied:

- Use high bitumen pen (e.g. Pen 160/220),
- Apply high bitumen temperature (e.g. 160^{o}C-180^{o}C),
- Apply low FWC (e.g. 1%-1.5%) or low ERm (e.g. 5-10),
- Use longer mixing time (e.g. 1-2 minutes),
- Use small batching mass (to be judged in relation to the mixer capacity),
- Use small maximum aggregate size (e.g. 10mm),
- Apply to a thin layer of recycled pavement (field in-situ case only).

<table>
<tr><th colspan="2">Bitumen grade</th><th>Pen 50/70</th><th>Pen 70/100</th><th>Pen 160/220</th></tr>
<tr><td colspan="2">Bitumen temperature</td><td>160^{o}C to 180^{o}C</td><td>No clear trend</td><td>140^{o}C to 160^{o}C</td></tr>
<tr><td colspan="2">Minimum ERm application limit</td><td>ERm= 10</td><td>ERm= 10</td><td>ERm= 5</td></tr>
<tr><td rowspan="5">Maximum application limit (FWC_{max})</td><td>140^{o}C</td><td rowspan="2">Not recommended</td><td></td><td>FWC= 3%</td></tr>
<tr><td>150^{o}C</td><td>FWC= 2.5%</td><td>FWC= 4%</td></tr>
<tr><td>160^{o}C</td><td>FWC= 2.5%</td><td>FWC= 3.2%</td><td>FWC= 4%</td></tr>
<tr><td>170^{o}C</td><td>FWC= 3.1%</td><td>FWC= 4%</td><td rowspan="2">Not recommended</td></tr>
<tr><td>180^{o}C</td><td>FWC= 4%</td><td>FWC= 4.8%</td></tr>
<tr><td colspan="2">Recommendation to achieve the best performance</td><td>ERm between 10 and 15 (or FWC around 1.5%-2%)</td><td>Between FWC_{max} and (FWC_{max}-1%)</td><td>ERm between 5 and 10 (or FWC around 1%-1.5%</td></tr>
<tr><td colspan="2">Mixer</td><td>High mixer speed</td><td>High mixer speed</td><td>A lower speed mixer can be used</td></tr>
<tr><td colspan="2">Climate</td><td>Hot region</td><td>Most climates</td><td>Cold region</td></tr>
</table>

Notes:

- ERm of 5 is considered as the start of wet foam structure (80% gas content), whereas ERm of 10 is the start of the stable dry foam that comprises 90% gas content.
- When using a better mixer, a higher ERm and/or FWC can be applied, except for bitumen Pen 160/220 for which mixer speed and ERm are not significant.
- Applying higher temperature (e.g. 190^{o}C and 200^{o}C) for bitumen Pen 50/70 can be considered.
- Foam with higher HL or FI value at the same ERm is preferred.

8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

This research project has studied the influence of foamed bitumen characteristics on cold-mix asphalt properties. The conclusions presented in this section have been drawn from the work conducted in this project. The conclusions deal with the general aim and the specific objectives of this research as stated in Chapter 1.

Initial study

1. Foamed asphalt is a unique mixture. Not all aggregate particles are coated by binder. Foamed bitumen, as a binder, is able to distribute onto wet aggregate at ambient temperature but is found on fine particles only. The important factor in attaining optimum mixture performance have been identified, namely (1) that the predetermined aggregate moisture is correct (Brennen et al, 1983), (2) that the quantity of fine particles is sufficient (Ruckel et al, 1982), (3) that proper design is made of selected foamed bitumen characteristics including binder content (see also Muthen, 1999), and (4) that suitable mixing is carried out (see also Long et al, 2004).

2. Following the pilot scale trial in the Nottingham University Pavement Test Facility, it can be deduced that foamed asphalt mixture (FAM) has definite potential for use in road pavements. The results can be summarised as follows: (1) the process of FAM manufacture is easily handled, clean and allows storage as required prior to compaction, (2) FAM exhibits good structural capability to support traffic load and has a good ability to spread load, (3) The evidence of rutting in FAM is mainly due to densification and the weakness of bonds during early life, (4) FAM demonstrates excellent fatigue resistance which indicates the flexibility of the material, and (5) the use of cement can accelerate the curing process, enhance the strength of the material and significantly reduced the measured rutting.

3. The current parameters used to characterise foamed bitumen are found to be questionable and hence the method of selecting the optimum foamed bitumen characteristics remains a problematic issue. Lack of understanding of foamed bitumen characteristics in association with the manufacturing process and in-service performance of the mixture is considered to be the root problem.

Foamed bitumen characteristics

4. Foamed bitumen has been confirmed as a member of the general foam family. Foamed bitumen is mainly composed of bitumen liquid and steam gas, with the possibility of both wet and dry foam formation (Breward, 1999), giving an indication that foamed bitumen behaviour complies with expected general foam behaviour. This includes the categories of 'foam quality', representing gas content, for which wet foam ranges between 52% (Mitchell, 1971) and 87% (Weaire et al, 1993), stable dry foam between 87% and 96% (Rankin et al, 1989), and beyond 96% the dry foam becomes unstable (Rankin et al, 1989). A dry foam tends to have higher elastic modulus and apparent (kinematic) viscosity than a wet foam (Weaire and Hutzler, 1999; Heller and Kuntamukkula, 1987). Foam modulus is small and dependent upon foam surface properties (Weaire and Hutzler, 1999). The term 'apparent or effective viscosity' is used to describe a foam rheology that is affected by the presence of compressible gas bubbles (Heller and Kuntamukkula, 1987). The term 'kinematic viscosity', ratio between viscosity and density is likely to be a more suitable representation of the resistance to foam flow since foam viscosity is dependent on its density, viscosity increasing at lower density (increased gas content).

5. Foamed bitumen is a soft material with complex behaviour. It is generated by a heat transfer process between hot bitumen and cold water and is successfully formed by the presence of a surfactant (Koelsch and Motschmann, 2005) which is primarily contained in asphaltenes (Barinov, 1990). Low penetration bitumen with high asphaltene content will therefore tend to produce a longer foam life, but high viscosity bitumen also makes the bubbles difficult to develop and hence the quality of foam reduces. The heat energy of hot bitumen is needed by the water to develop

steam bubbles (Jenkins, 2000), but high bitumen temperature also causes the bitumen viscosity and surface tension to decrease, which initiates bubble collapse (He and Wong, 2006). Both viscosity and surface tension have complex effects and they are interrelated. A bitumen with low viscosity enhances foam quality. The surface tension, which is strongly dependent upon viscosity, is not only primarily required by thin lamella to balance the internal pressure of an explosive bubble (Jenkins, 2000), but also required to balance Plateau border suction, in order to resist liquid drainage (Breward, 1999). All these complex aspects of the behaviour of foamed bitumen are likely to be linked with its temperature, as a result of the heat transfer process. Thus, an essential balance is required in order to generate foamed bitumen with optimum properties.

6. The effect of foaming water content (FWC) on foamed bitumen characteristics in terms of maximum expansion ratio (ERm) and half-life (HL) has been identified. It is clearly evident that the value of ERm increases with increasing FWC; however the HL value follows the opposite trend. This is because dry foam (high ERm) tends to be more unstable than wet foam (low ERm) (Kraynik, 1983). The trend of HL at high FWC is to be constant or to increase slightly, since foam temperature reduces with increasing FWC, causing foam bubbles to collapse more slowly. Thus, the lower foam temperature balances the higher gas content and the result is near constant HL at high FWC. In addition, the effect of FWC on Foam Index (FI) values is mainly a function of ERm, and therefore the values of FI and ERm over various FWC are approximately proportional.

7. Bitumen temperature and binder type represent a combined factor in affecting the foaming process since both influence viscosity. In general, for FB 160/220, lower bitumen temperature produces higher ERm, whereas for FB 50/70, this trend is reversed. For FB 70/100 the ERm performed inconsistently with bitumen temperature. On the other hand, the HL generally tends to decrease with increasing temperature for all foams except for FB 70/100. So, according to the ERm value, a moderate bitumen grade appears more suitable than an extreme soft or hard bitumen. However, considering the HL value, a bitumen with low pen and temperature will be

more suitable.

8. The effects of foaming water content (FWC) and bitumen temperature on the foam flow behaviour can be clearly observed. The rate of foam flow through orifice(s) tends to decrease with increasing FWC and decreasing bitumen temperature. In agreement with Kraynik (1988), foam flow rate can be linked to its viscosity and hence it can be deduced that a foam with a higher FWC tends to have a higher apparent viscosity; this is in line with Marsden and Khan's finding (Heller and Kuntamukkula, 1987). The reduction of foam flow rate due to decreasing bitumen temperature is caused by increasing foam viscosity.

9. The effective foam viscosity estimated using the Kraynik equation is dependent upon gas content and bitumen viscosity, in which foam temperature plays a noticeably important role. An important point is that, at an ERm of around 25 (for FB 50/70 at 180^{o}C) or around 35 (for FB 70/100 at 180^{o}C), foam viscosity can be seen to reach a critical point at which the viscosity value increases dramatically.

10. Properties of the collapsed foamed bitumen have been investigated in terms of penetration, RTFOT and bulk density tests. The penetration test results are not valid due to the remaining bubbles causing problems during testing. From the RTFOT, it is found that the foaming process does not cause significant ageing to the bitumen. Based on bulk density test results, the density of collapsed foam is found to be lower at a higher FWC. It is therefore supposed that large number of tiny bubbles and water droplets are still trapped inside the collapsed foamed bitumen. This indicates that the collapsed foam does not completely return to the state of the original bitumen. This may allow the loose FAM to be stored for months.

Compactability of foamed asphalt mixture (FAM)

11. Indirect Tensile Stiffness Modulus (ITSM) values of cured FAM specimens are found not to be significantly affected by the compaction mode (force, angle and number of gyrations) as long as the final densities were comparable. It was clearly

observed that the ITSM values were linked to dry density.

12. Based on the linear trend line, the compactability of FAM tends to increase with the FWC; with the bitumen temperature it tends to increase for low penetration binder but tends to decrease for high penetration binder. The important point is that the binder type affected mixture compactability significantly, the mixture produced using binder of Pen 50/70 giving poor density. This means that FAM using a softer binder tends to give better compactability performance than those using harder binder.

Characteristics of ITSM values of foamed asphalt mixture (FAM)

13. The trends of horizontal deformation/ stress and test temperature effect on the ITSM value of FAM were found to be similar to those of Hot Mix Asphalt (HMA), the ITSM decreasing with those two parameters. However, the ITSM values of FAM were found to be more sensitive to applied horizontal deformation and less sensitive to temperature than those of HMA. These facts may indicate that the ITSM test is suitable to evaluate the stiffness of FAM materials. It is supposed that binder distribution in the mixture controls the stiffness value of FAM. Logically, well mixed specimens will tend to be less sensitive than poorly mixed specimens in terms of the effect of horizontal deformation, but more sensitive in terms of test temperature.

