Foamed Bitumen Stabilisation for New Zealand Roads

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An Important Note for the Reader

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Contents

Contents *continued:*

Abbreviations and Acronyms

ϕ = Angle of internal friction
θ = Bulk stress
ν = Poisson's ratio
$\tau_{1/2}$ = Half-life time of foam (in seconds)
σ_{1f} = Major principal stress at failure
σ_3 = Confining stress
σ_{df} = Deviatoric stress at failure
ε_p = Plastic strain
AASHTO = American Association of State Highway & Transportation Officials
BC = Binder content
CBR = Californian Bearing Ratio
EOBC = Estimated optimum binder content
ER = Expansion ratio
ER_a = Actual expansion ratio
ER_m = Measured expansion ratio
ESAL = Equivalent single axle loads
FI = Foam index
HLT = Half-life time
HVS = Heavy vehicle simulator
ITS = Indirect tensile strength
LVDT = Linear variable differential transformer
MC = Moisture content
MMC = Optimum pre-moulding moisture content
M_r = Resilient Modulus
OFC = Optimum fluid content of foamed treated mix
OMC = Modified AASHTO aggregate optimum moisture content
OMMC = Optimum mixing moisture content
PF = Percentage of materials passing 75µm sieve
SA = South Africa
Transfund = Transfund New Zealand
Transit = Transit New Zealand
UCS = Unconfined compressive strength
WC = Water content
G_{mb} = Bulk specific gravity of the compacted mix
G_b = Specific gravity of bitumen
P_b = Percent of bitumen by weight of total mix
V_b = Percent of bitumen by volume of the compacted mix

Executive Summary

Although foamed bitumen stabilisation is gaining wide acceptance in several countries around the world, it has not been practised in New Zealand. Part I of this report is a review of the technical literature up to 2002. Part II consists of the results of a preliminary experimental study, carried out in 2002, on the use of the foamed bitumen stabilisation method with New Zealand materials. The method offers the advantages of reduced transverse shrinkage cracking and a fast stabilisation process that minimises traffic delays. The performance of foam-stabilised aggregates is comparable with, or even better than, that of bitumen emulsion-stabilised materials.

The work forms part of a larger project aimed at investigating the feasibility and potential application of the foamed bitumen stabilisation technique in New Zealand, in particular as a means of improving marginal quality aggregates and promoting its use in urban areas. In the second phase the proposal is to study properties of the foamed bitumen stabilisation process in more detail. The use of foamed bitumen to recycle or manufacture asphalt mix for wearing course is not covered in this report.

Part I: Literature Review

The literature review covers topics related to methods of foamed bitumen characterisation, testing methods, engineering properties, design of foamed bitumen mixes, and recent field applications of the stabilisation technique.

Adding a small amount of water or steam (about 2%) to hot bitumen results in the formation of a foam that expands many times the original volume of the bitumen. The low viscosity of the foam allows it to mix easily with aggregates, and the foam breaks (i.e. separates into bitumen and water) preferentially coating the fine aggregate particles in the mix. Foamed bitumen mixes are distinctive in that, as opposed to hot mix and emulsion mixes, large aggregate particles are only partially coated, while the bitumen-coated fines form a mortar that binds the mixture together.

Bitumen Foam

Bitumen foam is usually characterised in terms of its expansion ratio (ER) and half-life ($\tau_{1/2}$). The expansion ratio is the ratio of the maximum measured volume of the foamed bitumen to the final bitumen volume once the foam has dissipated. High ER values give low viscosity foam that disperses well into the mix. Half-life ($\tau_{1/2}$) is the time in seconds it takes for the foamed bitumen to settle to half of its maximum attained volume. Long half-lives allow more time for the mixing process.

ER and $\tau_{1/2}$ are inversely related, in that increasing the amount of foaming water increases ER and decreases $\tau_{1/2}$. An optimum water content that optimises both parameters can be determined which, for practical applications, an ER of >10 and a $\tau_{1/2}$ >10 seconds is recommended.

A Foam Index (FI) has been suggested recently as a more useful measure of bitumen foaming characteristics. It takes into account both ER and $\tau_{1/2}$ by measuring the area under the ER decay curve. Foaming characteristics are affected by bitumen type, grade and additives, in particular by anti-foaming compounds which are often added to bitumens that have been produced by solvent precipitation processes.

Foamed Bitumen Mixes

Foamed bitumen can be used with a wide range of aggregate types (including marginal materials with high plasticity indices) and with cold, moist aggregates. In fact, an optimum moisture content is essential for good dispersion of the foam in the mix. The fines (<75μm) content greatly affects mix strength and should exceed 5% of the volume of the mix. Bitumen content usually lies in the 2-5% range and needs to be optimised depending on the fines content. Mix strength also appears to be affected by bitumen grade but relatively little work has been reported. Typically, bitumens used in hot mix asphalt (HMA) are also used in foamed bitumen stabilisation.

Foamed Bitumen Mix Properties

Foamed bitumen stabilisation increases the resilient modulus of the mix compared to the unbound material, and mix properties are dependent on the moisture content. Considerable work has been carried out into developing laboratory curing (drying) conditions to simulate field conditions. Usually, at least three days heating at 40°C or 60°C is used to cure laboratory samples.

The moduli of foam-treated materials are generally lower than those of HMA, but can approach or exceed those of lime- or cement-treated materials. The properties are moisture sensitive where soaked specimens can have moduli 30-50% lower than those of dry specimens. Moisture sensitivity can be reduced by the addition of 1-2% Portland cement.

The moduli and tensile strength of foamed bitumen mixes are temperature sensitive, but the sensitivity is less than that of HMA. At temperatures above 30°C, the moduli of foam-treated materials can exceed those of the equivalent HMA.

Although the fatigue resistance of foamed bitumen mixes is often stated in the literature as being superior to that of conventionally stabilised materials, little data has been reported and this subject is in need of further research in the second phase of this project.

Foamed Bitumen Mix Design

Laboratory design methods for foamed bitumen mixes have been recommended in various studies, including the new South African design method, and they are outlined in this report. The constitutive models that govern the behaviour of foamed-treated mixes are also discussed, but only very limited studies have been conducted in this area.

Field Performance of Foamed Bitumen Mixes

A major advantage of foamed bitumen stabilisation is that the pavement can carry traffic immediately after compaction. Although the strength of the mix increases over a period of months as moisture evaporates, the curing rate is more rapid than that of emulsion mixes. Many field applications of foamed bitumen stabilisation have been reported and some of the more recent studies are discussed in this report.

Environmental Effects of Foamed Bitumen Mixes

A detailed comparative environmental assessment of various stabilisation techniques available has not been reported. The foamed bitumen stabilisation process however, does not have any particular adverse environmental impacts. The stabilisation process does not involve the emission of volatiles, such as in cutback stabilisation, or of corrosive dust as when using lime or cement stabilisation, both of which are particularly important to consider in urban areas. Foamed bitumen stabilisation is likely to require less energy to use compared to bitumen emulsion or cement treatment.

Part II: Experimental Investigation

The experimental work presented in Part II compared the foaming characteristics of two grades of bitumen (180/200 and 80/100) from two sources (I and II), currently in use in New Zealand, and also the resilient moduli of mixes made using a modified AP-20 gradation aggregate. Manufacturing foamed bitumen mixes using locally sourced materials presents no difficulties and gives rise to high stiffness mixes.

The foamability of the two grades of bitumen differed according to the characterisation method used, which is based on expansion ratio, half-life, and foam index parameters. Also, within each type, it was obvious that softer grades provide better foamability results and also provide better mechanical properties for the foam mixes. However, although Source I bitumen showed inferior properties compared to Source II bitumen in terms of its foamability properties, the mechanical properties of the mixes prepared from Source I actually showed better properties than mixes prepared from Source II bitumen. This indicates that the current system for characterising foam properties is inadequate and more research in this area is needed.

The resilient moduli of the mixes showed that foamed bitumen stabilisation results in significant improvements in stiffness using an aggregate gradation based on AP-20 (with additional fly ash, or fly ash and cement, as filler). Moduli increased with increasing room temperature curing time, to values comparable to, or in excess of, that of HMA. Even after only 24 hours the moduli obtained were typically sufficient to sustain traffic loads in the field without significant problems.

A cost-analysis exercise comparing nine different stabilisation methods was carried out. The results of this analysis showed that foam bitumen stabilisation using high quality aggregates and about 2% cement is very competitive even compared to unbound materials, because of the much reduced thickness of seal required. Because quantitative data are lacking at this time, environmental benefits and road user effects (such as vehicle operating costs) arising from the various methods used in the analysis could not be estimated. From the literature reviewed, such factors seem likely to increase the competitiveness of the foam stabilisation method relative to other methods. The analysis was carried out assuming that the fatigue behaviours of the foamed bitumen material and HMA are approximately similar.

Summary

Based on the literature discussed in Part I of the report and the results of the feasibility study reported in Part II, foamed bitumen stabilisation appears to be a very cost-competitive alternative to those methods traditionally used in New Zealand. Although foamed bitumen stabilisation involves higher initial material costs than those of cement or lime stabilisation, it offers the advantages of reduced transverse shrinkage cracking. Because of its lower temperature sensitivity, the moduli of foamed bitumen-stabilised materials can be greater than those of HMA at high temperatures. Also the technique appears to offer significant advantages in terms of both speed of construction and amount of disruption to road users.

Abstract

Foamed bitumen stabilisation is gaining wide acceptance internationally for pavement stabilisation, but has not been practised in New Zealand. This report is of a preliminary experimental study, carried out in 2002, to investigate its feasibility and potential applications in this country.

Part I presents a review of the literature (up to 2002) on the use of the foamed bitumen process for pavement stabilisation. The review covers topics related to methods of foamed bitumen characterisation, testing methods, engineering properties, design of foamed bitumen mixes, and recent field applications of the stabilisation technique.

Part II consists of preliminary experimental work carried out to explore the feasibility of using the technique in New Zealand. The foaming characteristics of two grades of bitumen from two sources currently in use in New Zealand and the resilient moduli of mixes made using a modified AP-20 gradation aggregate are compared.

Part I: Literature Review

1. Introduction

1.1 Scope

Pavement stabilisation has been commonplace in New Zealand since the 1960s. Although bitumen (using an asphalt plant) and bitumen emulsion have been used for pavement stabilisation, the most common methods are lime and cement stabilisation (Hudson 1996).

Foamed bitumen stabilisation of basecourse materials is gaining wide acceptance in many countries around the world, but it has not been practised in New Zealand. This report forms part of a large project aimed at investigating the feasibility and potential applications of foamed bitumen stabilisation in this country, in particular as a means of improving marginal quality aggregates and for promoting its use in urban areas.

This report covers the technical literature on the characterisation of bitumen foam, the design and physical properties of foamed bitumen basecourse mixes, and field experience. The review is not intended as a practical guide to the use of foamed bitumen stabilisation in the field. Also the use of foamed bitumen to produce asphalt wearing course or in the recycling of old hot mix asphalt (HMA) as wearing course is not considered.

1.2 Foamed Bitumen and Mixes

Foamed bitumen, also referred to as expanded bitumen, is a hot bituminous binder that has been temporarily converted from a liquid to a foamed state by the addition of a small percentage of water (typically about 2% by weight of bitumen). Bitumen foaming was first investigated as a means of stabilising aggregate mixtures in 1956 by Csanyi (1957, 1962) at Iowa State University. The original process involved injecting steam into hot bitumen to produce the foam. Mobil Oil Australia later modified the process by using water rather than steam. Figure 1.1 shows a schematic of the foaming process of the bitumen, whereas Figure 1.2 shows typical foamed bitumen laboratory equipment.

When cold water droplets come into contact with hot bitumen at a temperature greater than 100°C, heat transfer from the bitumen converts the water droplets into steam. The large molar volume of the steam results in explosive expansion and formation of a foam usually in the order of 10-15 times the volume of the original bitumen. The encapsulated steam will expand until the slightly cooler bitumen holds the bubble through its surface tension. The expanded bubbles can stay stable for a short period of time, often in the order of tens of seconds, before they start collapsing. During this time the foam can easily mix with aggregates and coat the particles with bitumen as it collapses.

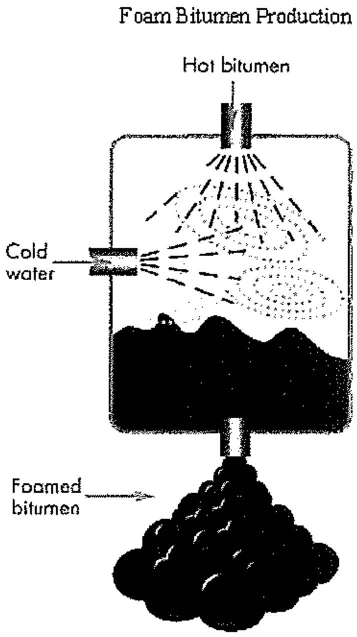

Figure 1.1 Creation of foamed bitumen inside expansion chamber.

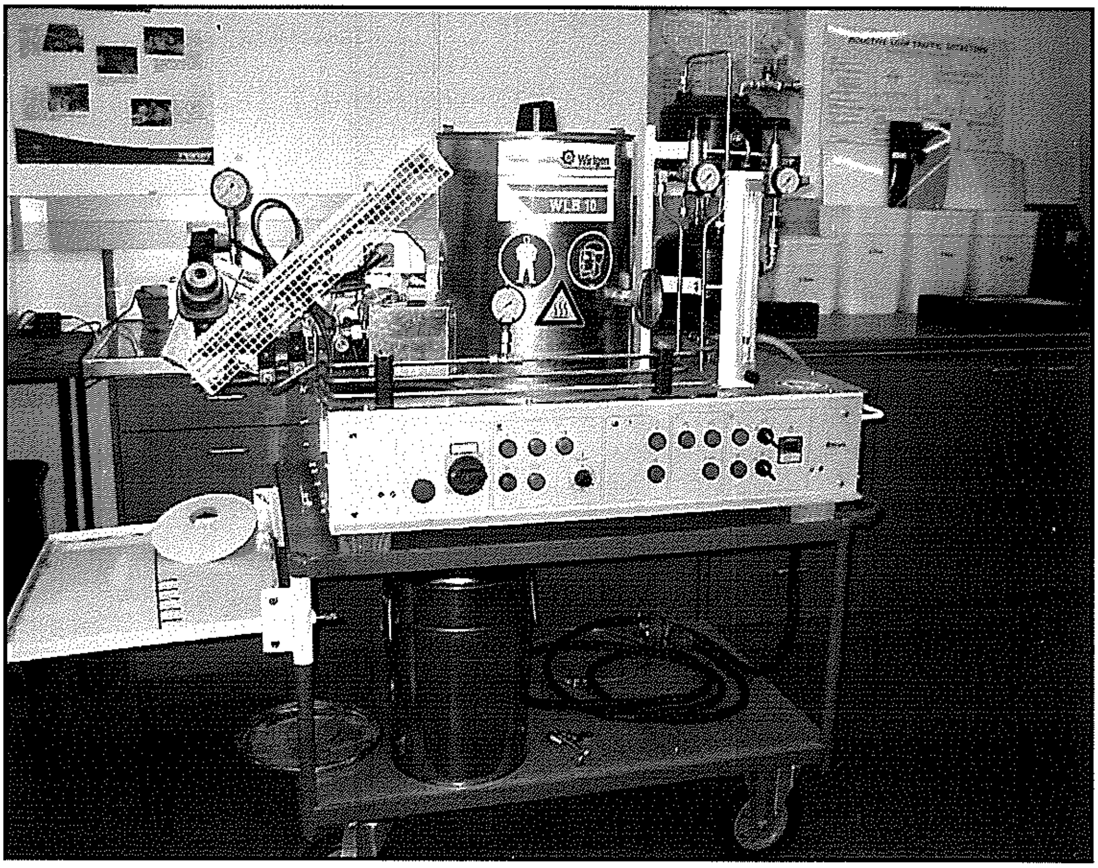

Figure 1.2 University of Canterbury laboratory foamed-bitumen equipment.

There is a distinct difference between foamed bitumen mixtures and asphalt or emulsion-stabilised mixtures in the way the bitumen is dispersed through the aggregates. In the case of asphalt or emulsion mixes, the bitumen tends to coat all particles while direct observation of foamed bitumen mixes reveals that fine particles (particularly those <0.75 mm) are preferentially coated with bitumen leaving larger aggregates substantially free of bitumen (Lee 1981, Castedo-Franco & Wood 1983). The increased modulus of the mix is attributed to the resulting bitumen-fines mortar, acting as a 'glue' for the larger particles (Muthen 1999).

In practice, foaming is usually carried out by introducing water and heated bitumen into an expansion chamber. The resulting foam extrudes through a nozzle and is mixed with aggregate. Foam bitumen mixtures can achieve high stiffness values (>4000 MPa), close to that of cement-treated materials while remaining flexible.