Foamed asphalt mixture (FAM) performance: Indirect Tensile Stiffness Modulus (ITSM)

14. The mixing process was found to be an important factor in developing mixture performance. It was observed that mixer agitator type significantly influences mixture performance. The flat agitator performed better at mixing than the dough hook and it aided material properties and enhanced the ITSM value. Binder in well mixed specimens was better distributed and hence more continuous than in poorly mixed specimens.

15. Specimens mixed at different FWC values using the dough hook exhibited no significant differences in ITSM; however when specimens were mixed using the flat agitator, their cured ITSM values increased dramatically and the effect of FWC was clearly evident in most cases. However the effect of FWC was still not clearly observed for specimens produced using bitumen Pen 160/220. It is supposed that mixtures produced using soft bitumen are easy to mix and result in very good mixing for all values of FWC and hence there is no difference in specimen performance.

16. Optimum mixture performance was obtained at a FWC of 5% for specimens prepared using bitumen Pen 70/100 (and 4% for Pen 50/70). This became particularly evident when the full range of testing temperatures and curing conditions was considered. Optimum performance is obtained with the best binder distribution in the mixture, in which state the mixture is most sensitive to testing temperature and curing condition.

17. At a test temperature of $5^{o}C$ binder distribution is found to be more important than binder stiffness in developing the ITSM value. At a test temperature of $20^{o}C$ the ITSM values of specimens produced using bitumen Pen 160/220, which had very good binder distribution, were lower than the ITSM values of specimens produced using bitumen Pen 70/100, but at a test temperature of $5^{o}C$ the ranking was reversed.

18. Well mixed specimens (using soft binder of Pen 160/220) were found to be more resistant to water damage than poorly mixed specimens (using hard binder of Pen 50/70). The effect of FWC (or ERm) on the water sensitivity is clearly evident in the specimens using bitumen Pen 50/70. At a FWC of 1%, the ITSM ratio is lowest, implying specimen damage. The ITSM ratio exhibits an increase with FWC, possibly indicating that binder ageing occurs more readily at higher ERm or FWC values.

19. At selected FWC, bitumen foaming temperature was found to be less significant than FWC in developing stiffness since foam properties do not change significantly with bitumen temperature.

Foamed asphalt mixture (FAM) performance: Repeated Load Axial Test (RLAT) and Indirect Tensile Fatigue Test (ITFT)

20. Effect of foamed bitumen properties on the resistance to permanent deformation under the RLAT was not clearly observed. The tests were carried out on specimens produced using FB 70/100 at 180^{o}C with two types of aggregate, i.e. 20mm and 10mm graded. The specimens were tested at a vertical stress of 200 kPa and a temperature of 30^{o}C (for 20mm graded specimens) and at 100 kPa and a temperature of 40^{o}C (for 10mm graded specimens), which resulted in strains at 1800 pulses of about 0.4-0.5% and 2-5% respectively. Two important points are (1) that the strain rate over the second 1800 cycles from tests using 20mm graded specimens was found to be highest at a FWC of 6%, and (2) that the resistance to permanent deformation of 10mm graded specimens was found to be closely related to compaction energy.

21. Based on fatigue lives at 200 micronstrain, the effect of FWC on the fatigue performance of FAM material is not significant. The fatigue life was significantly affected by applied stress level and clearly linked with the stiffness value. Based on the fatigue lines developed (from specimens using bitumen Pen 70/100 at 180^{o}C at FWC values of 2%, 5% and 10%), a FWC of 5% was found to give the lowest slope (best performance) and a FWC of 10% gave the steepest slope (worst performance). The $N_{critical}$ value (number of cycles (N) at the peak value of N/vertical deformation) was generally about 60%-66% of $N_{failure}$. This value is about 90% for HMA mixtures (Read, 1996). It means, following Read's (1996) concept that foamed asphalt materials will initiate cracking faster than HMA, but have a longer crack propagation life.

Effect of foamed bitumen characteristics on mixture performance

22. Foamed bitumen properties (with different FWC values) are concluded to have a moderate effect on foamed asphalt performance. Mixing protocol and binder type are found to have a more dominant effect than foam properties. The effect of foam properties is therefore only clearly defined in well mixed specimens, based on stiffness evaluation. The effect of foam properties is also found to vary with binder

type. A mixture produced at a FWC of 1% will perform poorly, whereas a FWC of 2% to 4% statistically results in better performance than other FWC applications. It can thus be argued that peak performance from a foamed asphalt (assessed for example using stiffness criteria) can only be guaranteed when the mixture is manufactured and compacted in the optimum range of FWC.

23. Binder distribution is found to be a key issue in understanding foamed asphalt properties. A uniform binder distribution will result in a high performance mixture. The mechanism of binder distribution has been fully described. Following the mixing process, foamed bitumen forms binder-fines particles that distribute across the aggregate phase. These particles are a combination of bitumen and fine aggregate, whose workability influences the uniformity of binder distribution. The properties of these binder-fines particles are expected to be proportional to those of the foamed bitumen. This is strongly supported by the fact that the ITSM over various FWC values is closely related to foamed bitumen properties such as ERm, apparent viscosity of foam and foam structure (wet or dry foam; stable or unstable foam).

24. The ITSM over various FWC values was found to be affected by a combination of ERm and apparent viscosity of foam. Three zones of ERm values are proposed, namely a poor zone, a stable zone and an unstable zone, which can be linked to types of foam structure, namely wet foam structure (52%-87% quality), stable dry foam structure (87%-96% quality) and unstable dry foam structure (>96% quality), respectively. In the poor and stable zones, which give relatively low apparent foam viscosity, the ERm is the main factor controlling the ITSM value. However, in the unstable zone, the apparent viscosity increases dramatically and influences mixture stiffness, causing variation in ITSM values.

25. The poor ERm zone will be between 3 and 8 (relating to the wet foam quality, i.e. between 52%-87%), the stable zone is between 8 and 25 (for binder Pen 50/70 at 180^{o}C) or 8 and 33 (for binder Pen 70/100 at 180^{o}C), and beyond this ERm value (25 or 33) is the unstable zone. The maximum ERm limit in the stable zone varies for different binder types and bitumen temperatures. It is expected that the use of harder

binder and lower bitumen temperature tends to produce a smaller stable zone. In this study, material using bitumen Pen 160/200 was found to produce no significant variation in ITSM over the range of ERm values and hence the zone categories could not be defined as clearly as for other bitumen grades.

26. The way that foamed asphalt develops its stiffness has been identified. Referring to Thom and Airey (2006), the absence of binder at certain points of contact between aggregate particles (due to poor binder distribution) will result in inter-particle movement within the aggregate and hence will give a low modulus. Conversely, if the binder is present, it will resist inter-particle motion and hence potentially increase stiffness. Alternatively, with reference to Brown and Brunton (1986), a better binder distribution may reduce the void volume and hence increase the stiffness.

Main considerations to achieve an optimum foamed asphalt performance

27. It is highly recommended to use as good a mixer as possible in terms of its speed and agitator type for any purpose in FAM production. However, whether or not such a mixer is available, the following suggestions may also be useful.

28. The considerations relating to binder type selection for FAM are basically similar to those for all bituminous materials, namely that a heavily trafficked road or a road in a hot region typically requires a harder binder. However, in FAM, the effects of binder type on the ERm and the workability of binder-fines particles during the mixing process should also be considered. It is suggested to use higher bitumen temperature for lower bitumen pen (Pen 50/70) or use lower bitumen temperature for higher bitumen pen (Pen 160/220). If a high quality mixer is not available, the use of higher bitumen pen or bitumen temperature is recommended. A FAM using soft binder will be more suitable for cold regions since binder distribution is more important in developing mixture stiffness at low temperature, whereas hard binder, which exhibits poor binder distribution and high stiffness, will be more suitable for hot regions. Bitumen Pen 70/100 is a moderate grade binder which may be used under most conditions. To select bitumen temperature for this bitumen type it is

suggested that a proper investigation (effect of bitumen temperature on ERm) is carried out.

29. ERm value is used to define a minimum FWC limit in producing a stable mixture performance. Currently, this limit is commonly based on ERm and HL values (e.g. CSIR 1999, TRL Report 386 or Wirtgen 2005, see Table 2.5), or FI (Jenkins, 1999). This study had reviewed these parameters. It was found that HL was only a complementary parameter to ERm; FI was found to be more meaningful than ERm in representing foam volume, but both are approximately proportional in their effects on mixture stiffness. It was therefore decided to use only ERm as a single parameter for the minimum FWC limit. This overcomes difficulty of selecting foam properties using both ERm and HL. This study recommends an ERm of 10 (after applying a safety factor) as a minimum in most cases. This value is considered as the starting point for stable dry foam structure (90% gas content). However, for high bitumen pen (e.g. Pen 160/220), this study recommends a lower ERm, i.e. 5 (after applying a safety factor), since this binder has very low viscosity causing the ERm effect to be less significant. This value is considered as the starting point for wet foam structure (80% gas content).

30. FWC and bitumen temperature are together used to define a maximum foam application limit since both are important factors affecting apparent foam viscosity. The maximum limit should be defined since mixture stiffness does not increase continually with increasing ERm value. This study has developed a procedure to identify this limit. Mixture stiffness increases initially with FWC due to increasing ERm value. However foam apparent viscosity also increases with FWC conteracting the ERm effect. At high FWC the viscosity becomes too high, causing the mixture stiffness to drop because of poor mixing of binder. Bitumen viscosity, as a component affecting foam viscosity, is used to identify the point at which the ITSM value drops, namely the critical bitumen viscosity which is found to be around 1.5 Pa.s. A critical temperature, the bitumen temperature which gives a viscosity of 1.5 Pa.s, can be determined for each binder type. Finally the critical FWC value, which produces a foam at the critical temperature, can be determined for each bitumen

temperature application. At this critical FWC the ITSM will drop and therefore the maximum application limit will be a slightly lower FWC value. Applying a safety factor, this study recommends a maximum FWC limit about 1% lower than the critical FWC. Maximum FWC limits to produce stable mixture performance have been recommended for three binder types at bitumen temperatures of 140^{o}C to 180^{o}C.

31. To achieve the best foamed asphalt mixture (FAM) performance, it is recommended to apply a relatively low ERm (in the stable zone), i.e. 10 – 15 (or FWC around 1.5-2%), when low bitumen pen (Pen 50/70) is used. However, when bitumen Pen 70/100 is used, the use of a higher ERm within the stable zone is suggested. It is recommended to apply a value within 1% of the maximum FWC limit. For bitumen Pen 160/220, the stiffness value is affected only very slightly by ERm for any FWC application due to its viscosity being low so that almost all FWC values can ‘in practice’ be used; however this study suggested a low ERm, i.e. between 5 and 10 (or FWC around 1% to 1.5%). It should be noted that these suggestions are based on mixtures created in a Hobart mixer using a flat agitator at a speed level of 3 (365 rpm); if a better mixer is used, a higher FWC or ERm can be applied.

Practical guidance to produce an optimised foamed asphalt mixture (FAM)

32. Practical guidance for mixture design considerations and FAM production is proposed as follows:

Mixer consideration:
For all FAM production, it is highly recommended to use a high speed mixer and suitable agitator. If an appropriate mixer is not available, the following suggestions can be applied:

- Use high bitumen pen (e.g. Pen 160/220),
- Apply high bitumen temperature (e.g. 160°C-180°C),
- Apply low FWC (e.g. 1%-1.5%) or low ERm (e.g. 5-10),
- Use longer mixing time (e.g. 1-2 minutes),
- Use small batching mass (to be judged in relation to the mixer capacity),
- Use small maximum aggregate size (e.g. 10mm),
- Apply to a thin layer of recycled pavement (field in-situ case only).

Bitumen grade		Pen 50/70	Pen 70/100	Pen 160/220
Bitumen temperature		160°C to 180°C	No clear trend	140°C to 160°C
Minimum ERm application limit		ERm= 10	ERm= 10	ERm= 5
Maximum application limit (FWC_{max})	140°C	Not recommended		FWC= 3%
	150°C		FWC= 2.5%	FWC= 4%
	160°C	FWC= 2.5%	FWC= 3.2%	FWC= 4%
	170°C	FWC= 3.1%	FWC= 4%	Not recommended
	180°C	FWC= 4%	FWC= 4.8%	
Recommendation to achieve the best performance		ERm between 10 and 15 (or FWC around 1.5%-2%)	Between FWC_{max} and (FWC_{max}-1%)	ERm between 5 and 10 (or FWC around 1%-1.5%
Mixer		High mixer speed	High mixer speed	A lower speed mixer can be used
Climate		Hot region	Most climates	Cold region

Notes:

- ERm of 5 is considered as the start of wet foam structure (80% gas content), whereas ERm of 10 is the start of the stable dry foam that comprises 90% gas content.
- When using a better mixer, a higher ERm and/or FWC can be applied, except for bitumen Pen 160/220 for which mixer speed and ERm are not significant.
- Applying higher temperature (e.g. 190°C and 200°C) for bitumen Pen 50/70 can be considered.
- Foam with higher HL or FI value at the same ERm is preferred.

8.2 Recommendation for future research

Based upon the findings of this research, the following recommendations are made:

1. Further investigations are needed to extend the main results of this research in terms of the influence of foamed bitumen characteristics on FAM properties for the following variables:

- Use various bitumen sources and aggregates types, e.g. granite, secondary and waste materials included RAP, and clay soils. Bitumen source and aggregate type are thought to be factors affecting adhesion between aggregate and binder, as well as binder distribution. Different bitumen sources will be accompanied by different chemical compositions and hence will result different foam characteristics. The use of RAP or other secondary materials is required since these materials are commonly used in cold recycling using foamed bitumen.
- Use confined laboratory tests such as the triaxial test. Since FAM is not a fully bonded mixture, its behaviour is found to be stress dependent, and this is likely to be affected by the uniformity of binder distribution. The influence of foam properties on mixture performance under confined test mode is required to simulate field conditions.
- Use various mixer types, especially a very high speed laboratory mixer. This study has identified that mixer type is a major influence on FAM performance. The effect of ERm on the binder distribution is found to be more evident when a higher mixer speed is used. Since the performance of FAM is mainly affected by mixing quality, it is required to standardise mixer type in order to compare the results of FAM studies between one research and another, as well as with the field mixing.
- Use various compactor types such as vibratory. In FAM, compaction is the process that changes a loose FAM in which the binder-fines particles are distributed across the aggregate phase to a compacted FAM in which the binder readily bonds to the aggregate.
- Use various curing methods such as at ambient temperature, with various curing times. As observed in the PTF test, the strength of FAM material increases with time due to the curing effect. This study investigated the performance of FAM

based on the specimens cured at 40^{o}C for 3 days. It is necessary to confirm the results of this study using ambient temperature curing to simulate the real field condition.

- Use various mix-design scenarios such as different gradations of aggregate, aggregate water contents and bitumen contents. In this study, it was observed that binder distribution across the aggregate phase depends upon the aggregate size, becoming less homogenous when larger aggregate size is used. Aggregate water content is an important factor in the mix-design of FAM. If the aggregate water is inadequate, foam will distribute ineffectively. However the quantity of aggregate water, together with bitumen content, bitumen temperature and FWC, will influence the resultant foam temperature, which controls the workability of binder-fines particles during the mixing process. It seems that a minimum aggregate water content is required to give a high foam temperature and therefore good foam distribution as long as there is sufficient overall fluid content for compaction. This study is based on an aggregate water content of 4.6% (% by mass aggregate) and a bitumen content of 4% (% by mass aggregate). When using RAP aggregate, the optimum binder content was found to be around 2.5%. This will reduce the workability of binder-fines particles (compared to having 4% binder content). The aggregate water content and bitumen content are therefore important factors in producing a homogenous FAM.

2. One of the significant findings from this research is that of the recommended foam application limits. The minimum application limit for each binder type was determined based on foam quality limits borrowed from a theoretical study of general foam, supported by the trend of ITSM at corresponding ERm values, whereas the critical FWC (a value slightly less than the maximum FWC limit) was determined from an estimated effective foam viscosity (using the Kraynik equation) and an analytical calculation of heat energy transfer, again supported by the trend of ITSM at corresponding ERm values. These limits need further verification by laboratory investigations of mixture stiffness at FWC values at small intervals (say every 0.25%) at around the limit value, for various binder types and bitumen temperatures, as well as binder contents and aggregate water contents.

3. The properties of the binder-fines particles are found to be crucial during the mixing process since they influence the uniformity of binder distribution in the FAM. In this study, these properties are, however, assumed to be proportional to those of the foamed bitumen. Further research is needed to investigate the properties of this binder-fines mixture, produced at various FWC values and bitumen temperatures using different binder types. Workability of the binder-fines mixture at various temperatures is the most important parameter to understand since it is believed to be a key factor affecting binder distribution quality.

4. A study of durability of FAM is actually required. Moisture damage and ageing are factors which contribute to pavement durability. Generally, FAM permeability is lower than that of granular pavement, but is higher than that of most asphalt pavements. The pattern of the aggregate-binder structure of FAM, in which not all aggregate particles are coated by binder, causes this material to be subject to weak bonds and hence its durability is questionable. In this study, in water sensitivity tests at 40^{o}C for 3 days, well mixed specimens (using soft binder of Pen 160/220) were found to be more resistant to water damage (lower permeability) than poorly mixed specimens (using hard binder of Pen 50/70). It is therefore reasonable to state that specimens generated at optimum FWC (expected to give best binder distribution) will be most durable. At a FWC of 1%, the ITSM ratio (ratio between dry ITSM and wet ITSM) was lowest, implying specimen damage. At a FWC of 10%, the ITSM ratio was highest and this was interpreted as indicating binder ageing. However, the individual effects of FWC on either specimen damage or binder ageing are still unclear. Further research is required to investigate the level of specimen damage and binder ageing at various FWC.

5. Limited laboratory testing has identified a moderate effect of foamed bitumen properties on FAM, as well as recommended foam application limits. On the basis of satisfactory performance in the laboratory, field trials are recommended to verify the influence of foamed bitumen properties on pavement performance under real conditions. It is required to compare laboratory prepared specimens with what is actually achievable in the field. The pilot scale trial in this research has highlighted

the effect of binder type (bitumen Pen 50/70 and Pen 70/100) on mixture performance. One of the objectives for further research would, therefore, be to investigate any effect of FWC variation on pavement performance with various mixer qualities. Since binder type, FWC and mixer quality are interrelated their effects on uniformity of binder distribution, a standardised mixer type is therefore most important. Ultimately the key research outcome would lead to full understanding of how to manufacture, lay and pave FAM so that the construction risk can be reduced and the mixture performance can meet the requirements.

6. Since the influence of foamed bitumen properties on mixture performance has been understood, in which the binder distribution during the mixing process is found to play a key role, further research to investigate coatability improvement could be progressively developed. It is suggested to investigate coatability of FAM in the following areas: a) improve foam volume, b) improve binder workability during the mixing process, c) improve the structure of the mixer agitator and d) improve affinity between binder and aggregate.

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APPENDICES

Appendix A – Figures

Appendix B – Tables

Figure A.4.1 means the first appendix figure in Chapter 4

Table A.6.1 means the first appendix table in Chapter 6

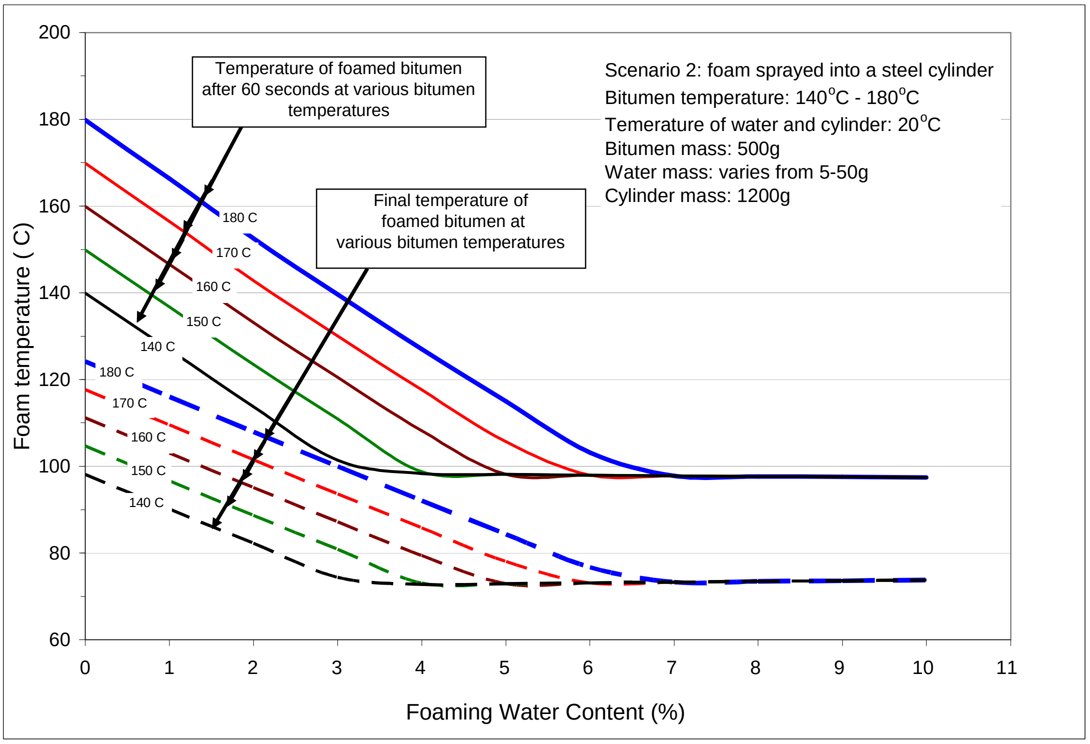

Figure A.4.1 – The final and at 60 seconds temperature of foamed bitumen for Scenario 2