2. Features of Foamed Bitumen-Stabilised Mixes

Foamed bitumen-treated aggregates behave, in general terms, as granular materials rather than as hot mix asphalt and must be treated as such. Features peculiar to foamed bitumen-treated materials as opposed to other stabilising agents include:

1. Foamed bitumen increases the shear strength of unbound aggregates, in some cases to levels close to that produced by lime or cement stabilisation while keeping the mix flexible enough to provide high fatigue resistance.
2. Foamed bitumen provides high moisture resistance because it waterproofs aggregate particles.
3. The foamed bitumen technique provides energy savings because cold moist aggregates can be used.
4. Foamed bitumen is an environmentally friendly technique because no harmful emissions are released. It does not present the problems (particularly in urban areas) caused by corrosive lime or cement dust.
5. Traffic delays are minimal because the road can be opened immediately to traffic after compaction. Time restrictions that apply when working with cementitious binders and the curing time requirements of bitumen emulsion treatments do not apply.
6. The foamed bitumen-stabilisation technique can be used with a wider range of aggregate types than other mix processes.
7. Foamed bitumen can be stockpiled with no risk of binder run-off and without loss of workability since foamed bitumen mixtures remain workable for an extended period of time.
8. The foamed bitumen technique can be used even in adverse weather conditions, such as wet or cold weather, without affecting the quality or the workability of the finished product.

3. Projects Suitable for Foam Bitumen Stabilisation

Stabilisation of pavement materials using foamed bitumen is normally carried out in situ using specially designed machinery that allows aggregate milling, bitumen foaming, and mixing in one continuous operation. In-plant mixing is also practised (e.g. for the construction of new roads) but in most cases is less favourable because of increased transport costs. The in-plant process does, however, provide for greater control of the aggregate properties. Although the foamed bitumen stabilisation technique has been practiced in many countries (Ruckel et al. 1980), the technology has gained greatest acceptance in Australia and in particular South Africa.

Although the foamed bitumen-stabilisation process provides scope for rehabilitation of a wide range of material types and in different circumstances, in some situations the process will be unsuitable, either on economic or technical grounds. The South African Interim Technical Guidelines (South Africa (SA) Transportek 2002) provides a list of factors affecting the suitability of particular projects for foamed bitumen stabilisation.

Generally, roads with thin pavements and poor structural capacity can seldom be adequately rehabilitated using only in-situ recycling (although this limitation is not limited to foamed bitumen). Furthermore, the variability (in aggregate gradation or moisture content) of the pavement layers plays a significant role in determining the suitability of the project for foamed bitumen stabilisation. Too great a variability will necessitate the constant adjustment of the quantities of bitumen and water used. Pavements that are severely cracked through fatigue may also present problems when milling with older recycling machines. The grading so obtained may be unsuitable for foamed bitumen treatment. Table 3.1 summarises a list of the factors affecting the suitability of specific projects for foamed bitumen treatment.

Further information on the selection of the appropriate stabilisation method for a given application is given in the Austroads *Guide to Stabilisation in Roadworks* (1998a).

Table 3.1 Factors affecting the suitability of a project for foam bitumen treatment (SA Interim Technical Guidelines 2002).

Favourable	Unfavourable
Layers of uniform material and thickness	Variable layers with mixed materials
Existing pavement has deep structure and problems are due to base layer	Existing pavement has shallow structure, subgrade problems
Little patching or rut filling	Significant patching and/or rut filling
Crack intensity is not too severe	Severe cracking, small diameter blocks in thin asphalt pavements (50–100 mm) when using older recycling machines
Thin asphalt surfacing with granular or lightly cement-treated base or sub-base	Deep asphalt pavements (AC>150 mm)
Little additional material is necessary	Significant additional material required
Uniform subgrade moisture	Variable subgrade moisture
Few buried services	Much road furniture and services, e.g. manholes
Consistent camber of pavement layers	Variable cambers of pavement layers
Few cuttings and embankments	Many cuttings and embankments

4. Characterisation of Foamed Bitumen

Foamed bitumen is usually characterised by two empirical indices: expansion ratio and half-life time. The expansion ratio of the foam is defined as the ratio between the maximum volume achieved during expansion and the final bitumen volume once the foam has dissipated. The expansion ratio should be high enough to yield low viscosity foam capable of providing a good coating to as much aggregate as possible after the introduction of the foam into the mixer.

The half-life is the time, in seconds, between the moment the foamed bitumen achieves its maximum volume and the time it dissipates to half of the maximum volume. The half-life time is an important parameter governing the distribution of the foam into the mixture.

If the half-life is short, the bitumen will revert to its high viscosity state after a short time, not allowing for good coating of aggregates and causing lumps of bitumen to form, either with or without mineral filler.

A long half-life time provides good workability. Although simple in concept, at present a standard method for measuring either parameter is not available. Also the values of both the expansion ratio and half-life are likely to depend on the geometry of the vessel in which they are measured because of interaction of the foam with the walls of the vessel.

For a given bitumen and foaming temperature the expansion ratio and the half-life are inversely related as the proportion of foaming water changes (Figure 4.1). The higher the water content, the greater the expansion ratio but the shorter the half-life. The foaming water content needs to be optimised to achieve the maximum possible expansion ratio and half-life to provide a workable mix.

Ruckel et al. (1983) describes an experimental method to determine the optimum foaming water content. Six samples of foamed bitumen are prepared using water content ranging from 0.5 to 3.0%. For each sample, the foamed bitumen is allowed to discharge for five seconds into a 20 litre steel drum. The maximum volume of the foamed bitumen is marked on the side of the drum. A stopwatch is used to record the time it takes the foamed bitumen to reduce to half its maximum volume, which is the half-life of the foam. The volume of the foamed bitumen is determined after total collapse (after a period of at least 60 seconds), and the expansion ratio is determined as the ratio between the maximum volume of the foamed bitumen and the final volume. The data for expansion ratio and half-life are plotted as shown in Figure 4.1. Note that the point of intersection between the two curves does not define the optimum foaming water content because the half-life and expansion ratio are two different quantities, and depending on the scale of the graph the point of intersection will change. South African investigators recommend a minimum expansion ratio of 10 and a minimum half-life time of 12 seconds to ensure that adequate coating of aggregate can occur (CSIR 1998). For practical applications, Australian researchers suggest a minimum expansion ratio of 10 but a half-life of at least 30 seconds (Ramanujam & Jones 2000).

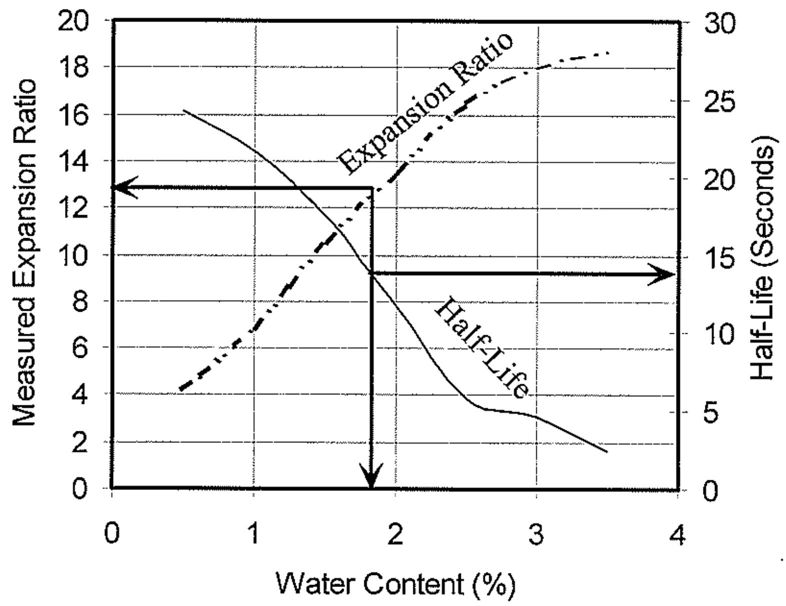

Figure 4.1 Relationship between expansion ratio and half-life versus % water content in the foamed bitumen.

Jenkins et al. (1999) argued that characterisation of foamed bitumen by determining the expansion ratio and half-life time of the foam does not provide a complete understanding of the foaming potential and the characteristics of the foamed bitumen. They performed an extensive study of bitumen foam characterisation, and found that a problem with the normal method for measuring half-life time and expansion ratio in the laboratory is that the process of spraying foam into the receiving vessel takes a considerable time (≈5 seconds) compared to the half-life time itself. As a result a significant amount of foam collapse has already occurred by the time the maximum expansion ratio is recorded. The error is greatest for foams with the shortest half-lives. Jenkins et al. (1999) found that the collapse of the foam could be modelled by an exponential decay curve:

$$ER(t) = ER_m * e^{\frac{-\ln 2}{\tau_{1/2}} * t_s} \qquad (1)$$

where:

$ER(t)$	=	Expansion ratio with respect to time after foam discharge
ER_m	=	Maximum measured expansion ratio (immediately after discharge)
$\tau_{1/2}$	=	Half-life time in seconds
t_s	=	Time measured from the moment all foam is discharged in seconds

Knowing the spray time, the above relationship can be used to back-calculate the theoretical actual expansion ratio (in effect, assuming no decay had taken place during spraying). For a foam spray time of 5 seconds, the difference between measured and actual expansion ratio (ER_a) is about 6% for a half-life of 30 seconds, but is in the order of 130% for a half-life of 2 seconds. The same researchers also measured the viscosity of the foam with a rotational viscometer. The viscosity was independent of bitumen grade (comparing 80/100 and 150/200 penetration bitumens) but increased as the expansion ratio decreased. The rate of change in viscosity was slow for a decrease in ER from about 15 to 4, but rapid thereafter (the temperature of the foam is not reported). The authors concluded that, for adequate mixing with aggregate, the expansion ratio of the foam should be at least 4 (viscosity of about 0.3 Pa.s).

A 'Foam Index' (FI) is proposed based on the above information to characterise the 'foamability' of bitumen as shown in Equation 2.

$$FI = -\frac{\tau_{1/2}}{\ln 2} * \left(4 - ER_m - 4 * \ln\left(\frac{4}{ER_m}\right)\right) + \left(\frac{1+c}{2*c}\right) * ER_m * t_s \qquad (2)$$

where:

c	=	ER_m/ER_a
t_s	=	Time of spraying to discharge all foam (seconds)
$\tau_{1/2}$	=	Half-life (seconds)
ER_a	=	Actual Expansion Ratio

The value of ER_a is calculated and the index is then defined as the area under the decay curve from an ER value of 4 to an ER value of ER_a as shown in Figure 4.2. The figure shows two areas, A1 and A2, under the decay curve bounded by the dotted lines. These two areas are summed to yield the Foam Index (FI).

$$FI = A1 + A2$$

Equation 1 is seen as a reasonable fit for the experimental foam decay data of unmodified penetration grade bitumen.

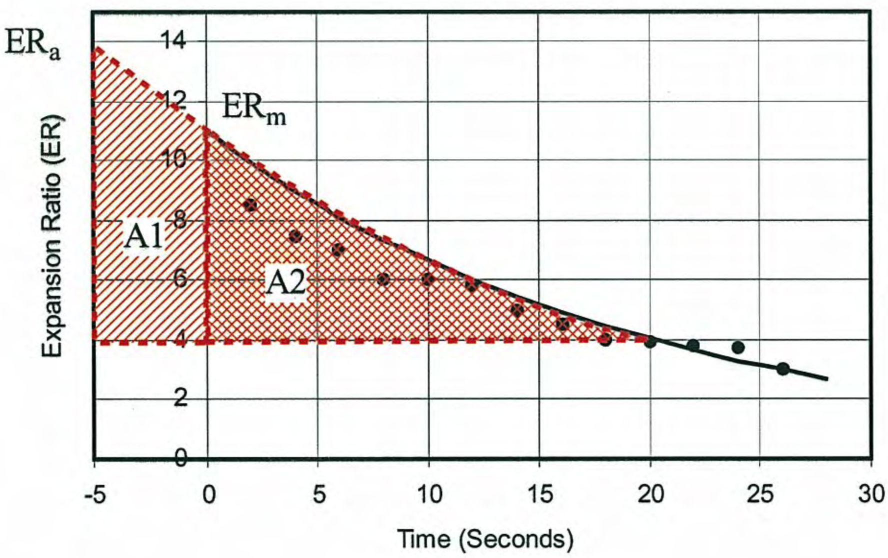

Figure 4.2 Foam Index (FI) is calculated as the area (A1 + A2) under the decay curve (Jenkins et al. 1999).

In contrast to the foam viscosity results of Jenkins et al. (1999) discussed above, Barinov (1990) found that the ultimate shear stress (failure stress) of bitumen foam increased as ER increased. This was explained as being related to an increase in strength of the bitumen films as they became thinner with an increasing degree of expansion of the foam.

5. Design of Foamed Bitumen Mixtures

5.1 Bitumen Foaming Potential

According to Castedo-Franco & Wood (1983), any grade of bitumen can be foamed by using an appropriate combination of nozzle type, water, air, and bitumen injection pressure. However, the foaming potential of bitumens differs according to their source and production method. Bitumens produced by a propane or butane precipitation process often have silicone anti-foaming agents added, the action of which needs to be nullified by the addition of foaming agents (typically <1%) such as various metal stearates (Ramanujam & Jones 2000, Castedo-Franco & Wood 1983).

Little information on the relationship between bitumen chemical composition and foaming potential is reported in the literature. Jenkins et al. (1999) reported that no correlation exists between results of saturates, aromatics, resins and asphaltenes (SARA) fractionation and foaming potential, although details of the analysis were not given. Barinov (1990), however, found that the foam expansion ratio increased with asphaltene content up to a maximum of 41.7%. The results were explained in terms of the presence of surface-active compounds in the asphaltenes. These compounds act to lower the surface tension of the bitumen but, as the concentration increases, a point is reached where the surface is saturated and no further reduction is achieved.

The effect of bitumen viscosity or penetration on foaming potential is not entirely clear. Abel (1978) reported that bitumen of low viscosity produced higher expansion ratios and longer half-lives than bitumen of high viscosity, but the use of high viscosity bitumen resulted in superior aggregate coating. He also found that the presence of anti-stripping agents (surface-active agents) intensified the foaming ability of the bitumen and that acceptable foaming was only achieved at temperatures above 149°C.

Jenkins et al. (1999) suggested that for two bitumens of different viscosities (but from the same refining process) at a given temperature, the softer material should have marginally lower surface energy and should thus foam more easily.

Bissada (1987) measured expansion ratios and half-lives for three bitumens of differing grades (but the source of the bitumens was not given). Results obtained by Bissada are summarised in Table 5.1 and indicate that, as bitumen viscosity decreases, both expansion ratio and half-life time increase.

Table 5.1 Foam characteristics for different penetration grades (Bissada 1987).

Bitumen Penetration at 25°C (0.1mm)	Bitumen Viscosity at 165°C (mPa.s)	Expansion Ratio[1]	Half-life (seconds)
67	120	9	9
135	80	11	18
310	50	13	22

[1] Foamed at 165°C, 2% water.

However, in contrast to the above, a dissimilar trend was observed by Brennan et al. (1981). In a study of eight bitumens ranging in penetration from 42 to over 200, no relationship was found between expansion ratio or half-life time and viscosity (at the foaming temperature of 163°C).

5.2 Grade of Bitumen

Most studies on foamed bitumen mixtures appear to give little attention to the grade of bitumen used. Usually only a single grade of bitumen is used (that typically used for asphalt manufacture in the particular locality), and in some cases the properties of the bitumen used are not even specified (Bowering 1970a, Leek 2001).

Maccarrone et al. (1993) reported no difference between the moduli of mixes prepared using Australian class C170 or C320 (typically with penetration at 25°C of about 80 and 60, respectively). Similarly Lee (1980) found no difference between mixes made using a US AC-10 grade (typically of about 80 pen at 25°) and 200-300 pen bitumen. This is probably because most foamed bitumen mixes develop their shear strengths through aggregate interaction rather than binder cohesion. However, other researchers found that low penetration grades of bitumen provided a high stiffness mix with a better coating of large aggregates than high penetration grades.

Also well known is that the moduli of foamed bitumen mixtures are temperature- and loading rate-dependent, suggesting that bitumen properties have some role in overall performance. Shackel et al. (1974) compared the effect of 80-100 and 180-200 bitumens on mixture performance. For a given bitumen content and degree of water saturation, the lower pen binder resulted in a higher resilient modulus (M_r) for the mix. For example at 4.0% bitumen content, M_r (at 20°C, >80% moisture saturation) increased 40% from 93 to 132 MPa; while the same mix in a 'dry' condition showed an increase of 19% of M_r value from 185 to 220 MPa. The initial (first cycle) strain was also greater for the softer bitumen, but the total number of cycles to a given strain showed no obvious dependence on bitumen grade.

A comprehensive study of the effect of bitumen grade or the use of modified binders (e.g. with polymer) does not appear to have been conducted, but such a study would appear to be well warranted.

5.3 Foamed Bitumen Content

Unlike hot mix asphalt, foamed bitumen mix design does not show a clear optimum bitumen content. Determination of the optimum bitumen content is based on the loss of stability and moisture susceptibility. Studies showed that the ratio between foamed bitumen content and the amount of fines in the mix is an important parameter that significantly affects mix stability. Sakr & Manke (1985) showed that mixes with higher percentages of fines showed higher stabilities, while Bissada (1987) showed a similar trend for tensile strength. The mix of foamed bitumen and the fines creates a mortar-like material that binds coarse aggregates together giving a high strength material. However, excess binder will work as a lubricant and decrease stability. Therefore, the optimum binder content to fines ratio should be determined to yield the maximum strength. Table 5.2 from Ruckel et al. (1980) may be used as a guide to select the appropriate binder content based on the fines content of the mix. Table 5.3 provides an estimate to the optimum binder content based on the aggregate classification by the Unified Soil Classification System (USCS) (Bowering & Martin 1976).

Table 5.2 Estimation of foamed bitumen (%) based on the percentage of fines and sand (Ruckel et al. 1980).

% passing 4.75mm Sieve	% passing 0.075mm Sieve	% Foamed Bitumen
Less than 50% (Gravels)	3-5	3
	5-7.5	3.5
	7.5-10	4
	>10	4.5
Greater than 50% (Sand)	3-5	3.5
	5-7.5	4
	7.5-10	4.5
	>10	5

Table 5.3 Estimation of binder content (%) of foam-treated materials based on soil classification (Bowering & Martin 1976).

Soil Type	Optimum Range of Binder Content (%)	Additional Requirements
Well graded clean gravel	2.0-2.5	
Well graded marginally clayey–silty gravel	2-4.5	
Poor graded marginally clayey gravel	2.5-3.0	
Clayey gravel	4-6.0	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	Possibly lime
Clayey sand	3-4	Lime modification

5.4 Aggregate Gradation

Test results indicate that low plasticity materials with a relatively large percentage of fines (passing 75μm sieve) perform best because the foamed bitumen tends to coat the fines, and only partly coat larger particles (Abel 1978). In addition, studies have shown that a wide range of aggregate types and gradations could be used with foamed bitumen stabilisation, ranging from crushed stones to silty sands. Akeroyd & Hicks (1988) developed the Mobil Oil foam stabilisation chart (redeveloped in Figure 5.1). This chart consists of three gradation zones, where:

- Zone A is ideal for heavy trafficked roads;
- Zone B is suitable for light traffic, but could be adjusted to zone A by the addition of more coarse material;

- Zone C is unsuitable for foamed bitumen stabilisation unless adjustments to the fine fraction are made to bring it into zone A.

The fines content is very crucial in the foamed bitumen stabilisation and should ideally exceed 5% (Ruckel et al. (1983) and Akeroyd (1989) recommend between 5% and 20%). Figure 5.1 also shows the AP-40 and AP-20 gradations, which are commonly used for base materials in New Zealand. Previous studies (Ruckel et al. 1983, Akeroyd 1989) suggest that gradation AP-20 needs some adjustment to the mineral filler part to bring it to the midpoint of band A, which would then make it ideal for foamed bitumen stabilisation. AP-40 gradation lies exactly on the boundary between zones A and C and clearly needs adjustment to the fine part to bring it to zone A.

In addition to gradation of foamed bitumen mix, the type of skeletal structure in the mix needs to be determined according to the SA Interim Technical Guidelines (SA Transportek 2002) recommendation. Figure 5.2 shows the triangular distribution of coarse aggregates (materials retained on sieve 2.36mm), fine aggregates (materials passing 2.36mm and retained on 75μm sieve), and mineral filler (materials passing 75μm sieve). Any aggregate gradation curve will be represented by a single point on this triangular chart. Three zones are shown in this figure: sand skeleton[1], stone skeleton, and filler skeleton. Sand and stone mixes, which produce the densest grading, are ideally suited for treatment by foamed bitumen. Mixes that are mainly stone skeleton or filler skeleton are not suitable for foamed bitumen mixes and their gradations must be adjusted if foamed bitumen is to be used.

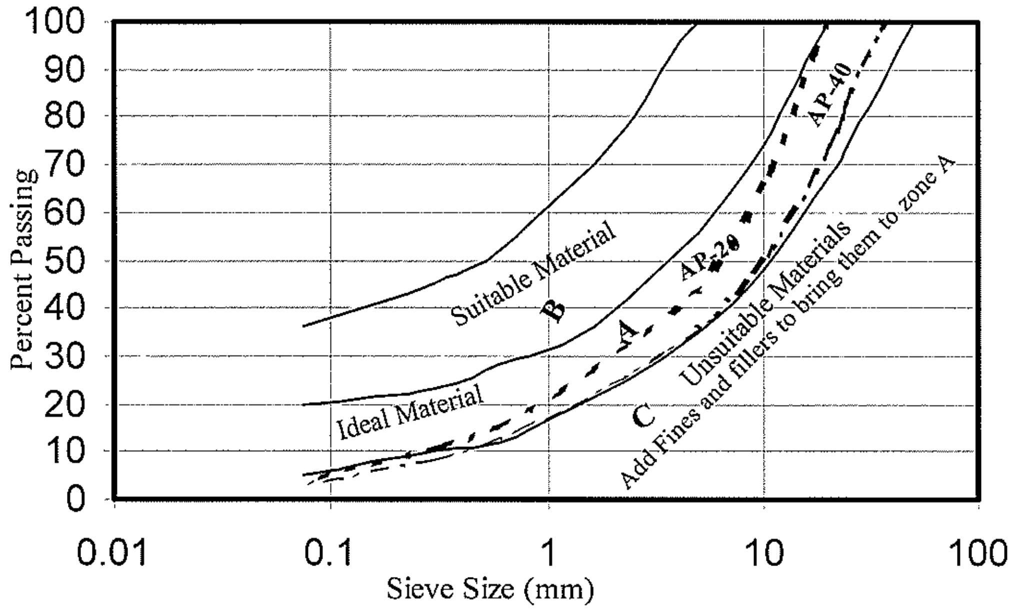

Figure 5.1 Gradation envelopes and their suitability for foamed bitumen mixes.

[1] Skeleton – term used in SA Interim Technical Guidelines to express composition of aggregate.

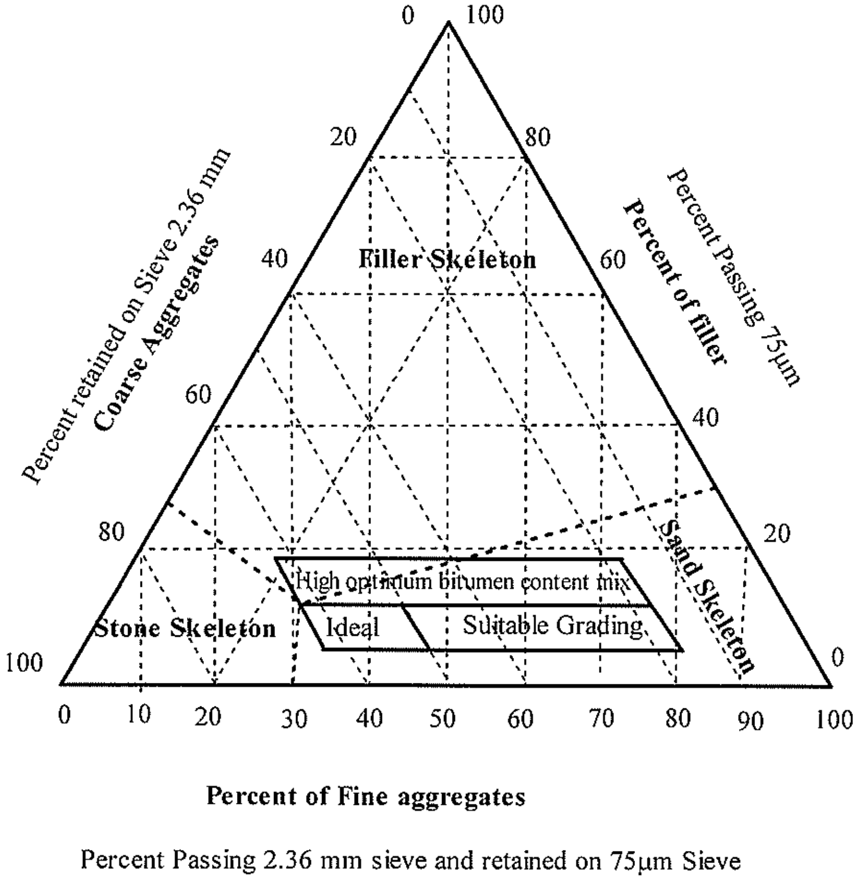

Figure 5.2 Triangular distribution of aggregates (SA Interim Technical Guidelines, from Bowering & Martin 1976).

5.5 Moisture Content at Mixing

Moisture content during mixing and compaction of foamed bitumen-treated materials is one of the most important factors in the design. Moisture content affects the dispersion of foam during mixing and affects the workability and ease of compaction. Insufficient moisture content decreases the workability and reduces the dispersion of the foam in the mix. On the other hand, too much moisture increases the curing period of these mixes and reduces their strength.

The mixing water is needed to moisten the aggregates to help break up any agglomerations. It has also been postulated (originally by Csyani 1957) that the mixing water envelops and separates the fine particles by thin layers of moisture, providing channels through which the foamed bitumen can penetrate to coat the fine particles (Sakr & Manke 1985).

Mobil Oil investigators and Bowering (1970b) suggested moisture content around the 'fluff point' of aggregates which is about 70% to 80% of the modified AASHTO optimum moisture content (OMC). The fluff point was defined as the moisture content at which a given weight of soil yields the maximum loose bulk volume consistent with easy manipulation. Bissada (1987) and Lee (1980) recommended a similar range for OMC (65% to 85 % of modified AASHTO OMC) at mixing.

The concept of total fluid content is also used in foamed bitumen mix design in which the lubricating effect of both water and binder is considered. Studies have shown that the best compaction of foamed-treated mixes occurs when the total fluid (binder and moisture) content is equal to optimum moisture content (Castedo-Franco et al. 1982).

Sakr & Manke (1985) developed a regression equation for optimum pre-moulding moisture content (MMC) that maximises the density of foamed bitumen as a function of the modified AASHTO aggregate optimum moisture content, percent of materials passing 75µm and binder content, as shown in Equation 3. In this study the researchers found no significant difference in the mix properties if both mixing and compaction moisture contents were assumed to be the same as MMC, and the properties of those mixes that use mixing-moisture content about 10% to 20% higher than MMC.

$$MMC = 8.92 + 1.48 * OMC + 0.4 * PF - 0.39 * BC \tag{3}$$

where:

MMC	=	Optimum pre-moulding moisture content
OMC	=	Aggregate modified AASHTO optimum moisture content,
PF	=	Percent of materials passing the 75 µm sieve
BC	=	Binder content

Note: the values of MMC, OMC, and BC were calculated as percentage of the dry weight of aggregates.

5.6 Type of Compaction

Nataatmadja (2002) investigated the effect of the compaction method used on the characteristics of foamed bitumen-stabilised materials. Marshall compaction was used at two compaction effort levels: 50 and 75 blows per face and gyratory compaction at 85 cycles (angle of gyration 3°). The results of this study showed that gyratory compaction provided higher bulk densities and lower resilient moduli (up to about 1500 MPa lower) than those obtained using the Marshall compactor. However, specimens compacted with the Marshall compactor clearly were also more sensitive to bitumen content variation. Compared to 50 Marshall hammer blows, the effect of the additional compactive effort provided by 75 blows was small (about 250 MPa) at lower bitumen content, but showed significant differences (about 750 MPa) at higher bitumen contents. The study also showed that gyratory compacted specimens were less moisture susceptible than Marshall specimens.

5.7 Curing Conditions

In order to assess the properties of mixtures in the lab in a consistent way and to relate laboratory-measured parameters to field performance, various accelerated curing methods have been adopted by different researchers and agencies. Field experience has indicated that the strength of foamed bitumen mixes tends to improve with age, traffic, and temperature (Clarke 1976). These conditions contribute to the removal of moisture from the compacted mix. Curing of foamed bitumen-stabilised materials is defined as the process of gaining gradual strength accompanied by continuous reduction of moisture content by evaporation.

Although foamed bitumen mixes do not have the long curing times associated with cutback or emulsion mixes, the curing condition must be considered in the mix design and evaluation (Csanyi 1957, Nady & Csanyi 1958). Acott (1980) found that the time required for the mix to develop strength in the field, equal to that after long-term laboratory curing, ranged from 23 to 200 days. This varies according to road temperature, precipitation, and evaporation. The choice of whether the specimens are cured in or out of the mould affects the condition of the specimen.

Because cold mixes can initially be quite fragile, a procedure was developed at Purdue University (USA) in which specimens were cured initially in the mould followed by an out-of-mould curing period (Brennen et al. 1981, Castedo-Franco & Wood 1983). The most commonly used method is that of Bowering (1970b) which involves oven treatment for 72 days at 60°C. This was found to result in 0–4% moisture content, representative of the driest conditions likely to happen in service. This curing treatment has been found to give resilient moduli and creep properties similar to cores from 12 month-old field sites (Maccarrone et al. 1993). Some researchers criticise the high temperature of this curing technique because of the possibility of binder oxidation and because binder dispersion is likely to occur. T. Lewis (pers.comm. 1998[2]) suggested an alternative method using a temperature of 40°C until the specimen is completely dry (i.e. dried until constant weight).

Ruckel et al. (1983) suggested a different curing procedure to simulate the short-, intermediate-, and long-term conditions of the specimens in the field, as follows:

- To simulate the condition in the field after 24 hours from construction, laboratory specimens were left in their moulds on their sides for 24 hours at room temperature.
- To simulate intermediate curing periods of 7 to 14 days after field laydown under dry conditions, specimens were left on their sides in the moulds for 24 hours then removed from the moulds and transferred to an oven at 40°C for another 24 hours.
- For long-term curing (modelling 30 days of field curing), specimens were removed from the moulds and placed into an oven adjusted at 40°C for 72 hours.

Nataatmadja (2002) investigated the effect of curing method on the resilient modulus for dry and soaked specimens. Oven-cured specimens at 60°C for 72 hours provided the highest resilient modulus about three times that of specimens cured at 40°C for 3 days. The 7-day air-cured specimens at ambient temperature (25°C) provided resilient moduli similar to those cured at 40°C for 3 days. The resilient modulus of the soaked specimens cured at 60°C for 3 days was about 6 times higher than that of specimens cured at 40°C for 3 days. The resilient modulus of the soaked air-dried specimen was the lowest compared to the resilient moduli of samples oven dried at 60°C and 40°C.

Bissada (1987) studied the effect of three different curing conditions: in laboratory air at 23°C, in a humidity chamber (23°C and 100% humidity), and in an oven at 40°C. As curing time increased, moisture content of the samples decreased from the original value. Oven curing at 40°C for three days resulted in about 1% reduction of the moisture content and was equivalent to 14 days at 23°C. Marshall stability values

[2] T. Lewis, AA Louden & Partners Consulting Engineers, Pretoria. Also quoted in Muthen 1999.

improved as moisture content decreased although no consistent relationship was observed. The author concluded that factors other than moisture loss (such as binder aging) might contribute to the observed increase in strength.

Since specimen moisture content is a primary parameter affecting the mix strength; a wide range of curing procedures have been used by various researchers to reduce the moisture level of the specimen. Any procedure that produces compatible and repeatable results can be adopted as long as it does not alter the basic nature of the mix in comparison with changes occurring under field conditions.

Oxidative hardening of the bitumen is likely to be a minor factor in the field because of low subsurface temperatures (in New Zealand) and the limited access of oxygen. Laboratory-curing conditions are likely to over-estimate this effect because of the high surface area to volume ratio of laboratory specimens. However, probably of greater significance (especially using high curing temperatures) are errors resulting from bitumen flow or redistribution of the binder in the mix.

The relationship between the specimen moisture content and strength can be related to the mix condition in the field and can be used to define when the road can safely be opened to traffic. Thus, to address New Zealand conditions, extensive work must be done to relate field and laboratory conditions.

5.8 Mixing and Compaction Temperature Conditions

Bowering & Martin (1976) referred to a 'critical temperature' range between 13°C and 23°C for the minimum aggregate temperature, below which foamed bitumen mixes of poor quality are obtained. However, the authors did not investigate the effect of using higher temperatures on the quality of foamed bitumen mixes.

Warm aggregates should increase the dispersion of foam and provide better coating of larger aggregates. Jenkins et al. (1999) studied the effect of moderate heating of aggregates above ambient temperature before mixing with foamed bitumen. Among the factors considered in the study were aggregate type and gradation, aggregate temperature at mixing (30°-95°C), and compaction temperature (20°-70°C), bitumen grade, foam characteristics, and moisture content of the mix.

The study concluded that the temperature gradient between the aggregate and the foamed bitumen influences the rate of collapse of the foam. If the aggregate temperature is below 30°C and the temperature of the foam is about 100°C, then the temperature of the mix will drop to about 38°C. Therefore, the collapse of the foamed bitumen and consequently an increase in the binder viscosity will take place rapidly.

On the other hand, using aggregates at about 100°C results in an equilibrium mix temperature of about 95°C. In this case, the foam will last for a longer period of time allowing for better coating of large aggregates and dispersion of the bitumen in the mix.

The vane shear test was used to measure the shear strength at 50°C for different aggregate mixing temperatures (Jenkins et al. 1999). The shear strength was improved significantly at high aggregate mixing temperatures (about 95°C). This was interpreted as being related to the improved continuity of the binder in the mix. The indirect tensile strength was measured at different mixing and compacting temperatures. The results showed a general increasing trend of the indirect tensile strength with the increase of mixing and compacting temperatures. The investigators in this study did not show the effect of mixing temperature on rutting resistance or low temperature cracking.