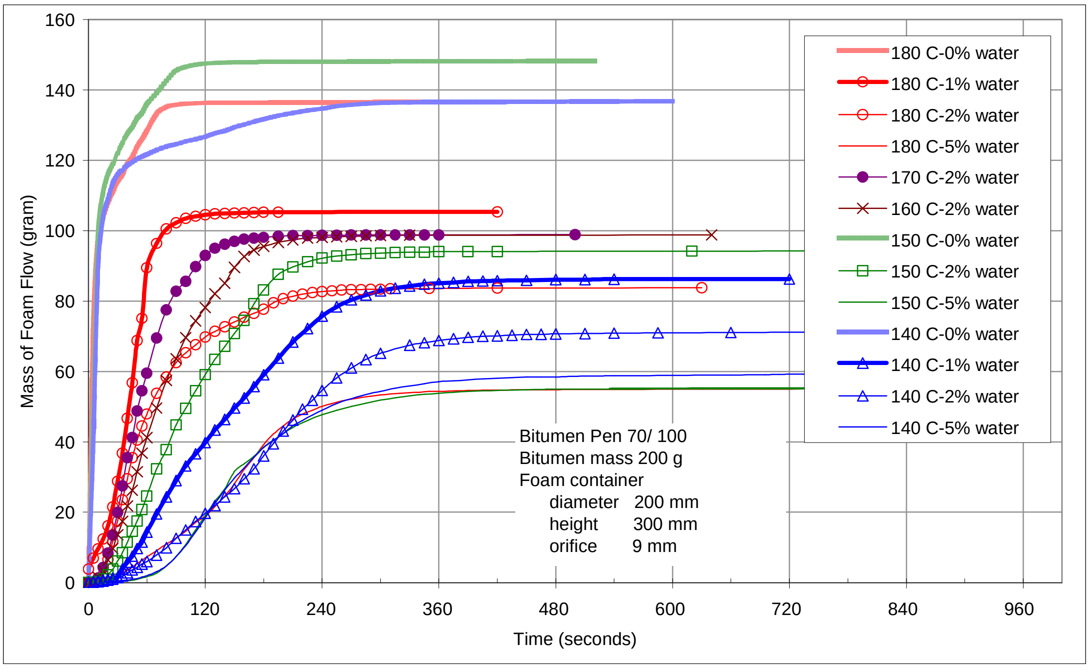

Figure A.4.2 – Flow rate through a 9mm orifice of foamed bitumen produced using bitumen Pen 70/100 at various bitumen temperatures and foaming water contents. Note: 0% water mean hot bitumen.

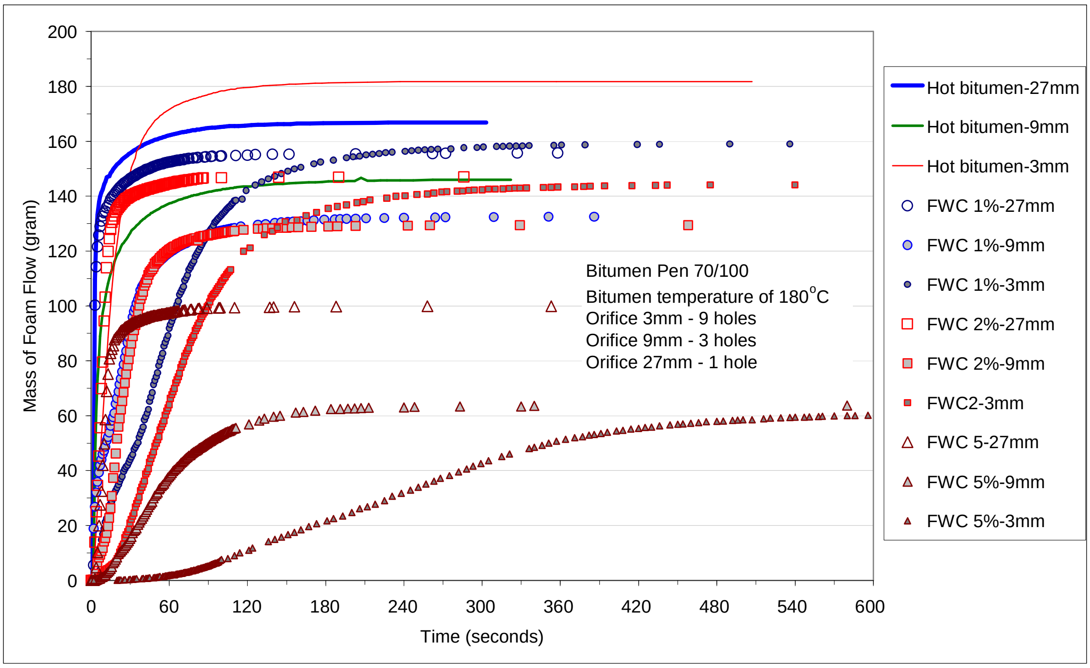

Figure A.4.3 – Flow rate through an orifice of foamed bitumen produced using bitumen Pen 70/100 at a temperature of 180°C and at various foaming water contents.

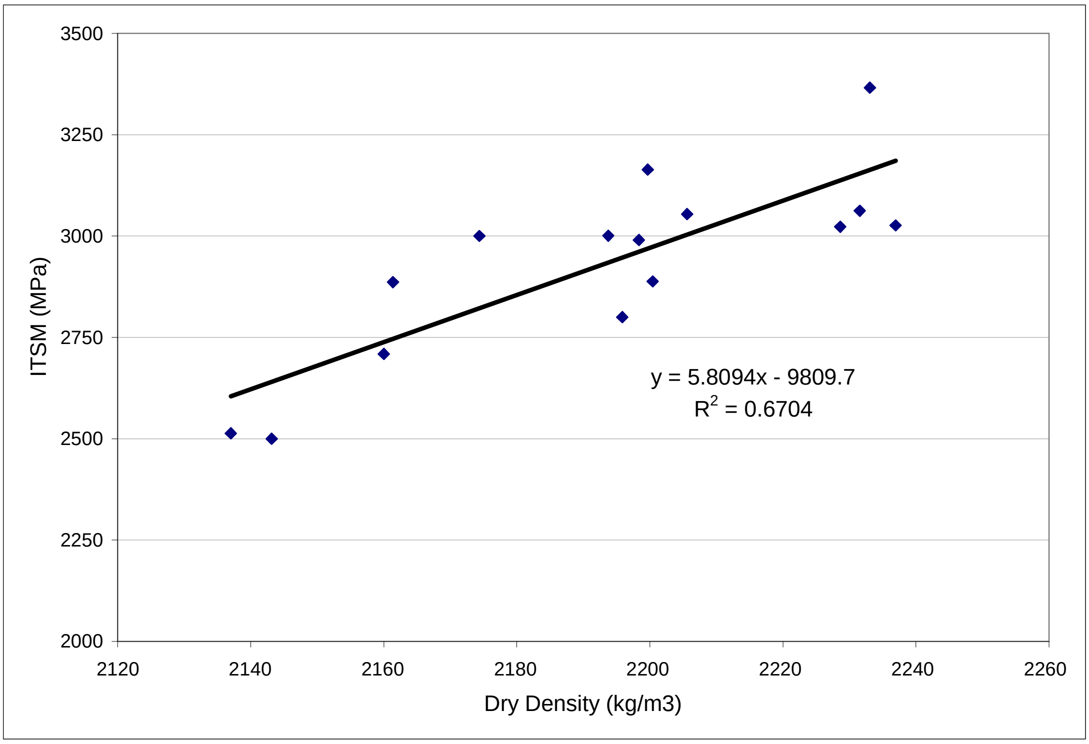

Figure A.5.1 – Relationship between dry density and ITSM values for specimens compacted at different gyration number.

Figure A.6.1 – Appearance of binder distribution of hot bitumen sprayed into cold wet aggregates: the mixture is an uncombined asphalt, in which large stiff aggregate-bitumen globules occur.

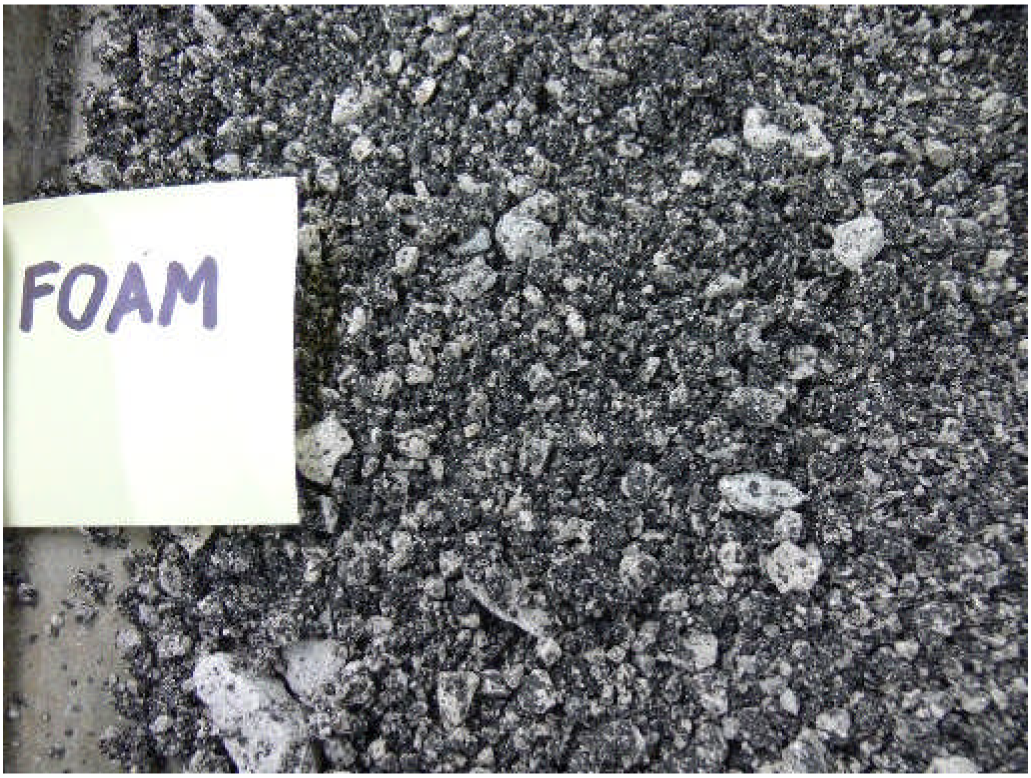

Figure A.6.2 – Appearance of binder distribution of foamed bitumen sprayed into cold wet aggregates: the binder is well distributed in the mixture and forms small binder-fines particles.

Figure A.6.3 – Appearance of binder distribution of foamed asphalt mixture at 2 seconds mixing time: forming large soft mastic globules

Figure A.6.4 – Appearance of binder distribution of foamed asphalt mixture at 5 seconds mixing time: forming broken soft mastic globules

Figure A.6.5 – Appearance of binder distribution of foamed asphalt mixture at 10 seconds mixing time: forming binder-fines particles.

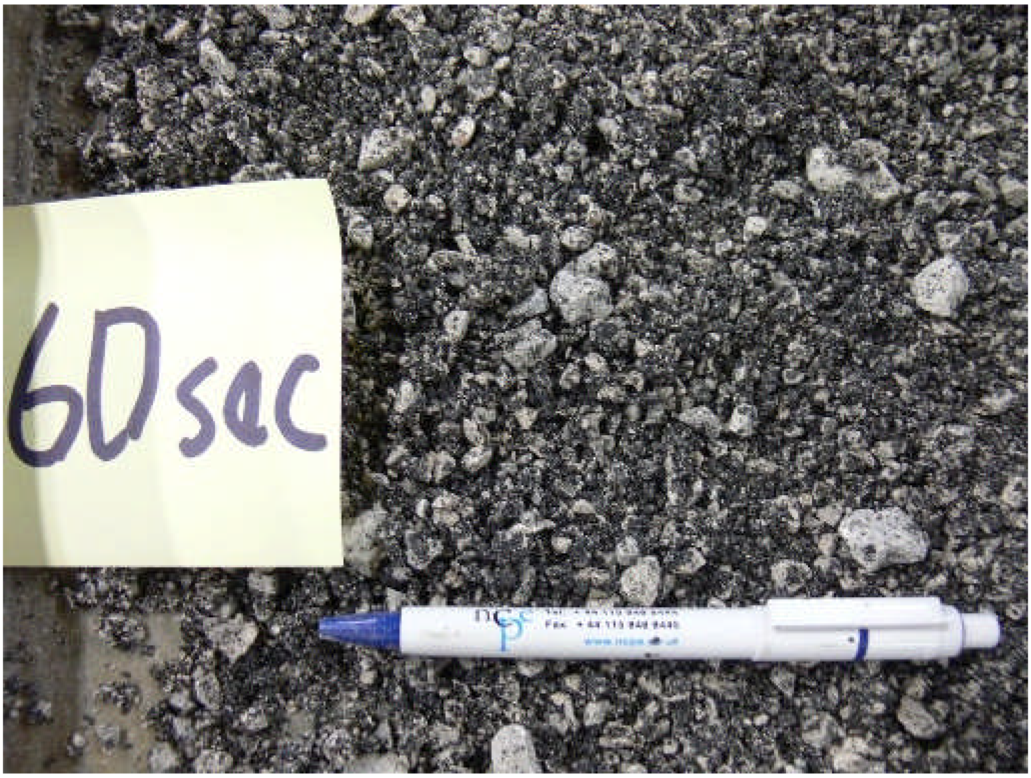

Figure A.6.6 – Appearance of binder distribution of foamed asphalt mixture at 60 seconds mixing time: forming small binder-fines particles (well distributed).

Figure A.6.7 – Appearance of binder distribution at 2 seconds mixing time of foamed asphalt mixture produced at a FWC of 1%.

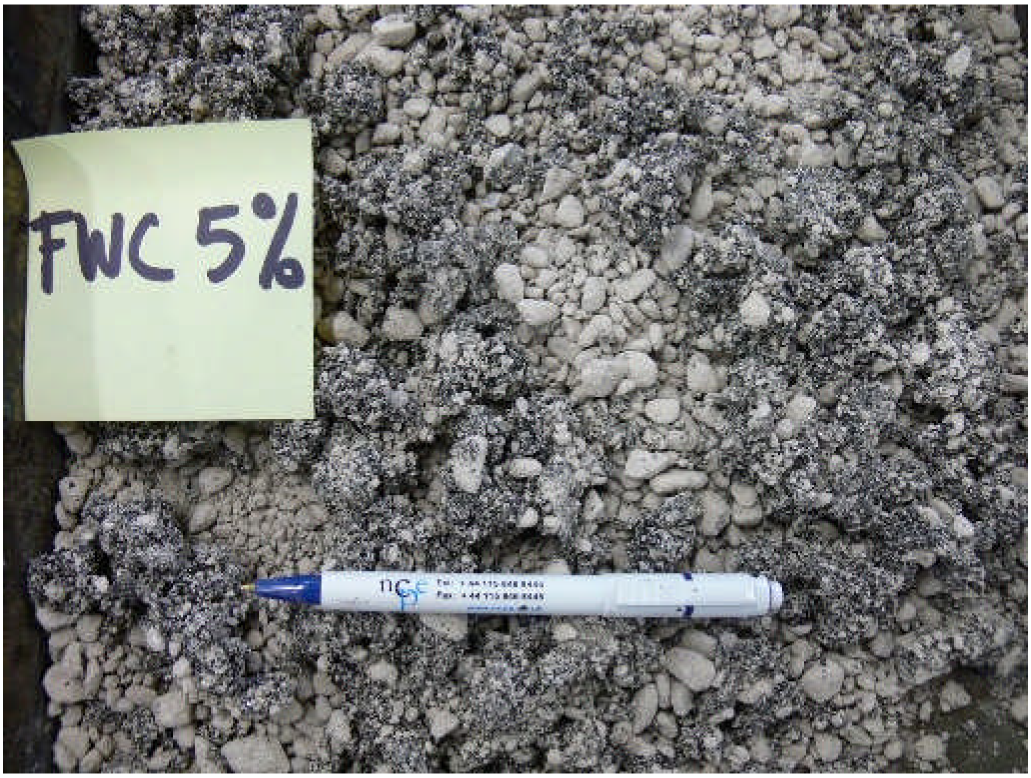

Figure A.6.8 – Appearance of binder distribution at 2 seconds mixing time of foamed asphalt mixture produced at a FWC of 5%.

Figure A.6.9 – Appearance of binder distribution at 2 seconds mixing time of foamed asphalt mixture produced at a FWC of 10%.

Figure A.6.10 – Appearance of binder distribution of foamed asphalt mixture produced using filler only.

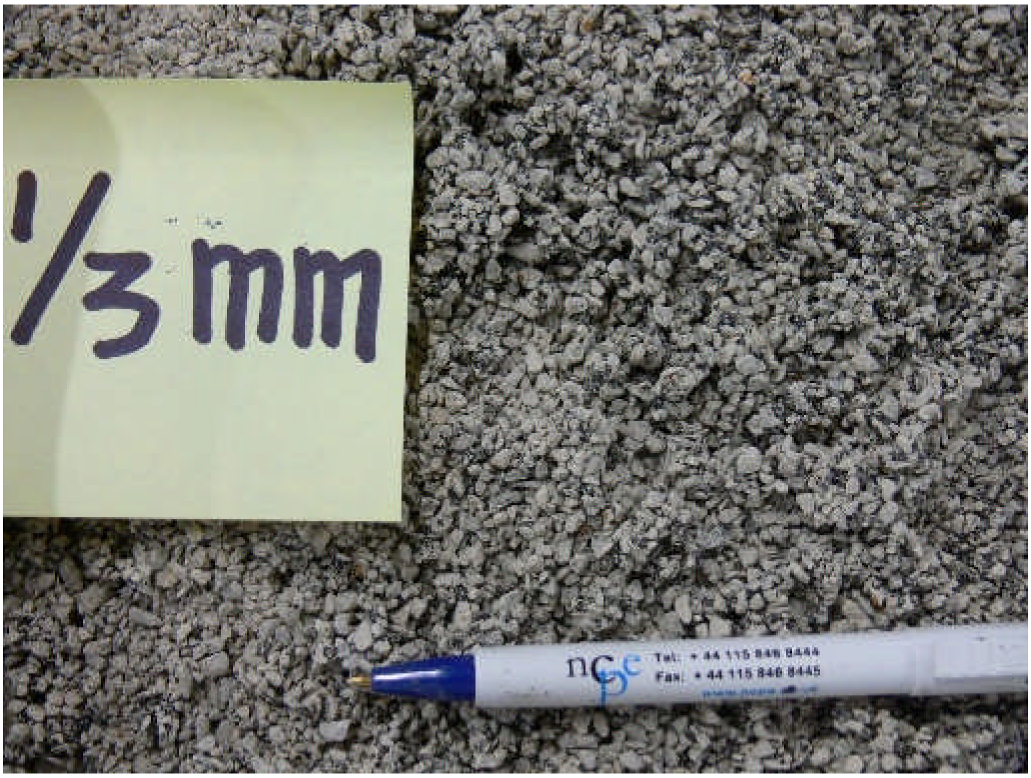

Figure A.6.11 – Appearance of binder distribution of foamed asphalt mixture produced using single size aggregates (1-3mm).

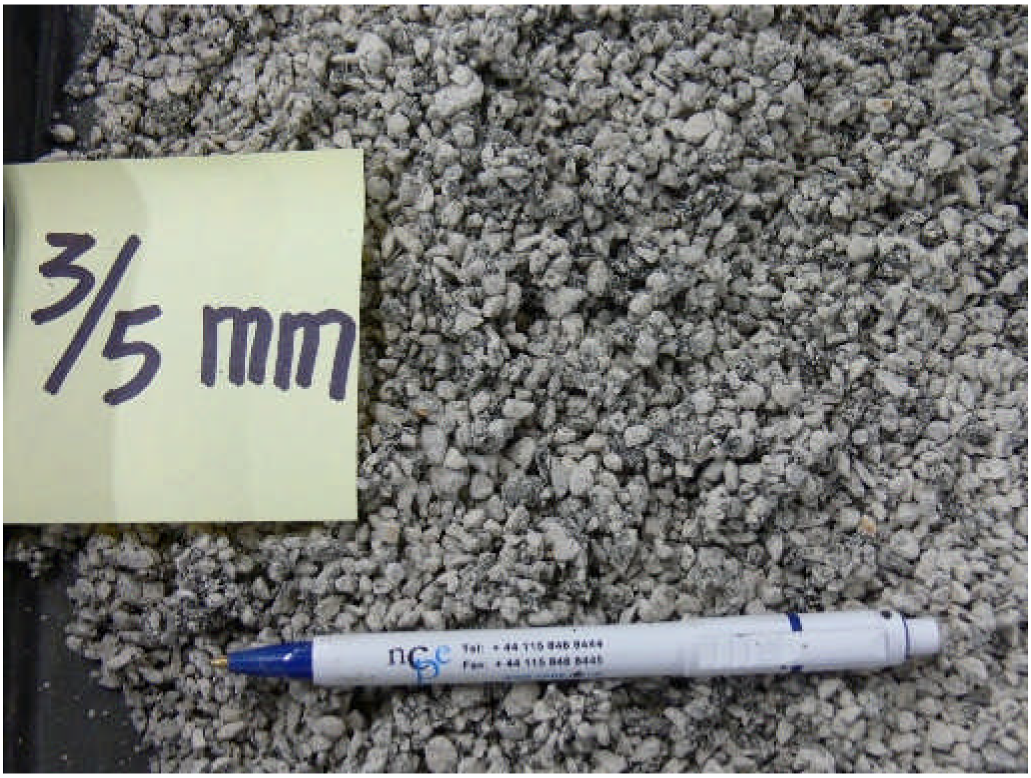

Figure A.6.12 – Appearance of binder distribution of foamed asphalt mixture produced using single size aggregates (3-5mm).