As discussed in Section 5.5 of this report, moisture content plays a major role in the quality of the foamed bitumen mix. If hot aggregates are used, part of the aggregate moisture will be lost and should be compensated for. Jenkins et al. (1999) established a regression equation to estimate the necessary correction. The equation is given as:

$$MC_f = 0.640 * MC_i - 0.0232 * T_a - 0.093 * BC + 2.978 \tag{4}$$

where:

MC_f = Final moisture content immediately after mixing as a percentage
MC_i = Initial moisture content immediately before mixing as a percentage
T_a = Aggregate temperature in °C
BC = Binder content of foamed bitumen (by weight of dry aggregate)

The above equation is valid only if the bitumen temperature is below 190°C, while the aggregate temperature is in the range between 45°C to 98°C and the mixing time does not exceed 20 seconds. Although the correlation coefficient of this equation is somewhat low (R^2 = 0.6), it provides useful estimation of the moisture loss that needs to be compensated for when using the 'warm aggregate' technique.

6. Engineering Properties of Foamed Bitumen Mixes

6.1 Moisture Susceptibility

Because of the high air voids and low binder contents in foamed bitumen mixes compared to hot mix asphalt (HMA), their strength properties are moisture-dependent. For example, Nataatmadja (2002) noted a reduction in M_r of about 2500 MPa (32%) for gyratory-compacted specimens at 2.5% bitumen content when soaked for about 10 minutes under vacuum, while equivalent Marshall-compacted specimens showed reductions in M_r of about 35-40%.

Maccarrone et al. (1993) reported comparative M_r results for dry and soaked mixes. Dry specimens were cured for 72 hours at 60°C. Soaked specimens were also soaked in a water bath adjusted to 60°C, for 24 hours before testing. Results were compared to field cores recovered about 12 months after construction. Reductions in strength of 15 to 25% were observed (Table 6.1).

Table 6.1 Comparison of resilient modulus values (MPa) of laboratory versus field specimens (Maccarrone et al. 1993).

Project	Laboratory-prepared specimens		Field cores
	Dry	Soaked	Dry
1	4060	NA	5100
2	4700	3500	3630
3	4600	3900	5000
4	4000	NA	3930

Studies have shown that the addition of Portland cement or lime tends to decrease moisture susceptibility (Castedo-Franco & Wood 1983, Bissada 1987). Asi et al. (2002) found that the addition of 2% Portland cement increased the soaked tensile strength (at 7% bitumen) of the mix from about 110 kPa to 125 kPa (12% increase). The retained Marshall stability of such mixes after soaking was about 60%. Similarly, Theriault (1998) found that the retained Marshall stability after soaking of foamed bitumen samples with 2% added lime, increased from 62% to 85%. Higher bitumen contents decrease air voids, resulting in better coating of larger aggregates, and consequently reducing moisture susceptibility.

6.2 Temperature Susceptibility

Various authors have studied the effect of temperature on the resilient moduli of foamed bitumen mixtures (Little et al. 1983, Castedo-Franco & Wood 1983, Ruckel et al. 1980, van Wijk & Wood 1983). They found that strengths of the mixtures decrease with increasing temperature. Nataatmadja (2002) reported decreases in M_r of approximately 40% between 10°C to 40°C for mixes with binder contents ranging from 1.5% to 4.2%.

In general, foamed asphalt mixes appear less temperature-sensitive than hot mix asphalt mixes. Bissada (1987) for example, found that above 35°C, foamed bitumen mixtures had resilient moduli above those of the equivalent hot mix asphalt. Muthen (1999) attributes this to maintenance of the friction between the larger particles because they are not coated in binder, even though the bitumen–fines mortar in a foamed bitumen mixture will soften with increasing temperature. This is not like a conventional hot mix. Button et al. (1980) also found that temperature susceptibility of foamed bitumen mixes was much less than that of hot mix asphalt. They interpreted this as being related to the discontinuous nature of the foamed bitumen-binding mechanism where the binder forms a discontinuous random matrix of primarily fines and bitumen.

6.3 Marshall and Hveem Stabilities

Marshall and Hveem stabilities have often been used to characterise foamed bitumen mixes. Ruckel et al. (1980) used foamed bitumen to improve mechanical properties of mixes using marginal aggregates, and monitored performance using Marshall and Hveem stabilities. In the study four types of aggregates were used:

- one-sized wind-blown sand,
- well-graded field sand,
- fairly well-graded 'off-beach' sand, and
- densely graded siliceous river gravel.

For both Marshall and Hveem stabilities, the prepared specimens were tested at 22.8°C instead of the standard 60°C to avoid any premature failure during handling or testing specimens. For well-graded sands, Hveem stability decreased with the increasing binder content, while the opposite occurred for the one-sized blow sand. On the other hand, Marshall stability showed an increasing trend with increasing binder content for all types of sands. The well-graded sands showed higher stabilities than the one-sized blown sand.

Other researchers, however, have found Marshall stabilities to decrease with increasing bitumen content (Bissada 1987, Sakr & Manke 1985, Asi et al. 2002). Marshall stability results presented by Lee (1981) increase to a maximum before decreasing at higher binder contents. The relationship between Marshall stability values and bitumen content is also dependent on aggregate particle shape and fines content (Sakr & Manke 1985).

6.4 Unconfined Compressive Strength (UCS)

The unconfined compressive strength (UCS) test has been used as an indirect measure of the shear strength of the stabilised mix, and more specifically of its cohesion. Bowering & Martin (1976) observed that, in practice, the UCS of foamed bitumen-stabilised aggregates is approximately 1.8 to 5.4 MPa.

6.5 Resilient Modulus

As foamed bitumen mixes contain viscoelastic bitumen, the mix strength properties are dependent not only on moisture, temperature, bitumen content and aggregate gradation, but also on loading rate. Maccarrone et al. (1993) found a 50% reduction in M_r when the pulse loading time was increased from 30 to 120 msec. The M_r values of foamed bitumen-stabilised materials are significantly greater than those of untreated materials (e.g. Shackel et al. 1974) and can be comparable or better than those of equivalent hot mixes especially at high temperatures. In some cases, foamed bitumen-stabilised materials can possess stiffness comparable to those treated with Portland cement but with the added advantage of flexibility and fatigue resistance (Ramanujam & Fernando 1997).

The resilient moduli of foamed bitumen mixes are discussed further in Chapter 10 of this report.

6.6 Abrasion Resistance

Muthen (1999) states that foamed bitumen-stabilised materials (using virgin aggregates) lack abrasion resistance and are not suitable for use as a wearing course. Actual test results, however, have not been reported in the literature. Abrasion resistance of mixes is usually measured using the Los Angeles abrasion test machine modified so that Marshall test sized specimens can be used.

6.7 Fatigue Resistance and Deformation Resistance

At high service temperatures, Bissada (1987) considered that permanent deformation of foamed bitumen mixes was the key criterion of their performance (as opposed to fatigue cracking). The author concluded that foamed bitumen mix would have a reduction in rutting of 35-50% compared to the equivalent HMA.

Ramanujam & Jones (2000) performed rutting tests on foamed bitumen mixes using a wheel-tracking machine. The tests were performed immediately, and 24 hours, after compaction to examine the effect of early trafficking on the stabilised road performance. The test results indicated that the immediate rut resistance of the stabilised material is relatively low. However, after 24 hours curing (at room temperature) the stabilised material showed only minimal rutting, equivalent to high strength HMA at 60°C. The implication drawn by the researchers was that, depending on the moisture content of the mix, traffic should be restricted within the first 24 hours after construction to limit the occurrence of rutting.

Fatigue resistance is an important factor in determining the structural capacity of pavement layers. Little et al. (1983) found the fatigue resistance of foamed bitumen mix to be well below that of HMA and emulsion mixes. These results are contradicted somewhat by the work of Ramanujam & Jones (2000) who performed beam fatigue tests on foamed bitumen mixes with 3.5% bitumen, 2.0% lime, and 70% of the optimum moisture content of the untreated aggregate. Compacted slabs were oven-cured for 72 hours at 60°C, and cut down into beams of suitable dimensions using a circular saw. The test results indicated that the fatigue potential of foamed bitumen mixes appears somewhat greater than that of asphalt mixes. However, the fatigue life appeared to be relatively short at high strain levels (450με). This indicates a low fatigue life if these mixes were to be used on low strength subgrades where excessive deflection would result in high strain levels.

7. Classification of Foamed Bitumen Mixes

Mechanical properties, such as UCS (described in Chapter 6 of this report), have been used to classify foamed bitumen mixes.

The SA Interim Technical Guidelines (SA Transportek 2002) present criteria based on the indirect tensile strength (ITS) and unconfined compressive strength (UCS) test values to classify foamed bitumen-stabilised mix into four grades (FB1 to FB4) as shown in Table 7.1. This approach utilises both the ITS and UCS to capture the complex behavioural characteristics of foamed bitumen mixes where shear strength and flexural characteristics interact in the material classification process.

Table 7.1 Classification system for foamed bitumen as presented in SA Interim Technical Guidelines (SA Transportek 2002).

Material Code		**ITS at 25°C (kPa)**	
		100 -300	300-500
UCS at 25°C (kPa)	700-1400	FB4	FB3
	1400-2000	FB2	FB1

Design charts have been developed for FB2, FB3.
Design charts have not been developed for FB1, FB4, as at 2002.

8. Laboratory Design Method for Foamed Bitumen Mixes

Foamed bitumen mixes have no standard laboratory design method equivalent to the Marshall, Hveem or Superpave design procedures that are used for HMA. However, many researchers have adopted an almost similar laboratory design method for foamed bitumen mixes. The differences are found mainly in the suggested curing method or in the procedure for the determination of the optimum bitumen content. Some researchers considered the optimum binder content as that at the maximum retained Marshall stability, while others have maximised the resilient modulus. In the following sections, a summary of the laboratory design method suggested by Ruckel et al. (1983) is presented. The procedure is very similar to that of the laboratory design method given in the SA Interim Technical Guidelines (SA Transportek 2002).

8.1 Binder Characterisation

Foamed bitumen is characterised by its half-life time and its expansion ratio. The recommended minimum allowable expansion ratio is 10-15 mL/g and the minimum half-life time is 12–20 seconds. Foaming agents can be used to improve the foaming characteristics of bitumen, but the cost impacts should be considered. The standard procedure suggested by Ruckel et al. (1983) to quantify these two parameters is as follows:

1. The sample containers, of sufficient volume (in millilitres) for the desired foam characterisation tests, is determined based on the container dimensions or by weight of water (assume a water density of 1 g/cm^3) to fill the container to the brim.
2. The foamed bitumen apparatus should be adjusted to control the rate of water and bitumen. The bitumen temperature should be adjusted at 180-190°C.
3. Five batches of foamed bitumen should be prepared; each batch has a different percentage of water: the first batch has 1.0% water, and water is increased in increments of 0.5% (to 3% in batch 5).
4. For each batch, the half-life is determined using a stopwatch. The half-life is the time the foam takes to collapse to half of its maximum value. The expansion ratio is determined as follows:

$$Expansion \quad Ratio = \frac{A}{B}$$

where:

A is the maximum volume of the foam (in millilitres) calculated from the average maximum height level attained by the foam, as shown by the film left inside the container and the height and capacity of the container; and

B is the mass of bitumen (in grams), including the foamant.

Note that the numerical value of the mass of bitumen in grams is the same as the volume in cm^3, assuming a bitumen density of 1 gm/cm^3.

5. A chart similar to Figure 4.1 should be constructed to demonstrate the relationship between water content and both the half-life and the expansion ratio. The amount of water that optimises both of these parameters is determined and the corresponding expansion ratio and half-life are reported.

6. If the foam parameters do not meet the specified values, either another bitumen grade or type should be tried, and/or a foaming agent might be used at different dosages to improve the foamability of the bitumen. Table 8.1 shows different characteristics of foamed bitumen and its suitability based on aggregate temperature during mixing. These values are recommended in the SA Interim Technical Guidelines (SA Transportek 2002) and the Foam Index (FI) as discussed in Chapter 4 of this report.

Table 8.1 Classification of the suitability of foamed bitumen for cold mixes (SA Transportek 2002).

Foam Index (seconds)	Aggregates at 15 °C	Aggregates at 25°C
<75	Unsuitable	Unsuitable
75-100	Very Poor	Poor
100-125	Poor	Moderate
125-175	Moderate	Good
175-200	Good	Very Good
>200	Very Good	Very Good

8.2 Aggregate Characterisation

Aggregate properties are the determining factor in many of the choices made concerning the optimum mix and, therefore, a thorough testing of the aggregates is important. A wide range of materials is suitable for use with foamed bitumen, including crushed stone, rock, gravel, sand, silty sand, sandy gravel, slag, recycled aggregate, ore tailings, and other synthetic materials. At least 50 kg of aggregates is required for a complete mix design. The aggregate gradation should be determined and the Unified Soil Classification System (USCS) should be used to classify the aggregate. The suitability of aggregates for foamed bitumen mixes is determined from Table 5.3. The aggregate gradation should satisfy the grading recommendations given in Figure 5.1. If necessary, the gradation envelope must be adjusted to contain the required amounts of coarse and fine aggregate particles.

Note that the gradations provided in Figure 4.1 are broad and can be refined by targeting a grading that provides the lowest percentage of voids in the mineral aggregates (VMA). Jenkins (1999) has shown that the most desirable properties of foamed mix can be obtained by minimising the voids in the mineral aggregates. Cooper et al. (1991) developed a relationship to achieve the minimum VMA for certain filler content and maximum aggregate size. The Cooper relationship provides flexibility in selecting the percentage of fines, and is given by Equation 5.

$$P = \frac{(100 - F) * (d'' - 0.075'')}{(D'' - 0.075'')} + F \qquad (5)$$

where:

d = Selected sieve size (mm)
P = Percentage by mass passing a sieve of size d
D = Maximum aggregate size (mm)
F = Percentage filler content (inert or active)
n = Variable depends on aggregate packing

A value of n = 0.45 is used to achieve the minimum VMA.

Another important consideration is the plasticity index (PI) of the fine particles that should be determined. If the PI exceeds 12%, lime or Portland cement may be used to reduce the material plasticity.

8.3 Moisture Content

Moisture content, as previously stated (Section 5.5 of this report), is a crucial parameter in the mixing and compaction process. The moisture content is selected as the greater of modified AASHTO OMC minus binder content and the moisture content at the fluff point. Ruckel et al. (1983) suggested determining the OMC in the laboratory by establishing the moisture density relationship using aggregates with different moisture levels and a fixed foamed bitumen content. The moisture contents of the batches are adjusted so that one batch has about 50%, one has about 60%, and one has about 80% of the AASHTO OMC. The moisture content that yields the maximum dry density of the compacted specimens is the OMC of the aggregate.

8.4 Binder Content

An initial estimate of the optimum binder content can be determined from Table 5.3 based on the aggregate gradation. Five binder contents should be tried with increments of 1.0% so that two increments are above the estimated optimum and two are below it. Five batches of mix, 10 kg each, are required for a complete mix design.

8.5 Mixing and Compaction

Each batch of the five combinations should be mixed with the foamed bitumen at the pre-determined level using a mechanical mixer. Eight specimens of Marshall size should be prepared from each batch. The weight of each specimen is around 1150 g to 1200 g. The compaction effort applied on each specimen depends on the expected traffic loads and volumes. For example, for heavy traffic volumes, apply 75 Marshall blows per face, or 120 gyrations at 240 kPa if a gyratory compactor is to be used.

8.6 Optimum Binder Content

A suitable curing method should be adopted. Ruckel et al. (1983) suggested three different curing methods, each representing a certain stage occurring in the field. If knowledge of the early strength is required, then short-term curing should be considered. However, if intermediate or long-term status are of concern, then the specimens should be cured using either intermediate or long-term methods, respectively, as described in Section 5.7 of this report.