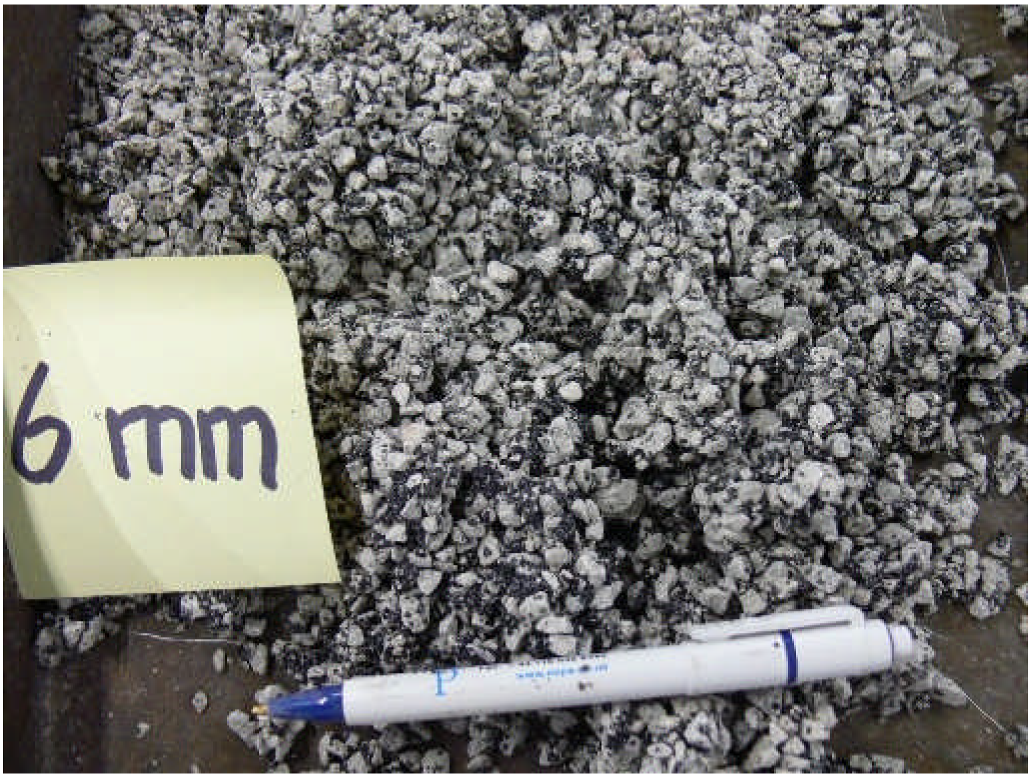

Figure A.6.13 – Appearance of binder distribution of foamed asphalt mixture produced using single size aggregates (6mm).

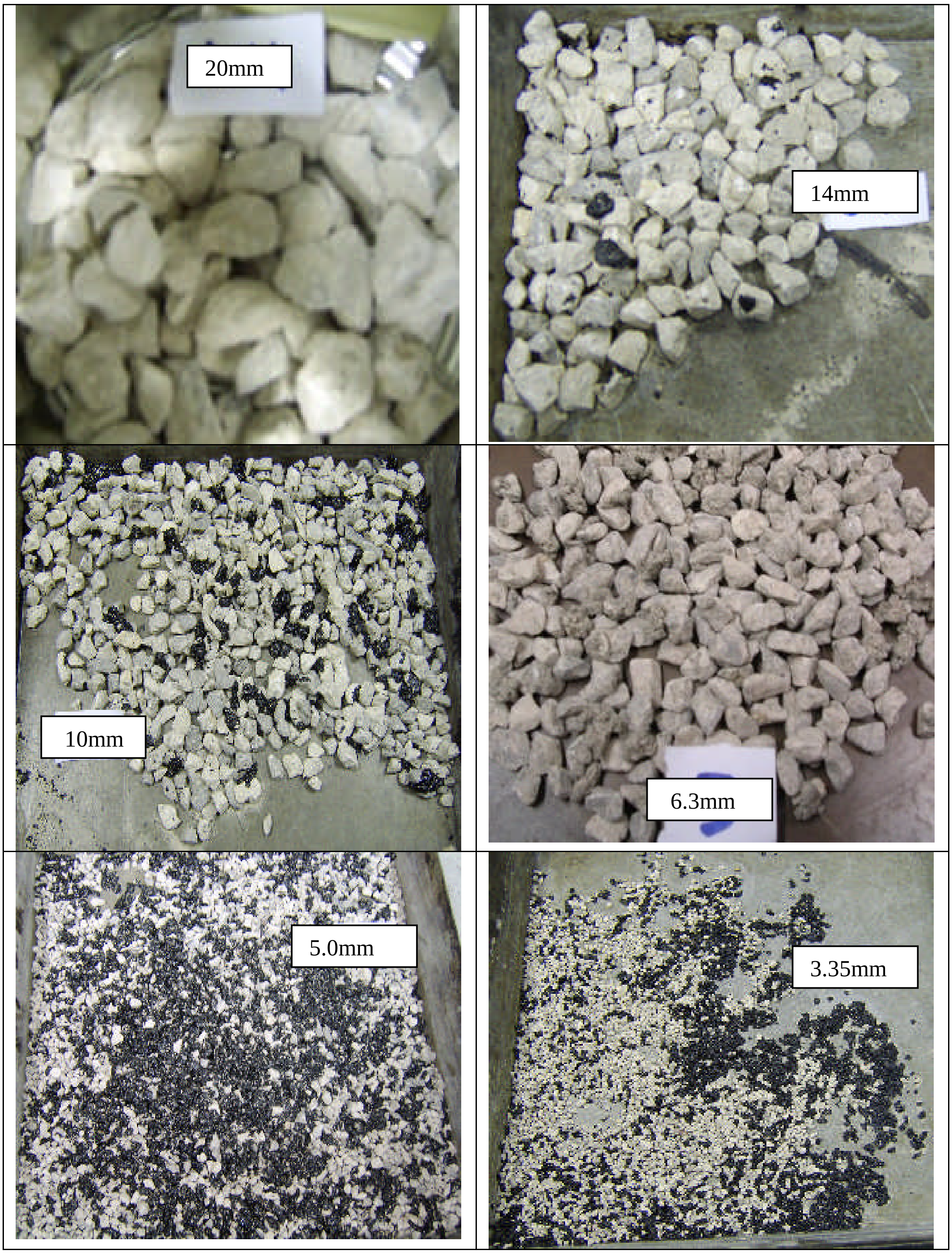

Figure A.6.14 – Binder distribution across the aggregate phase for each fraction

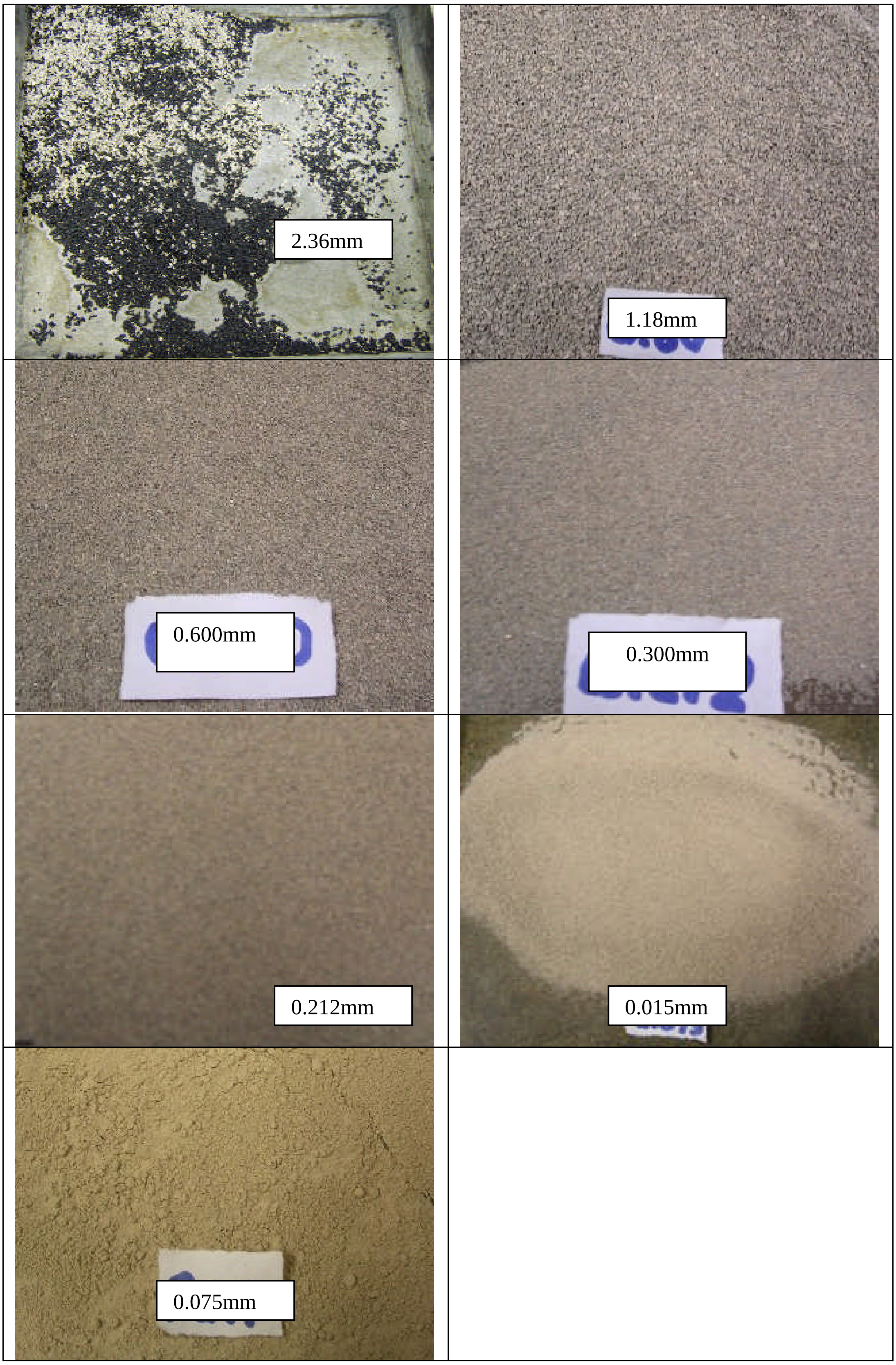

Figure A.6.14 – Binder distribution across the aggregate phase for each fraction (continued)

Table B.4.1 – Calculation of foamed bitumen temperature for Scenario 1

SCENARIO 1: FOAMED BITUMEN SPRAYED IN AIR

Qw = Qb

Qw= heat energy needed by water/steam = Mw*Sw*(100-Tw) + Ms*Ls + Ms*Ss*(T-100)

Qb= heat energy transferred by bitumen = Mb*Sb*(Tb-T)

Mb (Mass of bitumen)	500 g	
Sw (Specific heat of water)	1 cal/g	4.1858 J/g
Ls (Laten heat of steam)	539.4 cal/g	2257.82 J/g
Ss (Specific heat of steam)	0.5017 cal/g	2.1000 J/g
Sb (Specific heat of bitumen)	0.5 cal/g	2.0929 J/g
Tw (Temperature of water)	20 ^{o}C	
kw (Thermal conductivity of water)	0.58 J/s.m.^{o}C	
kb (Thermal conductivity of bitumen)	0.17 J/s.m.^{o}C	

Temp (C)	180	170	160	150	140
Qb100 (J)	83,716	73,252	62,787	52,323	41,858

Bitumen	Vol bit, mm3	500000
	r droplet,mm	1
	n bit droplet	119366
	A surface 1 droplet,mm2	12.566
	Total suface droplet,mm2	2E+06

					Bitumen temp (C)	180		Bitumen temp (C)	170		Bitumen temp (C)	160		Bitumen temp (C)	150		Bitumen temp (C)	140	
FWC	Mw=Ms (g)	Qw1(J)	Qw2 (J)	Qw1+Qw2 (J)	Tf	Qb (J)	Steam (g)	Tf	Qb (J)	Steam (g)	Tf	Qb (J)	Steam (g)	Tf	Qb (J)	Steam (g)	Tf	Qb (J)	Steam (g)
0	0	-	-	-	180	-	0	170	-	0	160	-	0	150	-	0	140		0
1	5	1,674	11,289	12,963	167	13,666	5	157	13,562	5	147	13,458	5	137	13,354	5	127	13,250	5
2	10	3,349	22,578	25,927	154	27,064	10	144	26,858	10	135	26,652	10	125	26,446	10	115	26,240	10
3	15	5,023	33,867	38,890	142	40,200	15	132	39,894	15	122	39,589	15	112	39,283	15	103	38,977	15
4	20	6,697	45,156	51,854	129	53,083	20	120	52,679	20	110	52,276	20	100	51,872	20	100	41,858	15.57
5	25	8,372	56,446	64,817	117	65,720	25	108	65,220	25	100	62,787	24.10	100	52,323	19.47	100	41,858	14.83
6	30	10,046	67,735	77,781	105	78,118	30	100	73,252	27.99	100	62,787	23.36	100	52,323	18.72	100	41,858	14.09
7	35	11,720	79,024	90,744	100	83,716	31.89	100	73,252	27.25	100	62,787	22.62	100	52,323	17.98	100	41,858	13.35
8	40	13,395	90,313	103,707	100	83,716	31.15	100	73,252	26.51	100	62,787	21.88	100	52,323	17.24	100	41,858	12.61
9	45	15,069	101,602	116,671	100	83,716	30.40	100	73,252	25.77	100	62,787	21.13	100	52,323	16.50	100	41,858	11.87
10	50	16,743	112,891	129,634	100	83,716	29.66	100	73,252	25.03	100	62,787	20.39	100	52,323	15.76	100	41,858	11.12

Table B.4.2 – Calculation of foamed bitumen temperature for Scenario 2

SCENARIO 2: FOAMED BITUMEN SPRAYED INTO A STEEL CYLINDER VESSEL

Qw + Qv = Qb

Qw= heat energy of water	= Mw*Sw*(100-Tw) + Ms*Ls + Ms*Ss*(T-100)
Qv= heat energy of vessel	= Mv*Sv*(T-Tv)
Qb= heat energy from bitumen	= Mb*Sb*(Tb-T)

Mb (Mass of bitumen)	500	g			Mass of cylinder (Mv,g)	1200	g
Sw (Specific heat of water)	1	cal/g	4.1858	J/g	A (Area of cylinder)	3.87E-01	m2
Ls (Laten heat of steam)	539.4	cal/g	2257.82	J/g	Thickness of cylinder wall	3.50E-04	m2
Ss (Specific heat of steam)	0.5017	cal/g	2.1000	J/g	Thermal conductivity of cylinder	16	J/s.mC
Sb (Specific heat of bitumen)	0.5	cal/g	2.0929	J/g	Thickness of foam (radius of cylinder),m	0.14	m
Sv (Specific heat of vessel)	0.112	cal/g	0.4688	J/g	Thermal conductivity of foam steam	0.17	J/s.mC
Tw (Temperature of water)	20	C			Tv (Temp of vessel)	20	C

		Bitumen temperature at 180 C								Foam temperature at 1 minute				Tfe (Equi. final temaperature of foam)			
FWC	Mw=Ms (g)	Tfe (C)	Qtransfer (J)	Contact area (m2)	Q/t cylinder (J/s)	Q/t foam (J/s)	Transfer time (min)	Energy in 1mnt (J)	Tf 1min (C)	Tb=170 C	Tb=160 C	Tb=150 C	Tb=140 C	Tb=170 C	Tb=160 C	Tb=150 C	Tb=140 C
0	0	124	58,610	1.29E-02	94,317	2.5	389.90	150	180	170	160	150	140	118	111	105	98
1	5	116	54,012	6.45E-02	433,094	11.5	78.25	690	166	156	147	137	127	110	103	97	90
2	10	108	49,474	1.81E-01	1,107,005	29.4	28.04	1764	152	143	133	123	114	102	95	89	82
3	15	100	44,995	2.45E-01	1,361,764	36.2	20.73	2170	140	130	120	111	101	94	87	81	74
4	20	92	40,572	3.09E-01	1,545,950	41.1	16.47	2464	127	118	108	99	98	86	79	73	73
5	25	84	36,206	3.48E-01	1,546,994	41.1	14.68	2466	115	106	98	98	98	78	73	73	73
6	30	77	31,894	3.87E-01	1,509,369	40.1	13.26	2406	103	98	98	98	98	73	73	73	73
7	35	73	29,989	4.26E-01	1,556,234	41.3	12.09	2480	98	98	98	98	98	73	73	73	73
8	40	73	30,082	4.51E-01	1,650,551	43.8	11.44	2631	98	98	98	98	98	73	73	73	73
9	45	74	30,174	4.77E-01	1,744,869	46.3	10.85	2781	98	98	98	98	98	74	74	74	74
10	50	74	30,265	5.03E-01	1,839,186	48.9	10.33	2931	97	97	97	97	97	74	74	74	74

Tfe= Equilibrium final temperature of foam	Transfer time= time needed of heat transfer from foam to the cylinder (based on the lower Q/t)
Qtransfer= heat energy transferred from foam to the vessel	Energy in 1 min= the energy transferred during 1 minute
Q/t cylinder= the rate of heat transfer to the cylinder vessel	Tf 1 min= temperature of foam at 1 minute
Q/t foam= the rate of heat transfer from foam	Tb= bitumen temperature

Table B.4.3 - Calculation of foamed bitumen temperature for Scenario 3

SCENARIO 3: FOAM SPRAYED ONTO MOIST AGGREGATES

Qf = Qwa+Qag	Qf= Q foam at temperature of scenario 1, Sf=Ss=Sb
Qwa= heat energy of agg water	= Mw*Sw*(100-Tw) + Ms*Ls + Ms*Ss*(T-100)
Qag= heat energy of agg	= Mag*Sag*(T-Tag)

Mb (Mass of bitumen)	500	g	Ma (Mass of aggregate)	12500	g	kw (Thermal conductivity of water)	0.58	J/s.m.°C
Sw (Specific heat of water)	4.1858	J/g	Mwa (Mass of aggregate water)	575	g	kb (Thermal conductivity of bitumen)	0.17	J/s.m.°C
Ls (Laten heat of steam)	2257.82	J/g	ka (Thermal conductivity of aggregate)	1.3	J/s.m.°C	ks (Thermal conductivity of foam steam)	0.016	J/s.m.°C
Ss (Specific heat of steam)	2.1000	J/g	A (Surface area of aggregate)	55.272	m2	A (Surface area of uncoated aggregate)	2.49E+01	m2
Sb (Specific heat of bitumen)	2.0929	J/g	A (Surface area of coated aggregate)	30.323	m2			
Sa (Specific heat of agg)	0.8	J/g	Lw (Average of water thickness)	1.33E-05	m			
Tw (Temperature of water)	20	C						
Tb (Temperature of bitumen)	180	C						
Ta (Temperature of wet aggrega	20	C						

FWC	Mw =Ms (g)	Ti (C)	Q transfer (J)	Tm (C)	Q transfer (J)	Tf (C)	Q transfer (J)	Q/t foam to water (J/s)	Q/t water (J/s)	Transfer time f-w (s)	Q/t foam to coated agg (J/s)	Q/t coated agg (J/s)	Transfer time f-cagg (s)	Q/t total agg (J/s)	Transfer time to final (minutes)	
0	0	91	152,857	63	51,792	32	131,169									
1	5	90	132,383	63	51,346	32	130,423									
2	10	89	112,073	63	50,908	32	129,690									
3	15	89	91,924	63	50,477	32	128,968									
4	20	88	71,931	62	50,053	32	128,256	6.18E+05	5.44E+07	0.12	295917	3502	14.29	905.37	2.36	
5	25	88	52,088	62	49,637	32	127,556									
6	30	87	32,393	62	49,227	32	126,867									
7	35	87	24,035	62	48,824	32	126,187									
8	40	86	25,161	61	48,428	32	125,518									
9	45	85	26,283	61	48,038	32	124,858									
10	50	85	27,399	61	47,654	32	124,209									

Table B.4.4 - Calculation to estimate the contact area and thickness of bitumen (for Scenario 3)