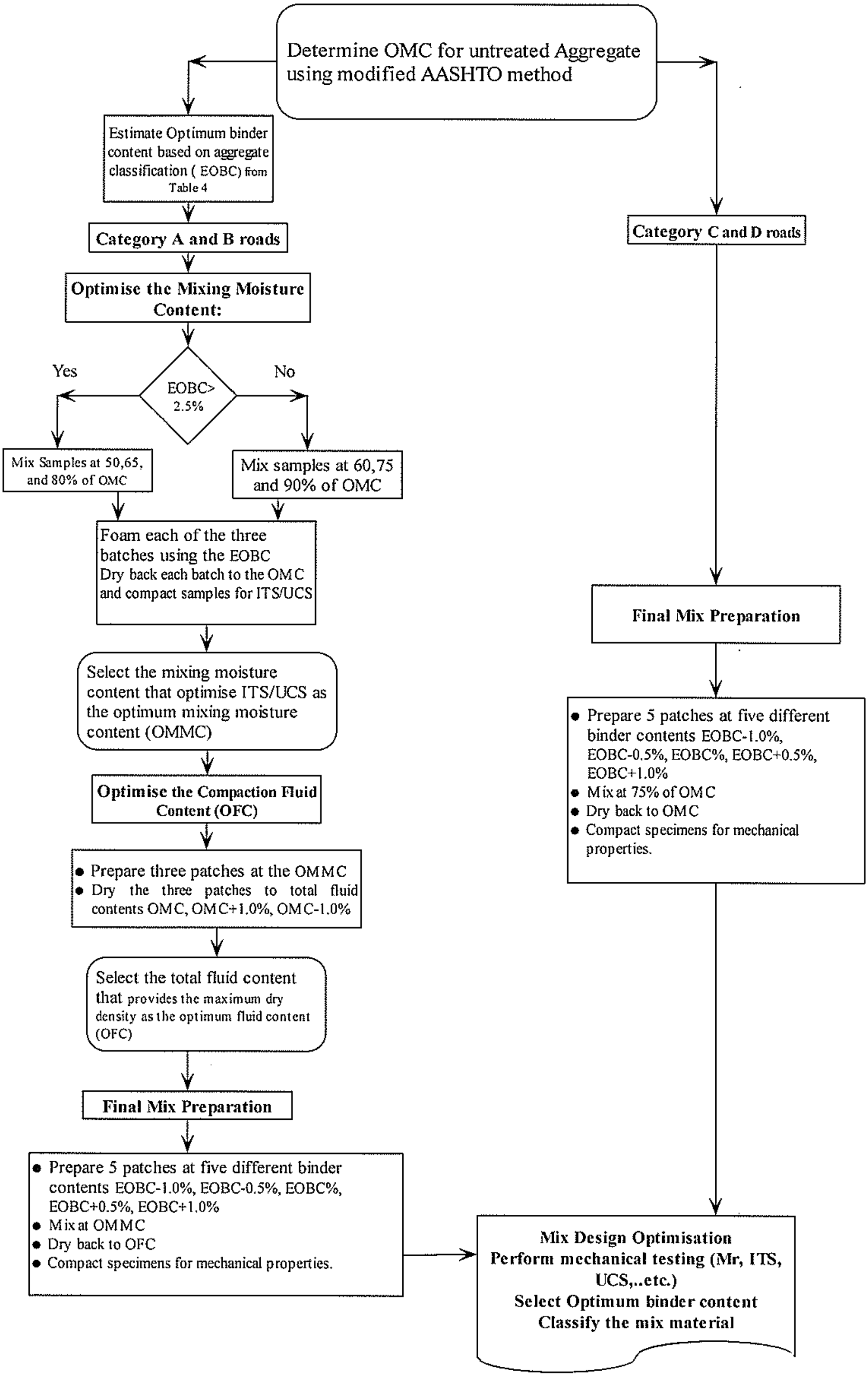

Figure 8.1 Flow chart explaining the detailed steps of the South African foamed bitumen mix design (SA Transportek 2002).

After curing, the resilient modulus and indirect tensile strength of dry and soaked specimens should be determined. Ruckel et al. (1983) suggest soaking at room temperature and at 100 mm Hg for one hour followed by another hour soaking at atmospheric pressure. The SA Interim Technical Guidelines (2002) recommend a less severe 24-hour soak at 25°C (and atmospheric pressure). Flexural tests on cured beams may also be done to determine the stiffness modulus and the strain levels at failure.

The optimum bitumen content is determined so that the retained indirect tensile strength and the retained resilient modulus are maximised. The retained indirect tensile strength and the retained resilient modulus are defined as:

$$\text{Retained ITS} = \frac{\text{ITS}_{(\text{after soaking})}}{\text{ITS}_{(\text{Dry})}} * 100 \tag{6}$$

$$\text{Retained } M_r = \frac{M_{r(\text{after soaking})}}{M_{r(\text{Dry})}} * 100 \tag{7}$$

Figure 8.1 shows a flowchart for the SA design method for foamed bitumen-treated mixes as described in the SA Interim Technical Guidelines (2002). As the flowchart clearly shows, two suggested design methods are presented, based on the highway hierarchy system. Four different categories of highways were considered:

- Category A represents major interurban freeways and roads.
- Category B represents interurban collectors and major roads.
- Category C represents lightly trafficked rural roads.
- Category D represents light pavement structures, rural access roads.

For Category A and B roads, a more comprehensive mix design is recommended as shown in the flowchart. In this method, three quantities need to be optimised:

- water content at mixing; water content at compaction, and
- binder content.

For Category C and D roads, a less comprehensive mix design is devised. In this method, the optimum water content for both mixing and compaction is assumed to be the same and is considered to be 75% of AASHTO optimum water content.

9. Comparison of Foamed Bitumen with Other Stabilisation Agents

Table 9.1 compares the characteristics of a well-graded crushed aggregate treated with five different stabilising agents (Lewis & Collings 1999). Except in cement-stabilised specimens, these properties are based on Marshall specimens compacted with 75 blows per face and cured for 24 hours at ambient temperature, then for 72 hours at 60°C. Table 9.1 shows clearly that foamed bitumen provides comparable or better results than other stabilising agents, in particular when a small percentage of cement is added to the mix.

Table 9.1 Comparison between mechanical properties for different stabilising agents (Lewis & Collings 1999).

Test Parameter	Cement (3.0%)	Bitumen Emulsion (3.0%)	Bitumen Emulsion (3.0%) + Cement (2.0%)	Foamed Bitumen (3.5%)	Foamed Bitumen (3.5%) + Cement (1.0%)
UCS (MPa)	3.0	nd	nd	nd	nd
ITS-Dry (kPa)	250	200	500	200	500
ITS-Soaked (kPa)	nd	80	250	80	300
Marshall Stability-Dry (kN)	nd	10	20	10	20
Resilient Modulus-Dry (MPa)	5000	2000	3500	2000	3500

UCS = Unconfined Compressive Strength, ITS = Indirect Tensile Strength, nd = not determined.

Roberts et al. (1984) carried out a laboratory investigation to study the feasibility of using foamed bitumen to recycle salvaged asphalt mix and to stabilise clean sand. A comparison of foamed bitumen, cutback bitumen, and two types of bitumen emulsion was made. Specimens were prepared, cured and tested under both dry and soaked conditions. The Hveem stabilities at 60°C of dry foamed bitumen mixes (cured for 4 days at 60°C and 3 days at 24°C) were comparable to those of hot mix. Hveem stabilities of the foam-treated materials were superior to those of cutback- and emulsion-treated cold mixes, whereas the tensile strengths of the foam-treated materials were about 40% higher than those of the emulsion and cutback specimens. The retained strength after water soaking was about 50% of the dry strength (see Section 6.1 of this report), but the emulsion and cutback specimens showed comparable reductions. Asi et al. (2002) also found foamed bitumen mixes to be superior to emulsion mix based on a statistical analysis of soaked Marshall stabilities.

10. Constitutive Models for Foamed Bitumen Mixes

Despite the widespread use of foamed bitumen stabilisation, there is a lack of performance criteria to assist in predicting the structural capacity and performance of pavement layers constructed with these materials. By changing the mix proportions of the aggregate, bitumen and active filler, a mix can be created that has behaviour resembling the behaviour of unbound materials, or cemented materials, or HMA.

10.1 Granular-type Foamed Bitumen Mixes

According to the SA Interim Technical Guidelines (SA Transportek 2002), granular-type foamed mixes are produced by the addition of a small amount of bitumen (<2%) to an aggregate with less than 2% Portland cement or without cement at all.

Jenkins et al. (2002) investigated the behaviour of granular-type foamed bitumen-stabilised materials. Based on the results of this study, these mixes exhibited an increase in the cohesion in excess of 100 kPa after moderate curing, compared to the properties exhibited by equivalent granular materials. An associated moderate reduction in the friction angle of less than 10^{o} occurs after the inclusion of the binder.

These authors also found that the preconditioning of the foamed bitumen samples has a profound effect on the magnitude of the resilient modulus and the behaviour at different stress levels. An exposure to 10,000 load pulses changed the resilient deformation behaviour from stress-independent to stress-dependent. The use of conditioning pulses is necessary to simulate field conditions and therefore to obtain representative results. The change in stress-dependent behaviour after conditioning with this large number of load pulses has been interpreted as being caused by the breaking of bonds between the large aggregates. This breaking changes the state of the foamed mix from bound to unbound or granular material.

A set of models for resilient modulus, Poisson's ratio, and permanent strains has been developed in this research. To model the stress dependency of both the resilient modulus and Poisson's ratio, several models were found to be applicable to the material behaviour demonstrated by the foamed bitumen mixes. They are described in the following Sections 10.1.1 – 10.1.7.

10.1.1 Resilient Modulus and Bulk Stress (M_r-θ) Model

$$M_r = k_1 * \left(\frac{\theta}{\theta_o}\right)^{k_2} \tag{8}$$

where:

M_r = Resilient modulus
k_1, k_2 = Regression constants
θ = Bulk stress
θ_o = Reference stress (= 1kPa)

This model was found applicable to address the stress dependency of the foamed bitumen mixes.

10.1.2 Linear Model

$$M_r = k_3 * \theta + k_4 \tag{9}$$

where:

k_3 and k_4 = Regression constants

In some other mixes, the linear model was the best to address the stress dependency of the resilient modulus.

10.1.3 M_r - σ_3 - $\sigma_1/\sigma_{1,f}$ Model

Van Niekerk & Huurman (1995) developed a relationship between the resilient modulus and the vertical stress ratio ($\sigma_1/\sigma_{1,f}$) as shown.

$$M_r = k_5 * \left(\frac{\sigma_3}{\sigma_{3,0}}\right)^{k_6} * \left\{1 - k_7 * \left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{k_8}\right\} \tag{10}$$

10.1.4 M_r - θ - $\sigma 1/\sigma_{1,f}$ Model

$$M_r = k_5 * \left(\frac{\theta}{\theta_o}\right)^{k_6} * \left\{1 - k_7 * \left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{k_8}\right\} \tag{11}$$

This model is similar to the model in Equation 10 but it utilises the bulk stress instead of the confining stress.

10.1.5 M_r- θ - $\sigma_d/\sigma_{d,f}$ Model

$$\mathrm{M_r} = \mathrm{k_5} * \left(\frac{\theta}{\theta_o}\right)^{\mathrm{k_6}} * \left\{1 - \mathrm{k_7} * \left(\frac{\sigma_d}{\sigma_{d,f}}\right)^{\mathrm{k_8}}\right\} \tag{12}$$

This model is also similar to the models in Equations 10 and 11 but it uses the stress ratio represented by ($\sigma_d/\sigma_{d,f}$) instead of the ratio between the major principal stresses.

For equations 10, 11, 12:

k_5, k_6, k_7 and k_8 = Regression constants

$\sigma_{3,0}$	=	Reference value (=1kPa)	σ_3	=	Vertical stress
$\sigma_{1,f}$	=	Major principal stress at failure	σ_1	=	Principal vertical stress
$\sigma_{d,f}$	=	Deviatoric stress at failure	σ_d	=	Deviatoric stress

10.1.6 Poisson's Ratio (ν) Model

$$\nu = a + b * \left(\frac{\sigma_1}{\sigma_3}\right) \tag{13}$$

$$\nu = c * \left(\frac{\sigma_1}{\sigma_3}\right)^d * \left(\frac{\sigma_3}{\sigma_{3,0}}\right)^e \tag{14}$$

where:

a, b, c and d = Regression constants

10.1.7 Permanent Deformation Model

$$\varepsilon_p = A * \left(\frac{N}{1000}\right)^B + C * \left(e^{D * \frac{N}{1000}} - 1\right) \tag{15}$$

where:

$$A = a_1 * \left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{a_2}$$

$$B = b_1 * \left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{b_2}$$

$$C = c_1 * \left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{c_2}$$

$$D = d_1 * \left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{d_2}$$

ε_p = Permanent strain
a_1, a_2, b_1, b_2, c_1, c_2, d_1, d_2 = Regression constants
N = number of load applications

10.2 Cemented-type Foamed Bitumen Mixes

Cemented-type foamed bitumen mixes are generally produced by the addition of a moderate (1-2%) amount of active filler, such as cement or lime at about 1% to 2% by the weight of aggregates, to a small amount of foamed bitumen (<2%) to the aggregate, according to SA Interim Technical Guidelines (2002). The behaviour of this type of mix is similar to that of a cement-treated material with a high initial stiffness, high resistance to rutting and moisture, and performance properties that are less sensitive to the density that is achieved. These mixes tend to be brittle, less flexible and susceptible to fatigue cracking.

The behaviour of cemented-type foamed bitumen-stabilised materials under loading is characterised by two distinct phases. In the initial phase, the stiffness is gradually reduced because of the continuous breaking of cemented bonds at the microscopic level. The reduction in stiffness of the layer has been shown to be related to the level of tensile strains developed at the bottom of the layer and the ability of the material to deform in tension. The second phase occurs after subsequent loading results in permanent deformation.

10.3 Viscoelastic-type Foamed Bitumen Mixes

Viscoelastic-type foamed bitumen mixes are produced by adding moderate to high amounts (>2%) of foamed bitumen to the aggregate with low amount of active fillers (<1%), or without active filler (SA Interim Technical Guidelines 2002). Marginal material stabilised in this way should produce a mix of sufficient quality for moderate traffic, while good quality aggregate could produce a mix suitable for high traffic levels.

The behaviour of this type of mix with high binder contents is similar to the viscoelastic behaviour of HMA. The main advantage of these mixes is their improved flexibility represented by their high strain at failure that they can sustain compared to cemented-type mixes. The structural behaviour of these mixes is characterised by high stiffness that is reduced with time under the effect of repeated flexing by traffic loads. They exhibit permanent deformation similar to the behaviour of cemented-type foamed mixes. However, the initial stiffness of the viscoelastic foamed bitumen mixes is not as high as that of cemented-type foamed mixes.

11. Pavement Thickness Design

Up until now (2003), there has been no standard design method for pavement thickness for foamed bitumen mixes. South Africa has recently developed mechanistic-empirical models for foamed bitumen-treated materials that, for the first time, provide a method for estimating pavement thickness using foamed bitumen mixes (SA Interim Technical Guidelines 2002). The models were developed based on data generated from advanced laboratory tests and accelerated pavement tests using the Heavy Vehicle Simulator (HVS). The design charts and catalogue were developed for two foamed bitumen materials classified as FB2 and FB3. The basis for the classification of these two types of foamed bitumen mixes is given in Table 7.1, Chapter 7 of this report.

12. Environmental Impacts of Foamed Bitumen Mixes

The only study of the environmental impacts of foamed bitumen mixes appears to be that of Koenders et al. (2002). In their study the authors evaluated the environmental impacts of Warm Asphalt Mixes (WAM) produced by foamed bitumen at about 100°C and intended for use as wearing surface rather than basecourse. WAM was compared to hot mix asphalt (HMA) regarding energy consumption and fume emissions.

Because the mixing temperature of WAM is lower compared to HMA, the result was a significant reduction in energy and emissions. The energy used was about 24% to 28% less than that required to produce similar amounts of HMA.

The fume emissions at an asphalt mixing plant during WAM-Foam production was measured and compared with that from HMA production at the same asphalt plant. Asphalt fume, or airborne particulate matter, consists of an inorganic part (dust from the mineral aggregates) and an organic part. In their research, both organic and inorganic fume emissions were determined as Total Particulate Matter (TPM). The organic part, called the Benzene Soluble Matter (BSM), was also measured.

Bitumens and their fumes contain traces of polycyclic aromatic compounds (PCAs) originating from the crude oil. These compounds are of concern because some of them are irritants and others are known to have carcinogenic effects.

This investigation also included the measurements of compounds that are in the gas phase, called semi-volatiles (SV), which are not retained on a particulates filter but are collected in an adsorbent tube.

The results of this study showed a significant difference in total fume emissions detected during WAM-Foam and HMA production at the asphalt plant. BSM emissions of 0.2 and 0.5 mg/m^3 were measured during the HMA production, while the WAM-Foam process produced BSM emissions that were below the estimated detection level of 0.05 mg/m^3.

Research intended to estimate organic emissions produced during in-situ foamed bitumen stabilisation does not appear to have been conducted. Levels of emissions are however likely to be less than those from cutback bitumen stabilisation, as this involves considerable amounts of solvent loss to the atmosphere.

The environmental benefits of the process compared to emulsion stabilisation lies in the energy savings associated with the need to transport less water and the energy involved in emulsion manufacture.

Cement- or lime-stabilisation also requires high energy consumption and correspondingly high carbon dioxide (greenhouse gas) emissions from the manufacturing stage.

13. Foamed Bitumen Stabilisation in Practice

The foamed bitumen stabilisation technique has been widely used for the past 25 years in other countries. The following section reviews details and performance of some stabilisation projects (in particular from Australia) undertaken in recent years.

13.1 Australian Experience

The use of foam bitumen as a sound and cost-effective stabilising agent has been widely reported in Australia.

13.1.1 Melbourne

Maccarrone et al. (1993) provide information on the in-situ stabilisation of a number of deteriorated pavements around Melbourne City. The old pavements were milled and the grade of aggregate was adjusted by adding new aggregates. The optimum moisture and bitumen contents were then determined.

Stabilisation was carried out using a Caterpillar RR250 Road Reclaimer fitted with a foam spray bar. The rate of production of the foamed bitumen-stabilised pavements was quite high (1400 to 2500 m^2/day) making it very convenient for highly trafficked highways. Foam-stabilised trial sections showed excellent performance and proved to be very cost-effective compared to other alternatives.

Somerton Road, 1993

About 2 km of Somerton Road, in Melbourne, was stabilised to a depth of 300 mm using the foamed bitumen technique. The road was heavily trafficked (about 20,000 vehicles per day) and about 15% of this traffic were heavy trucks. This road was badly distressed with extensive cracks, rutting, patching and general loss of shape. The pavement structure was mainly 400 mm granular base with a chipseal surface. Deflection measurements showed that at least 30% of this section of the road needed strengthening.

A comparison between foamed bitumen and three other surfacings: thick asphalt overlay, thick asphalt overlay and cement recycling, and thin asphalt overlay, showed that foamed bitumen was the most cost-effective option. The cost of the foamed bitumen treatment was about 40% less than the thick asphalt alternative. Figure 13.1 shows comparison between the thickness design and life of foam bitumen (345 mm) with thick asphalt strengthening (385mm). This assessment suggested that the foamed bitumen treatment will last about 1.7 times longer than thick asphalt strengthening despite the use of low quality material.

The field performance of the foamed stabilised material was assessed by measuring the degree of compaction throughout the stabilised depth, resilient modulus evaluations, and deflection measurements.

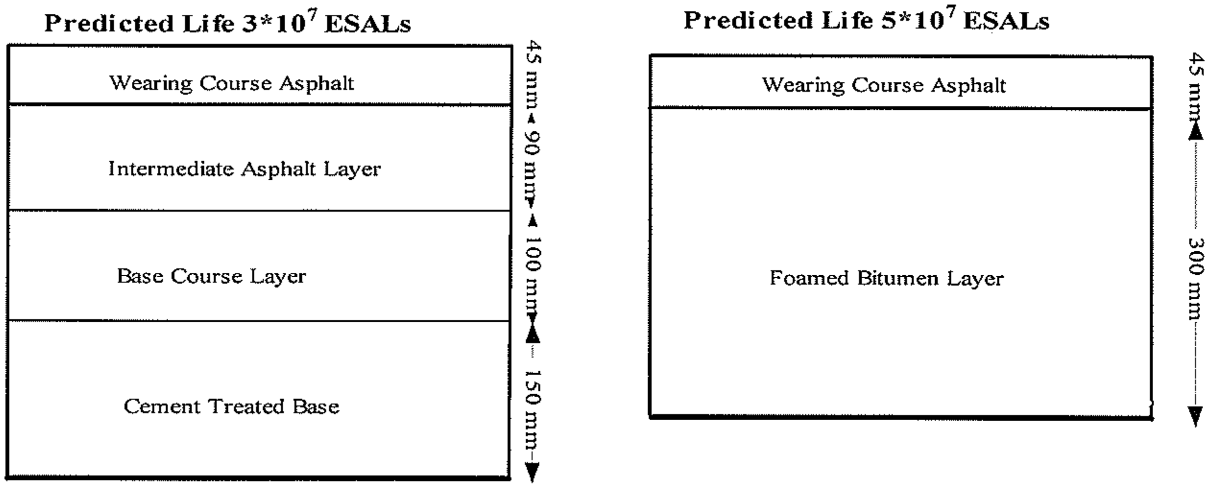

Figure 13.1 Comparison of thick asphalt strengthening with foamed bitumen.

The field assessment results showed the following:

- Density measurements indicated that compaction was achieved to the full stabilised depth.
- Resilient modulus tests were determined by laboratory tests on compacted field material. Results showed that the minimum resilient moduli values for soaked specimens exceeded 1300 MPa, while the dry specimens showed modulus values of about 4000 to 6000 MPa.
- The deflection measurements after two weeks of construction showed a significant decrease, of about 50% the deflection before stabilisation. A further decrease in deflection was expected a few months after construction because of curing of the stabilised materials.

Stud Road, 1993

Foamed bitumen stabilisation was used to rehabilitate about 22,000 m^2 of Stud Road in the eastern suburbs of Melbourne. This road is a highly trafficked arterial route. The initial construction was a chipseal over unbound crushed rock and natural gravel. The initial condition of the road showed edge cracks and high deflection readings which indicated that the pavement was lacking stiffness. About 250 to 300 mm depth was milled and stabilised with foamed bitumen.

The deflection measurements after two months of construction showed significant improvements. Samples of stabilised material taken from the roadbed on Stud Road were tested for resilient modulus (Lancaster et al. 1997). Dry modulus values ranged from 5000 to 10 000 MPa with corresponding soaked values of 2000 to 6000 MPa. In this project the optimum binder content was the value of binder that maximised both the dry and soaked resilient moduli.

Tullarmarine, 1993

Tullamarine Freeway is a heavily trafficked road (80,000 vpd) linking the city centre of Melbourne to the airport. A widening of this freeway was made by converting an emergency lane to a traffic lane. Foamed bitumen was used in the construction of the new traffic lane to stabilise 300 mm of the granular base. It was chosen by VicRoads because it would cause minimum disruption to traffic, it can be constructed quickly, and the facility can be opened to traffic a few hours after construction. The average dry resilient modulus for samples taken from the field was 5000 MPa, and the average resilient modulus for soaked specimens was 2180 MPa. The measured performance was excellent.

Browns Road, 1993

Another successful use of foamed bitumen stabilisation is the Browns Road link to the Mornington Freeway south-east of Melbourne. This road showed fatigue cracking and potholes. Foamed bitumen stabilisation was preferred over cement stabilisation or reconstruction on the basis of its lower cost and potential for elimination of shrinkage cracking. The original pavement structure consisted of chipseals over unbound material. The foam-stabilised base was 200 mm deep and covered with a double treatment chipseal. The average resilient modulus for dry samples extracted from the field was about 5600 MPa, while the average for soaked samples was 2800 MPa. Nine months after construction, the pavement showed excellent performance with no signs of shrinkage cracking.

13.1.2 Queensland

Ramanujam & Jones (2000) reported the Queensland Department (QD) of Main Roads experience in using foamed bitumen stabilisation. Different field projects were investigated in this study where foamed bitumen stabilisation had showed excellent performance. Presented in this section are some of these projects and the performance tests conducted on them. The pavements were stabilised in-situ using a Wirtgen WR2500 recycler with a specially built bitumen-foaming chamber spray bar.

Cunningham Highway, Gladfield, 1997

The first trial conducted by QD Main Roads, was made along a 1600 m section of the Cunningham Highway, Gladfield (Ramanujam & Fernando 1997). This section of pavement is heavily trafficked (AADT = 4000 vpd) with 24% heavy commercial vehicles. Laboratory tests showed that the subgrade soil is a silty clay type with 60% passing 75μm and a plasticity index of 35%. The subgrade soil also has high swelling characteristics. The dynamic cone penetrometer showed a subgrade CBR (Californian Bearing Ratio) of less than 2%. In this project, foam bitumen was used at a rate of 3.5% with lime at 2% rate. Diesel cutter was used with bitumen class C170 (equivalent to 80 pen grade) to increase its foamability. The lime was used to reduce the plasticity of the fine fraction and also to increase the adhesion between the foam bitumen and aggregate particles. Early performance was relatively good, although some isolated block cracking had occurred within two years of service.

Deflection measurements using the falling weight deflectometer (FWD), and back calculations were carried out on the data. At the end of the first day of construction, deflections of the order of 0.75 mm were measured on the treated pavement. This indicates its ability to gain strength rapidly and consequently allow early trafficking. The stabilised layer showed a high resilient modulus value of 1250 MPa. The indirect tensile strength results showed that the dry ITS was 380 kPa and the soaked ITS was 240 kPa. This means that the retained ITS was 65% which indicates good moisture resistance of the stabilised material.

Rainbow Beach Road, 1998

QD Main Roads conducted another foam bitumen stabilisation trial at Rainbow Beach Road. The length of this pavement section was relatively short, about 200 m. The material inside the wheelpath was a beach sand and that outside of the wheelpath was low quality rock. Foamed bitumen and emulsion were used to stabilise two different sections of the road.

The foam bitumen stabilisation was carried out on the basecourse layer. The foamed bitumen section held up to early traffic better than the section stabilised by emulsion. During construction, some rain fell that seriously affected the emulsion section, and it became very slick and rutted significantly.

The deflection measurement using FWD and the back analysis were done on this road to monitor the road performance after stabilisation. From the back analysis, the resilient modulus of the foamed bitumen stabilised base was 1100 MPa.

Cunningham Highway, Inglewood, 1999

Foam bitumen stabilisation was conducted at another section of Cunningham Highway, near Inglewood. The length of this section was 1600 m. Foamed bitumen and hydrated lime were used in the stabilisation process at rates of 4% and 2%, respectively. During construction high rainfall fell (about 142 mm) within the first 6 weeks before sealing. In addition, this section suffered from poor drainage because the road surface was basically flat, and the ground-water table was high from the flood irrigation used by the farms on the sides of the road. Despite the extreme conditions for moisture in the subgrade, the overall performance was relatively good. Deflection measurements were performed twice along the whole length of the stabilised section, at 6 weeks and 10 months after construction. The results showed significant decrease in deflection, which means that the pavement has stiffened noticeably since construction.

New England Highway, Allora, 1999

Another successful use of the foamed bitumen was reported in the stabilisation of New England Highway, Allora. This is a heavily trafficked highway (AADT = 2500 vpd) and the percentage of heavy commercial vehicles is about 10%. The length of the stabilised section was about 17 km and the basecourse is composed of well-graded clayey gravel. Bitumen and lime were used in the stabilisation process at rates of 3.5% and 2%, respectively.

This section of the road has performed very well since construction. Core samples were taken one year after construction and the resilient modulus values of the core samples were compared with the resilient modulus values just after construction. The core samples were taken both inside and outside of the wheelpath.

The results showed that the resilient modulus and density values decrease with depth. Also, the resilient modulus at the top of the stabilised layer approached that of samples compacted in the laboratory.

13.2 United Kingdom Experience

Khweir et al. (2002) carried out laboratory and field investigations on the use of foamed bitumen-stabilised reclaimed asphalt pavement (RAP), as basecourse. The road used for the trial of the foamed stabilised RAP was a two-lane dual carriageway in a major town in the Kingdom of Fife, Scotland. The roadway carried large articulated vehicles and bus traffic. A site investigation indicated that the foundation of the existing roadway was formed of compacted sand and large gravels overlaying a boulder-clay natural soil. A panel 1 m wide and 2 m in length was excavated to the level of the sub-base. The thickness of the base plus surface construction was 240 mm. Of that, 100 mm was made of HMA, and the 140 mm base was formed of foamed bitumen-stabilised RAP. Core samples were taken 10 days after construction. The dynamic modulus of samples was measured and values around 2000 MPa were obtained. Marshall laboratory compaction was found to closely match the field compaction under a dynamic plate compactor.

13.3 South African Experience

Paige-Green & Gerryts (1998) successfully used the foamed bitumen technique with marginal aeolian sand and Berea Red Sand. This project provided a passable, all-weather, durable road on a very limited budget. The climate of the project area is classified as humid with mild temperature and an annual moisture surplus. The annual average rainfall is about 900 mm, so the area is classified as wet for pavement design purposes. The properties of the in-situ existing sand are shown in Table 13.1.

Table 13.1 Properties of the local sands available for a road project in South Africa (Paige-Green & Gerryts 1998).

Property	Aeolian sand	Berea Red Sand
% passing 2 mm Sieve	100	100
% passing 0.075 mm Sieve	3 – 6	39
Liquid Limit, %	NP	24
Plasticity Index, %	NP	11
Linear Shrinkage, %	0	5.4
CBR	5 – 8	5 – 8

NP – not provided

To construct a basecourse out of these materials using conventional stabilisation methods was impractical. Alternative methods to import better quality materials were evaluated but none of them were considered cost effective. The use of a foamed bitumen-treated mix of the Berea Red Sand and the white aeolian sand was considered to be the most cost-effective solution. One of the key issues considered during the construction was to provide sufficient compaction of the subgrade to minimise deflection and rutting, and consequently to reduce the possibility of fatigue failure of the foamed-bitumen layer. Dynamic compaction was used with a three-sided Landpac 25 kJ impact compactor. Since the energy applied to the subgrade during the impact compaction was significantly higher than that will ever be applied by traffic, the possibility of traffic-induced rutting in the subgrade was eliminated.

The final design of the mix consisted of 89% white aeolian sand, 5% Berea Red Sand, 2% cement and 4% foamed bitumen (150/200 penetration grade). The foamed bitumen stabilisation work was done to the basecourse layer. Apart from some localised potholes resulting from the disintegration of the plastic nodules of Berea Red Sand, the observed performance of the road has been good.

Joubert et al. (1989) investigated the use of foamed bitumen to stabilise Kalahari aeolian sands, and the performance of the test sections has been monitored for almost four years. The authors concluded that the traditional CBR method of design cannot be used in the case of foamed bitumen, although it is still valid for subgrade layers. In addition, foamed bitumen basecourse was found not to contribute to the structural strength in the first two years after construction. They also concluded that cracks are more likely to appear if the percentage of fines (<75 μm) is greater than 12%. Cracks were attributed to the plastic behaviour of these mixes.

To prevent or minimise this behaviour, Joubert et al. recommended the use of well-graded aggregates. The authors also concluded that the vane test and dynamic cone penetrometer (DCP) provide valuable information about the suitability of the stabilised materials. A minimum recommended strength value measured using the vane shear test was 300 kPa and the maximum recommended DCP value was 8 mm/blow. Overall, this investigation demonstrated that foamed bitumen can be used successfully to provide suitable construction materials.

13.4 North American Experience

Foamed bitumen stabilisation has not been used extensively in North America. Among the limited studies reported there, foamed bitumen was used to recycle reclaimed asphalt pavements (RAP) in Indiana (van Wijk & Wood 1983). About 130 mm of the asphalt pavement were milled, blended with additional aggregates, and then mixed with foamed bitumen. The foamed bitumen mix was compacted in two layers, each 65 mm thick. The expansion ratio of the foam was 10 and the half-life time was 12 seconds. The study showed the importance of the water content of the compacted material affecting the final mix properties which agrees with the South African studies. The properties of foamed-stabilised RAP showed reasonable improvement in sites where the moisture susceptibility improved significantly.

Part II: Experimental Investigation

14. Introduction

This Part II of the research study aimed at conducting preliminary testing to examine the feasibility of using foamed bitumen as an alternative for basecourse stabilisation using local New Zealand materials. Although several sources of bitumen with different grades were investigated, only the results of 80/100 and 180/200 penetration grade bitumens from two different sources (Sources I and II) are presented in this report. An aggregate gradation complying with the AP-20 gradation envelope was used, with a minor modification in the proportion of fines as explained below. The optimum moisture content for the aggregate used was determined for different compaction efforts (Gyratory compaction at 80 cycles, Marshall compaction at 75 blows per face, and vibratory compaction). Gyratory compaction only was used to prepare compacted specimens in this preliminary study. The foaming properties of each bitumen type were thoroughly investigated. In addition to the foaming properties, the physical properties of each type of bitumen were studied.

Two groups of foam mixes were prepared using the foam produced from Source I bitumen of grade 180/200. The first group of mix is made with AP-20, using fly ash and 2% cement (as a partial replacement of the fly ash) as the fine fraction (< 75μm), while the second group was identical to the first type, except that no cement was used and only fly ash was used as mineral filler.

The moisture content required to maximise the resilient modulus and bulk density of the compacted foam-stabilised mix for the two types was determined. The relationship between resilient modulus and curing time was also investigated.

After the optimum moisture content required to disperse the foam and provide the highest compaction had been determined, the next step was to determine the optimum foam content that yielded the maximum resilient modulus for the two types of foam-stabilised mixes.

Comparisons between the resilient modulus results for the mixes made with the two sources and two grades of bitumen were made, using a similar gradation envelope but with only 1.0% cement as a partial replacement of the fly ash.

A comparison between foam-stabilised aggregates and other stabilisation techniques, including cement stabilisation and lime stabilisation, was made to establish the feasibility of foam mixes as an alternative for stabilisation of unbound materials.

15. Testing of Materials

15.1 Aggregate Gradation

Figure 15.1 shows the lower and upper limits of the ideal zone for foamed bitumen mixes. The midpoints of AP-20, AP-40, and the midpoint of the ideal zone gradation envelopes are shown on the graph. The midpoint of the AP-20 band is quite close to the midpoint of the ideal zone except for the fine grain size part of the curve. The upper limit of the AP-40 will fit within the ideal zone better than the midpoints which are depicted on the graph in Figure 15.1.

To prepare consistent samples complying with the midpoint of the ideal zone, a large amount of AP-20 aggregate was sieved and separated into different sizes (19 mm, 12.5 mm, 9.5 mm, 4.75 mm, 2 mm, 75 μm, and passing75 μm). Aggregates sizes larger than 19 mm were not used in the preparation of foamed bitumen mix samples to ensure that the maximum aggregate size does not exceed ¼ of the 100 mm diameter specimen. Fly ash was used to provide the mineral fillers required to adjust the fine part of the AP-20 curve in order to comply with the midpoint of the ideal zone as shown in Figure 15.1. The percentage of mineral fillers (<75 μm) in this mix design was set as 12.5%. Also cement was used as a partial replacement of the fly ash at 1% and 2.0% by the dry weight of aggregate.