Use the result data of binder distribution investigation (dry sieving, bitumen Pen 50/70, FWC of 4%, bitumen temperature of 180 C)													
Fraction	# 20	# 14	# 10	# 6.3	# 5	# 3.35	# 2.36	# 1.18	#0.6	#0.3	#0.212	#0.15	#0.075
Bitumen (g)	0.00	0.70	3.20	2.30	8.20	9.00	20.90	11.50	2.70	0.80	0.30	0.80	0.00
Uncoated agg (g)	366.20	217.60	82.90	49.70	101.70	70.00	117.60	77.40	38.80	11.30	9.20	16.10	39.90
Coated agg (g)	0.00	2.97	17.90	11.80	23.20	35.40	72.00	40.20	7.10	1.10	1.60	1.20	0.00
Uncoated agg (mm2)	1.37E+05	1.18E+05	1.25E+05	1.65E+05	1.16E+05	1.05E+05	1.23E+05	1.58E+05	1.45E+05	9.10E+04	9.72E+04	2.55E+05	1.19E+06
Coated agg (mm2)	0.00E+00	2.33E+04	6.07E+04	1.22E+05	2.39E+05	5.35E+05	1.38E+06	8.21E+05	1.61E+05	1.78E+04	2.54E+04	3.58E+04	0.00E+00
Area of coated agg (%)	0.00	0.37	0.97	1.95	3.82	8.56	22.11	13.13	2.57	0.29	0.41	0.57	0.00
Bitumen thickness (mm)	0	0.15	0.18	0.05	0.09	0.04	0.03	0.03	0.03	0.07	0.02	0.03	0
Area of coated agg (for agg mass of 12500g)	0.00E+00	2.06E-01	5.37E-01	1.08E+00	2.11E+00	4.73E+00	1.22E+01	7.26E+00	1.42E+00	1.58E-01	2.24E-01	3.17E-01	0.00E+00

Table B.4.5 - Calculation to estimate the contact area and thickness of aggregate (for Scenario 3)

Use the result data of binder distribution investigation (bitumen Pen 50/70, FWC of 4%, bitumen temperature of 180 C)													
Size,mm	Mass of aggregate (g)			Total surface area of aggregate (mm2)			% coated aggregate		% uncoated aggregate		Agg thickness per size	Agg. surface area for a total agg.mass of 12500g (m2)	
	Coated	Uncoated	Total	Coated	Uncoated	Total,mm	mass	area	mass	area	rad (mm)	coated	uncoated
20	0	362	362	0	40522	40522	0	0	25.62	0.65	10	0.000	0.359
14	0	215	214.8	0	34350	34350	0	0	15.20	0.55	7	0.000	0.304
10	0	79	79.3	0	17754	17754	0	0	5.61	0.28	5	0.000	0.157
6.3	8.9	45	53.6	3163	15885	19048	0.63	0.05	3.16	0.25	3.15	0.028	0.141
5	6.6	99	105.9	2955	44463	47418	0.47	0.05	7.03	0.71	2.5	0.026	0.393
3.35	8.4	68	76.4	5614	45444	51058	0.59	0.09	4.81	0.73	1.675	0.050	0.402
2.36	12.4	117	129.8	11953	111181	123134	0.88	0.19	8.31	1.78	1.18	0.106	0.984
1.18	18	77	95.3	34720	146092	180812	1.27	0.56	5.47	2.34	0.59	0.307	1.293
0.6	25.5	39	64.6	96269	144776	241045	1.80	1.54	2.77	2.32	0.3	0.852	1.281
0.3	11.6	11	22.7	87313	82090	169403	0.82	1.40	0.79	1.31	0.15	0.772	0.726
0.212	10.9	9	20.2	116164	97156	213320	0.77	1.86	0.66	1.56	0.106	1.028	0.860
0.15	18.9	15	34.2	285075	225373	510448	1.34	4.56	1.08	3.61	0.075	2.522	1.994
0.075	93.27	61	154.07	2784179	1814925	4599104	6.60	44.57	4.30	29.05	0.0375	24.632	16.057
Total	214.47	1198.4	1412.87	3427405	2820011	6247416	15.18	54.86	84.82	45.14		30.323	24.949
Proportior	15%	85%		55%	45%								55.272

Table B.6.1 – Bitumen and aggregate content in the each fraction for foamed asphalt mixture produced using bitumen Pen 50/70 at Bitumen temperature of 180^{o}C and FWC of 4%.

Fraction	# 20	# 14	# 10	# 6.3	# 5	# 3.35	# 2.36	# 1.18	#0.6	#0.3	#0.212	#0.15	#0.075	Total	% total
Fraction mass (g)	366.2	221.2	104.0	63.8	133.1	114.4	210.5	129.1	48.6	13.2	11.1	18.1	39.9	1473.2	100.00
Bitumen in fraction (g)	0.00	0.70	3.20	2.30	8.20	9.00	20.90	11.50	2.70	0.80	0.30	0.80	0.00	60.40	4.10
Aggregate in fraction (g)	366.2	220.5	100.8	61.5	124.9	105.4	189.6	117.6	45.9	12.4	10.8	17.3	39.9	1412.8	95.90
Bitumen (% of fraction aggregate mass)	0.00	0.32	3.08	3.61	6.16	7.87	9.93	8.91	5.56	6.06	2.70	4.42	0.00		
Bitumen (% of total aggregate mass)	0.00	0.05	0.22	0.16	0.56	0.61	1.42	0.78	0.18	0.05	0.02	0.05	0.00	4.10	
Cum% bitumen(by mass of total aggregate)	0.00	0.05	0.26	0.42	0.98	1.59	3.01	3.79	3.97	4.03	4.05	4.10	4.10		
Aggregate (% of total aggregate mass)	25.92	15.61	7.13	4.35	8.84	7.46	13.42	8.32	3.25	0.88	0.76	1.22	2.82	100.00	

Note: (1) Fraction resulting from dry sieving (BS 812-103.1: 1985)
(2) Bitumen content determined using soluble binder content test (BS EN 12697-1: 2000)

Table B.6.2 – Aggregate size distribution for each fraction resulted from wet sieving for foamed asphalt mixture produced using bitumen Pen 50/70 at bitumen temperature of 180°C and FWC of 4%.

Fraction	# 20	# 14	# 10	# 6.3	# 5	# 3.35	# 2.36	# 1.18	#0.6	#0.3	#0.212	#0.15	#0.075	Coated (g)	Uncoated (g)	Total (g)
																0
20	**362**														362	362
14	0	**214.8**													214.8	214.8
10	0	0	**79.3**												79.3	79.3
6.3	0	0	8.9	**44.7**										8.9	44.7	53.6
5	0	0.3	1.4	4.9	**99.3**									6.6	99.3	105.9
3.35	0	0.1	0.6	0.5	7.2	**68**								8.4	68	76.4
2.36	0.2	0.6	1	0.4	1.5	8.9	**117.2**							12.6	117.2	129.8
1.18	0.3	0.6	1.4	0.6	2.1	2.5	10.8	**77**						18.3	77	95.3
0.6	0.3	0.4	1.6	0.9	2.6	3.5	7.9	8.6	**38.8**					25.8	38.8	64.6
0.3	0.1	0.1	0.7	0.3	1.3	1.6	4.1	2.2	1.3	**11**				11.7	11	22.7
0.212	0.1	0.2	0.7	0.3	1	1.5	3.6	2.3	0.6	0.7	**9.2**			11	9.2	20.2
0.15	0.2	0.2	0.9	0.5	1.7	2.6	6.6	4.1	0.7	0.1	1.5	**15.1**		19.1	15.1	34.2
0.075	0	0.47	0.7	3.4	5.8	14.8	39	23	4.5	0.3	0.1	1.2	**39.9**	93.27	39.9	133.17
washed	3	2.8	3.6	5	2.4	2	0.4	0.4	0	0.3	0	1	0		20.9	20.9
Total Aggregate	**366.20**	**220.57**	**100.80**	**61.50**	**124.90**	**105.40**	**189.60**	**117.60**	**45.90**	**12.40**	**10.80**	**17.30**	**39.90**	***215.67***	***1197.20***	***1412.87***
												Percentage		**15.26%**	**84.74%**	

Note: An example to define the uncoated and coated particle is follows. The aggregate mass from fraction #14 retained on sieve size 10mm is classified as uncoated, whereas all aggregate particles passing sieve size 10mm are classified as coated.

Table B.6.3 – Proportion of coated and uncoated aggregate for foamed asphalt mixture produced using bitumen Pen 50/70 at Bitumen temperature of 180°C and FWC of 4%.

Size,mm	Mass of aggregate (g)			Volume of aggregate (mm^3)			number of particle aggregate			Total surface area of aggregate (mm^2)		
	Coated	Uncoated	Total	Coated	Uncoated	Total	Coated	Uncoated	Total	Coated	Uncoated	Total
20	0	362	362	0	135075	135075	0	3.22E+01	3.22E+01	0	4.05E+04	4.05E+04
14	0	214.8	214.8	0	80149	80149	0	5.58E+01	5.58E+01	0	3.43E+04	3.43E+04
10	0	79.3	79.3	0	29590	29590	0	5.65E+01	5.65E+01	0	1.78E+04	1.78E+04
6.3	8.9	44.7	53.6	3321	16679	20000	2.54E+01	1.27E+02	1.53E+02	3.16E+03	1.59E+04	1.90E+04
5	6.6	99.3	105.9	2463	37052	39515	3.76E+01	5.66E+02	6.04E+02	2.96E+03	4.45E+04	4.74E+04
3.35	8.4	68	76.4	3134	25373	28507	1.59E+02	1.29E+03	1.45E+03	5.61E+03	4.54E+04	5.11E+04
2.36	12.6	117.2	129.8	4701	43731	48433	6.83E+02	6.35E+03	7.04E+03	1.20E+04	1.11E+05	1.23E+05
1.18	18.3	77	95.3	6828	28731	35560	7.94E+03	3.34E+04	4.13E+04	3.47E+04	1.46E+05	1.81E+05
0.6	25.8	38.8	64.6	9627	14478	24104	8.51E+04	1.28E+05	2.13E+05	9.63E+04	1.45E+05	2.41E+05
0.3	11.7	11	22.7	4366	4104	8470	3.09E+05	2.90E+05	5.99E+05	8.73E+04	8.21E+04	1.69E+05
0.212	11	9.2	20.2	4104	3433	7537	8.23E+05	6.88E+05	1.51E+06	1.16E+05	9.72E+04	2.13E+05
0.15	19.1	15.1	34.2	7127	5634	12761	4.03E+06	3.19E+06	7.22E+06	2.85E+05	2.25E+05	5.10E+05
0.075	93.27	60.8	154.07	34802	22687	57489	1.58E+08	1.03E+08	2.60E+08	2.78E+06	1.81E+06	4.60E+06
Total	215.67	1197.2	1412.9	80474	446716	527190	1.63E+08	1.07E+08	2.7E+08	3.43E+06	2820011	6.25E+06
Percentage	15%	85%		15%	85%		60%	40%		55%	45%	

Note: (1) Aggregate is assumed to be spherical in shape
(2) Density of aggregate is assumed to be 2680 kg/m^3 for all sizes