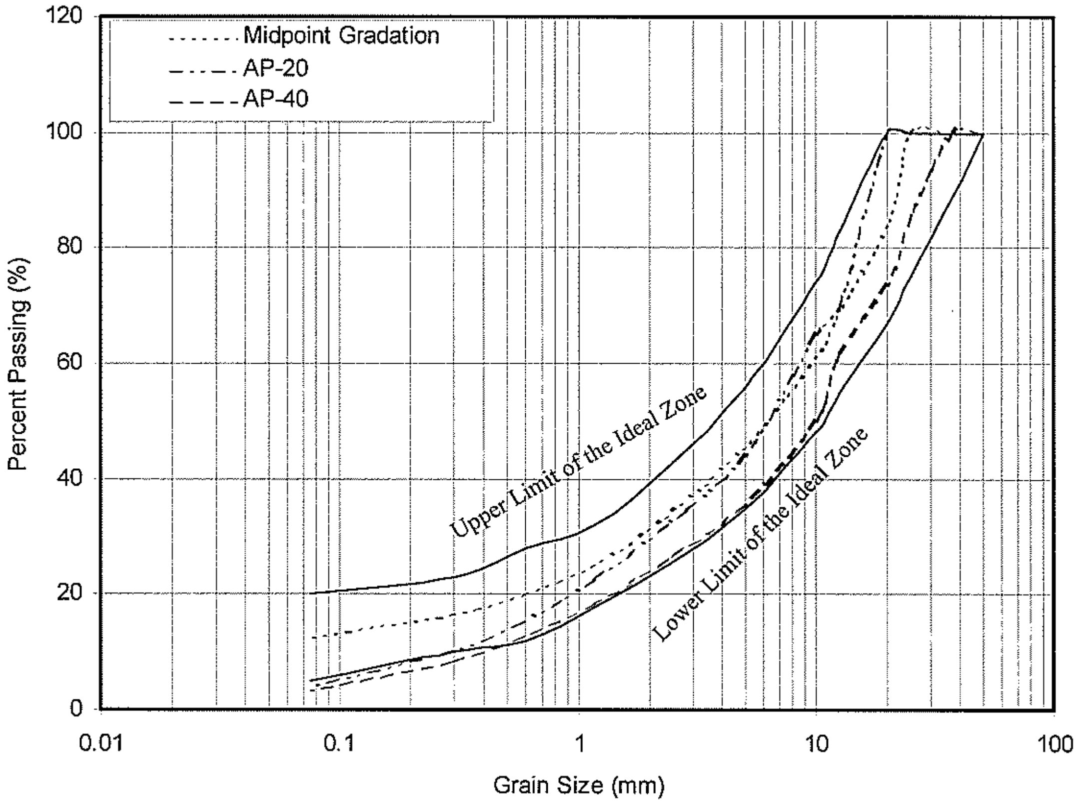

Figure 15.1 Gradation envelopes for AP-20, AP-40, and the ideal zone.

15.2 Optimum Moisture Contents for Different Compaction Efforts

Specimens 100 mm by 63 mm in size were prepared using three types of compactors: Gyratory, Marshall, and Vibratory. The moisture content versus the dry density for the AP-20 aggregate was studied for the three different compaction efforts. In order to obtain statistically unbiased and representative results of the water content-density relationship for each compactive effort, three replicates were prepared at each water content for five water contents (2, 4, 6, 8, and 10%).

For the gyratory compaction, 'Gyropac' Gyratory compactor manufactured by IPC Ltd Australia was used (Figure 15.2). Each specimen consisted of about 1300 g of aggregates and was subjected to 80 cycles of gyratory compaction at 240 kPa pressure and a gyration angle of 2^{o}. For the Marshall compaction, 75 blows per face were applied using the standard manual Marshall compaction hammer. For the vibratory compaction, 100 mm-diameter specimens were prepared. Since the standard method was based on 150 mm-diameter specimens, the compaction energy per unit volume was back-calculated and the compaction time for the 100 mm-diameter specimen was adjusted to maintain the same the energy per unit volume.

Figures 15.3 to 15.5 show the moisture-density relationships for the Gyratory, Marshall and Vibratory compactions, respectively. The figures show that the optimum moisture content for the pure aggregate is different from one compaction method to the other, depending on the compaction effort and the way it affects aggregate orientation during the compaction process. The gyratory compaction is shown to provide the highest density and the lowest optimum water content. The Marshall and vibratory compaction methods provided similar maximum dry density values, with slightly higher optimum moisture content for the vibratory compaction.

In this feasibility study, foam-stabilised specimens were prepared by gyratory compaction. However, in the second phase of the project, the effect of compaction method on the mechanical properties of the mix will be investigated. In the last phase of this project where full-scale experimental sections will be constructed, a comparison between the laboratory compaction methods and field compaction will be made. The outcome of this study will help in identifying the suitable laboratory compaction method that provides the best match to field compaction.

Figure 15.2 University of Canterbury Gyratory Compactor (Gyropac).

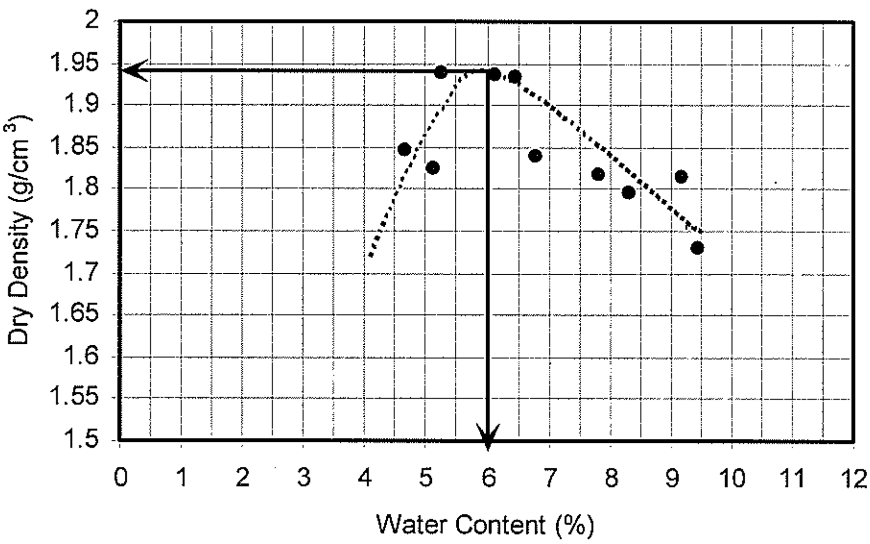

Figure 15.3 Optimum moisture content using the Gyropac compactor at 80 compaction cycles.

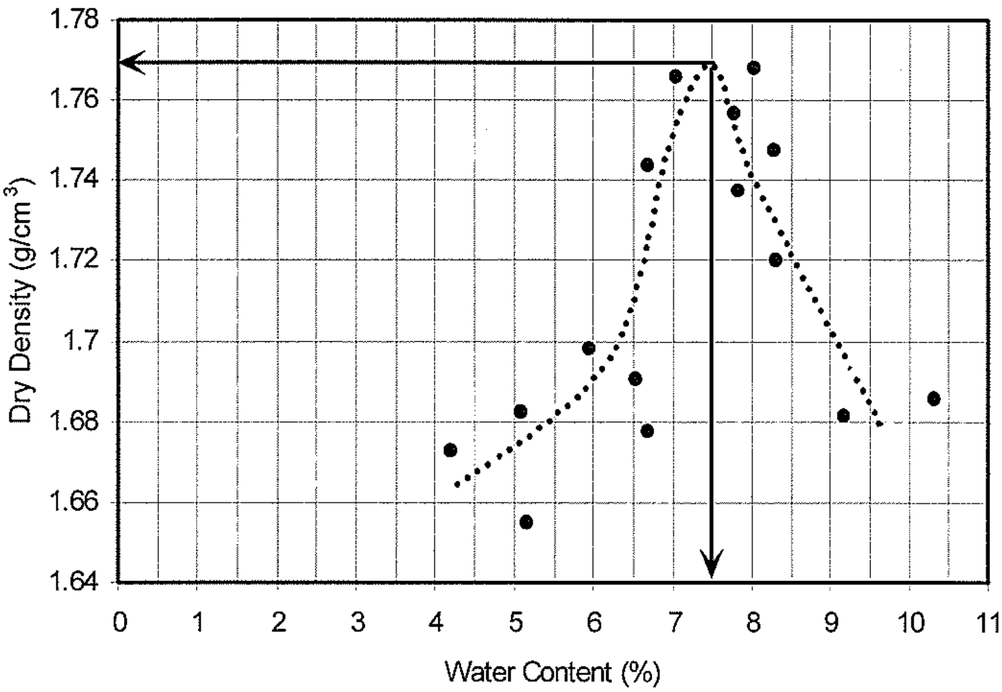

Figure 15.4 **Optimum moisture content using the Marshall compactor at 75 blows per face.**

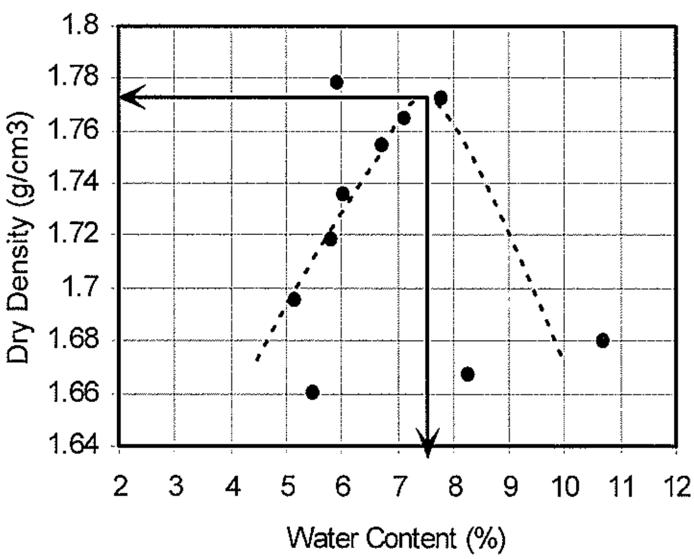

Figure 15.5 **Optimum moisture content using the vibratory compactor.**

15.3 Bitumen

15.3.1 Physical and Engineering Properties

Two sources of bitumen currently used in New Zealand were investigated. For each source, two penetration grades, namely 80/100 and 180/200, were considered for the physical characterisation testing. Penetration, viscosity, and softening point were measured.

Because of the sudden change of temperature upon contact of bitumen with cold water during the foaming process, the temperature susceptibility of the bitumens might have an effect on the foamability and the quality of the foam that is produced. Thus, the penetration and viscosity were also measured at different temperatures to determine the bitumen temperature susceptibility (Table 15.1). The penetration indices for the different bitumens were calculated. The temperature susceptibility results will be reported in the second phase of the study as part of a thorough investigation into the relationship between temperature susceptibility and foamability.

Table 15.1 Physical properties of the different sources and grades of bitumen used.

Property	Source 1		Source 2	
	80/100	**180/200**	**80/100**	**180/200**
Penetration at 25 °C	86	**169**	78	187
Viscosity* at 60 °C (mPa.s)	50129	18654	84312	20963
Viscosity* at 135 °C (mPa.s)	435	250	550	261
Softening Point (°C)	47.5	39	47.5	38

80/100, 180/200 Penetration
*by Brookfield viscometer

15.3.2 Foaming Characteristics of Bitumens used in New Zealand

The characteristics of the foam produced from bitumens used in New Zealand (which are imported from several sources) were investigated. The foaming properties of only Source I and Source II bitumens are presented in this report, and their expansion ratios and half-life times were measured. The Wirtgen model WLB10 foaming apparatus at University of Canterbury (shown in Figure 1.2) was used in the experimental work. The following procedure was used:

1. Heat at least 10 litres of bitumen in an oven to 180°C.
2. Preheat the laboratory foaming plant to about 180°C before putting in the bitumen.
3. Put the bitumen into the kettle of the apparatus and circulate until the temperature is stabilised at 180°C.
4. Measure the discharge rate of the bitumen from the nozzle, for the given pump settings of the apparatus. This is done by discharging bitumen into a tared vessel for 3 seconds.

5. Weigh on a balance to at least 0.5 g accuracy. This procedure should be repeated at least twice using different time periods (e.g. 4 and 5 seconds) without any foamant water or air being added during discharge.

6. Calculate the amount of bitumen required for the given measurement vessel in the laboratory. The vessel and the measuring gauge shown in Figure 15.6 are calibrated for 500 g of bitumen. The gauge shown has graduations of 6 expansion ratios. This means that, if the foam reaches the first step, then it has an expansion ratio of 6. If it reaches the second step, then the expansion ratio is 12, and so on.

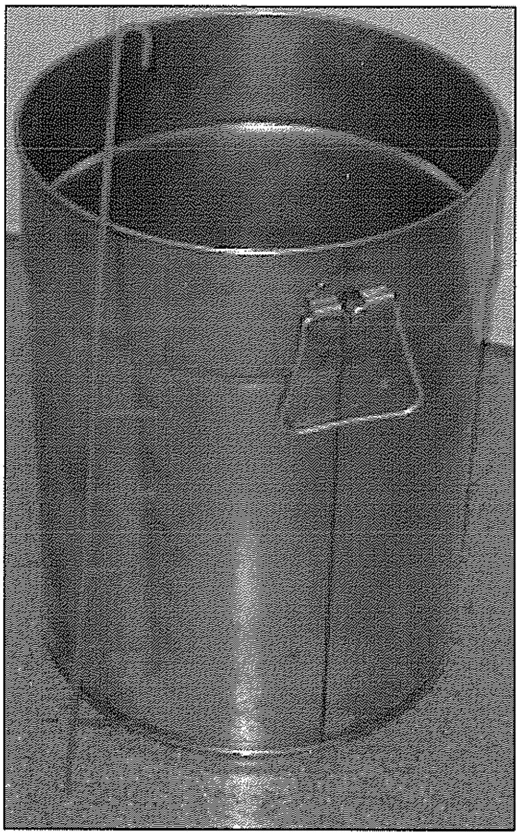

Figure 15.6 Vessel and calibrated stick for measuring Expansion Ratio.

7. Calibrate the foamant water flow to achieve the desired application rate e.g., 2.5% by mass of bitumen. The bitumen flow rate measured in step 5 is required for the calculation of the water flow rate.

$$Q_w = \frac{\%P_w * Q_b}{100 * \gamma_b} * \frac{3600}{1000} \qquad (16)$$

where:

Q_w	=	Water flow rate in litres/hour
$\%P_w$	=	Percentage of water by the weight of bitumen
Q_b	=	Bitumen flow rate in grams/second
γ_b	=	Density of bitumen (assumed to be 1 g/cm^3)

8. The bitumen discharge time or spray time should then be set on the apparatus to yield 500 g of bitumen.
9. Foamed bitumen is discharged into the vessel for the calculated spray time. Immediately after discharging the foam, a stopwatch is started.
10. The expansion ratio is measured using the graduated gauge and the time recorded for the foam volume to collapse to half of its original value.
11. Repeat discharge and measurement of the foam expansion ratio and half-life three times for a given set of conditions.
12. Repeat steps 7 to 11 for a range of at least four foamant water concentrations. Typically, values of 2.0%, 2.5%, 3.0%, 3.5%, and 4.0% are used.

The water pressure was set at 5 bar while the air pressure was fixed at 3 or 4 bar during the test. The air was simultaneously injected with the water into the bitumen to help improve the foamability of the bitumen.

Table 15.2 summarises the foamability results for the two sources of bitumen for each grade. Table 15.3 contains the optimum water content which will yield the optimum foaming characteristics. According to the SA Interim Guidelines, only 80/100 penetration grade of Source I is classified as poor. The classification was made assuming that the temperature of the aggregate during mixing is 25°C or higher.

Although the foam produced from the 80/100 penetration grade of Source I was classified as poor, it could be mixed effectively with aggregates without causing any problems, and the mechanical properties of the produced mix were quite promising (see Section 15.9 of this report). This suggests that the expansion ratio and half-life time, which are empirical parameters, cannot be solely relied on for the classification of foaming characteristics. A new foam characterisation technique is under investigation by the research team and will be discussed in the second phase of this study.

Figures 15.7 to 15.10 portray the foamability results for the two sources of bitumen and for the two penetration grades. The 180/200 penetration grade from Source II provides the best foam characteristics, and its optimum moisture content, expansion ratio and half-life are 2.6%, 16.5, and 13.5 seconds, respectively. The foam index of this grade of bitumen is 224 (see equation 2) which is classified as a very good quality foam according to the SA Interim Guidelines.

Note that the point of intersection of the expansion ratio curve and half-life has no meaning and it does not correspond to the optimum foaming conditions because expansion ratio and half life have different units, and changing the scale of either quantity will change the point of intersection.

The optimum water content can be determined as the point which corresponds to the maximum foam index, according to the SA Interim Guidelines. However, a close look at Figure 15.11 shows that not all types of bitumen have a peak or maximum value for the foam index. Only 180/200 penetration grade from Source I shows a peak value at 3.75% water content. Therefore, the research team is investigating another method to determine the optimum water content and consequently the optimum foam characteristics. This new technique will be discussed in the second phase of this study. However, in this first phase using the original method, the optimum water content was determined with a minimum half-life time of 7 seconds.

Table 15.2 Bitumen foaming parameters.

Source-I (80/100)				Source-I (180/200)			
W_c	ER	HLT	FI	W_c	ER	HLT	FI
2	6	12.7	40.0	2	5.8	10.4	37.4
2.5	7.7	6.2	53.4	2.5	7.6	9.5	56.4
3	9.7	4.7	72.8	3	9.3	8.8	77.3
3.5	11	3.5	88.4	3.5	11.3	7.6	99.1
4	12	4.0	93.4	4	13.9	7.2	130.7
4.5	12	3.2	97.5	4.5			
Source-II (80/100)				**Source-II (180/200)**			
W_c	ER	HLT	FI	W_c	ER	HLT	FI
2	12.0	10.3	119.7	2	9.7	23.7	123.5
2.5	12.5	8.0	115.6	2.5	15.3	16.0	218.4
3	13.3	7.5	124.6	3	18.0	6.3	176.6
3.5	15.3	5.7	141.9	3.5	20.0	5.7	196.7
4	17.7	4.9	163.5	4	24.0	5.0	237.9
4.5	20.0	4.3	185.1	4.5	24.0	5.2	240.2

W_c = Water Content, ER = Expansion Ratio, HLT = Half-Life (seconds), FI = Foam Index

Table 15.3 Optimum bitumen foaming parameters.

Bitumen Type	OWC	ER	HLT	FI	Classification*
Source-I 80/100	2.5	7.8	7.0	55.7	Poor
Source-I 180/200	3.5	12.0	7.8	109	Moderate
Source-II 80/100	3	13	7	114.9	Moderate
Source-II 180/200	2.6	16.5	13.5	224	Very Good

* Classification as given in the SA Interim Technical Guidelines, SA Transportek 2002.

OWC = Optimum Water Content, ER = Expansion Ratio,
HLT = Half-Life (seconds), FI = Foam Index

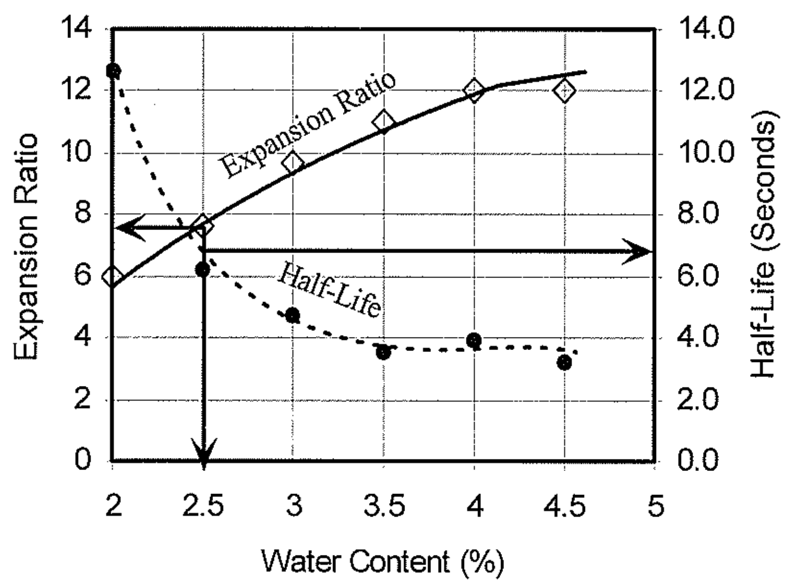

Figure 15.7 Relationship between expansion ratio and half-life versus water content for Source I 80/100 grade bitumen.

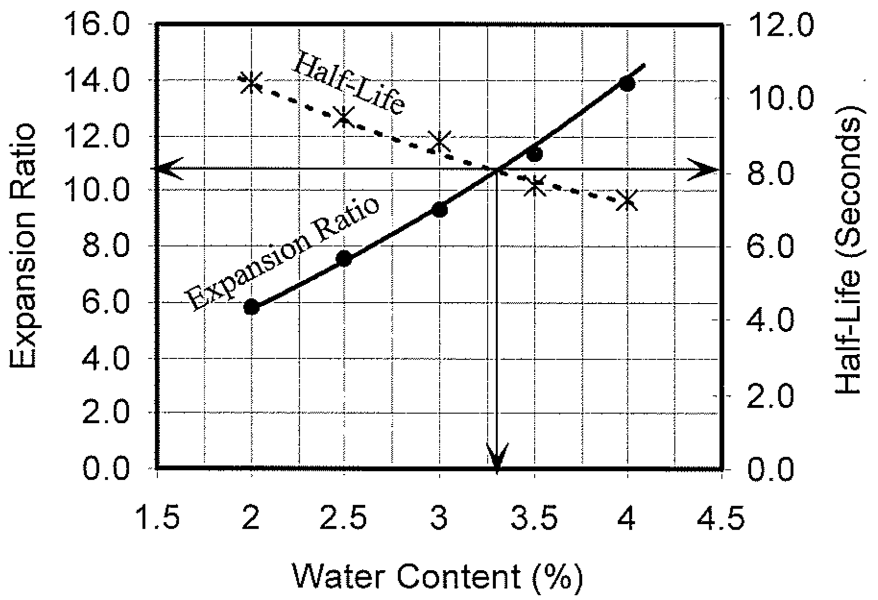

Figure 15.8 Relationship between expansion ratio, half-life and water content for Source I 180/200 grade bitumen.

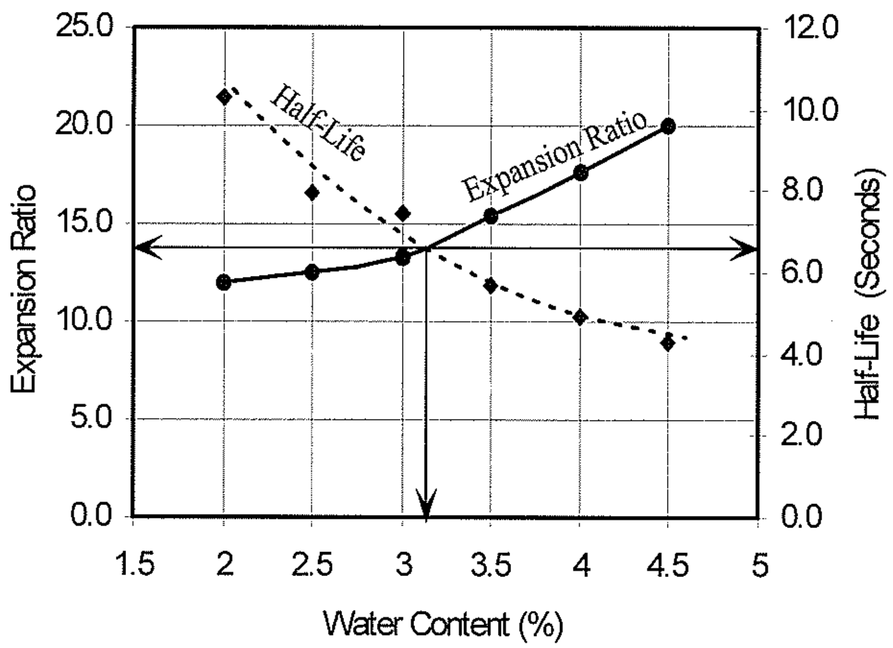

Figure 15.9 **Relationship between expansion ratio, half-life and water content for Source II 80/100 grade bitumen.**

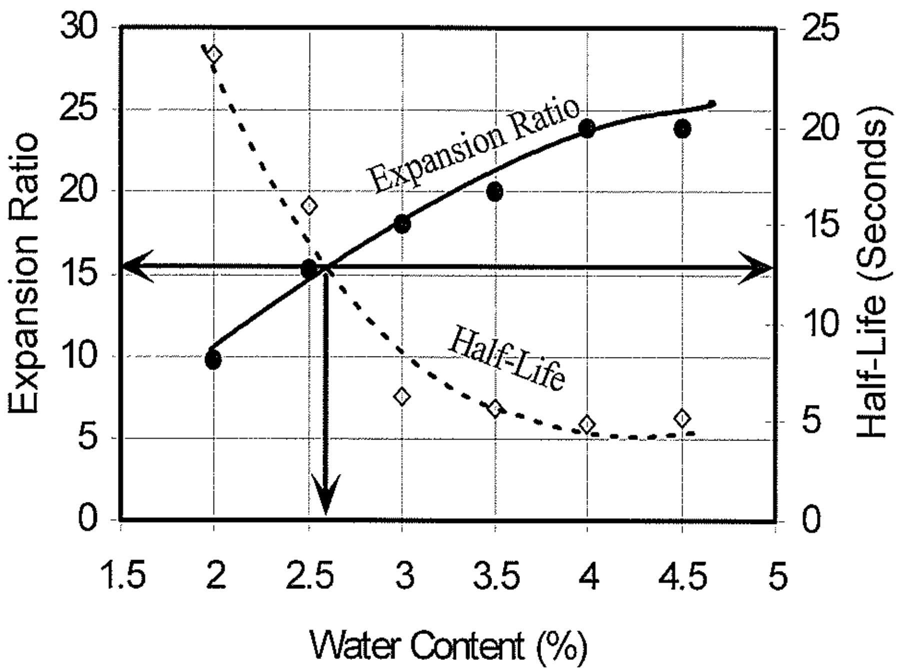

Figure 15.10 **Relationship between expansion ratio, half-life and water content for Source II 180/200 grade bitumen.**

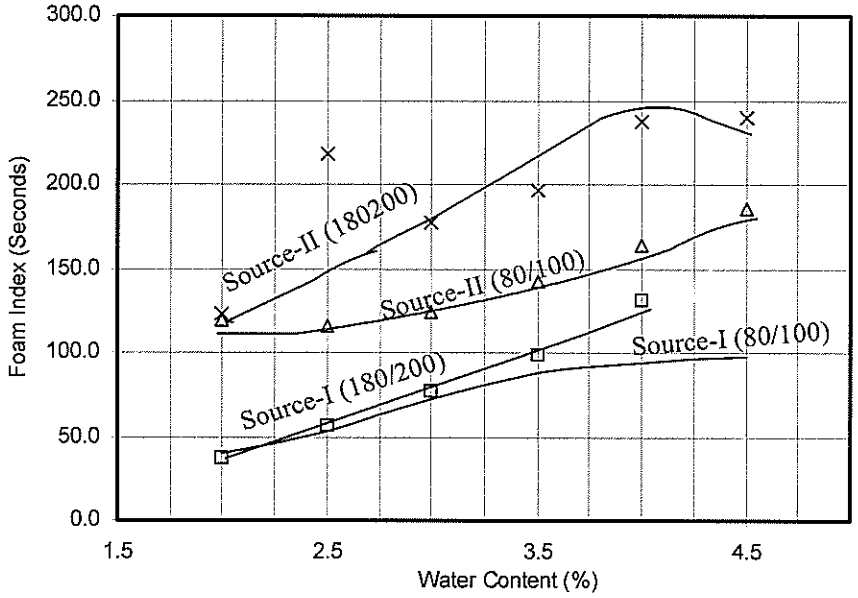

Figure 15.11 Effect of water content on the foam index for the two sources of bitumen.

15.4 Resilient Modulus Testing

Resilient modulus is based on the recoverable strain under repeated loads and is defined as:

$$M_r = \frac{\sigma_d}{\varepsilon_r} \tag{17}$$

where:

σ_d is the deviator stress,
which is the axial stress in an unconfined compression test or the axial stress in excess of the confining pressure (i.e. $(\sigma_1\text{-}\sigma_3)$) in a tri-axial compression test.

ε_r is the resilient or recoverable strain under repeated loads.

For the indirect tensile test, the specimen is loaded on the diametrical plane and the lateral recoverable strain is measured by two linear variable differential transformers, LVDTs, as shown in Figure 15.12. The resilient modulus is computed by:

$$M_r = \frac{P*(\nu + 0.2734)}{\delta * t} \tag{18}$$

where:

P = Dynamic repeated load (N)
ν = Poisson's ratio
δ = Horizontal recoverable deformation (mm)
t = Height of specimen (mm)

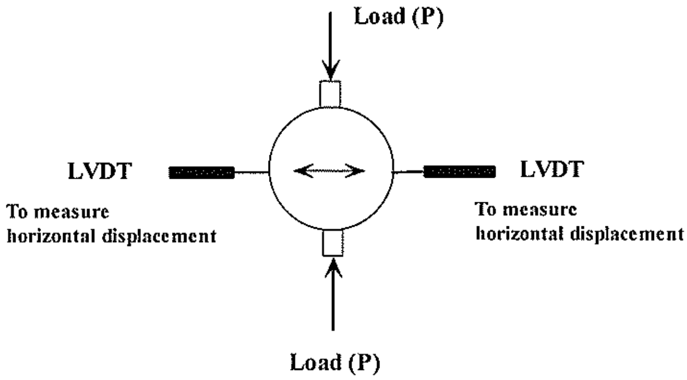

Figure 15.12 Indirect Tensile Test.

In this research, the resilient modulus was measured using repeated load indirect tensile techniques, according to AS2891.13.1:1995 procedure. The Material Testing Apparatus (MATTA), a pneumatic testing machine shown in Figure 15.13, was used to measure the resilient modulus in our research.

This machine is capable of applying an approximately triangular shaped or haversine load pulse, with a rise time in the range of 0.025 s to 0.1 s, with an accuracy of ±0.005 s. The rise time is defined as the time required for the load pulse to rise from 10% to 90% of the peak force. The machine is provided with a temperature controlled cabinet that is capable of maintaining a temperature within ±0.5°C of the test temperature. For this test, the temperature was maintained at 19°C which was about the room temperature at the time of testing.

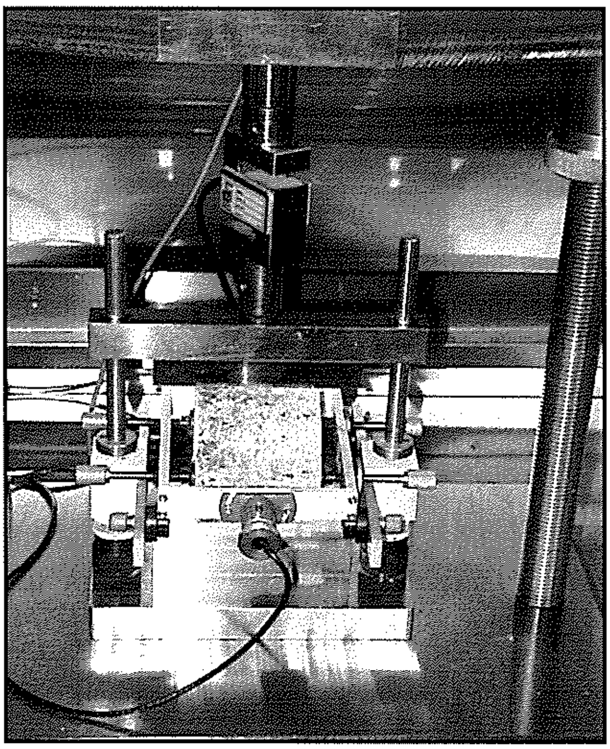

Figure 15.13 University of Canterbury MATTA machine used to measure resilient modulus.

15.5 Optimum Moisture Content for Mixing and Compaction

Various techniques have been used by different researchers to determine the optimum moisture content required to achieve the best dispersion of the foam and the best compaction of the mix. The total fluid content and the fluff point are two different approaches which have been discussed in detail in Section 5.5 of the literature review in Part I of this report. In this investigation the foam produced from Source I bitumen, grade 180/200, was used.

The optimum moisture content for mixing and compaction was determined for two groups of mixes. The first group is pure aggregate with fly ash and 2% cement as mineral fillers (<75 μm fraction); and the second group is identical to the first group except that fly ash only was used as mineral filler. Both groups of mixes comply with the midpoint gradation of the ideal zone for foam mixes (Figure 15.1).

For the first group, five batches of aggregates each of 2500 g were prepared, assuming an initial estimate of 3.5% for the optimum foam content. The optimum moisture content for the pure aggregate using the gyratory compactor was calculated as 6.0% (Figure 15.3). Water contents of 70%, 80%, 90%, 100%, and 120% of the optimum moisture content (i.e. 4.2%, 4.8%, 5.4%, and 7.2% moisture contents) were investigated. Each batch was mixed with foam at 3.5% by dry weight of aggregate. Each batch was then divided into two samples, each compacted in the standard gyratory compaction mould at 80 cycles, 2^{0} gyration angle and 240 kPa (Figure 15.14).

Resilient modulus tests were conducted on the compacted specimens at different water contents after different curing times, according to the AS2891.13.1:1995 procedure. Figure 15.13 shows the resilient modulus test equipment (MATTA) used in the study. Tables A1 and A3 in the Appendix of this report contains the results of the bulk density and resilient modulus for the two groups at different water contents.

Figures 15.15 and 15.16 show water content versus bulk density and resilient modulus, respectively, for group 1 specimens (containing fly ash + 2% cement). Both bulk density and resilient modulus of the compacted mix show the same trend with the water content. It is apparent that 6.0% water content (100% of the optimum moisture content of the virgin aggregate) maximises both the density and resilient modulus. Therefore, water content of 6.0% is considered the optimum water content for mixing and compaction for this group 1 type of mix.

Figures 15.17 and 15.18 show the relationship between bulk density, resilient modulus and water content for group 2 specimens (containing only fly ash as filler). The maximum bulk density occurs at water content of 7.2% (120% of the optimum moisture content of the pure aggregates). The trend of the resilient modulus and water content is similar to that of density and water content. Clearly a water content of 7.2% maximises both bulk density and resilient modulus of this group of mixes. Therefore, for group 2, water content of 7.2% is the optimum water content that will produce the best dispersion of the foam and yield the best compaction of the mix.

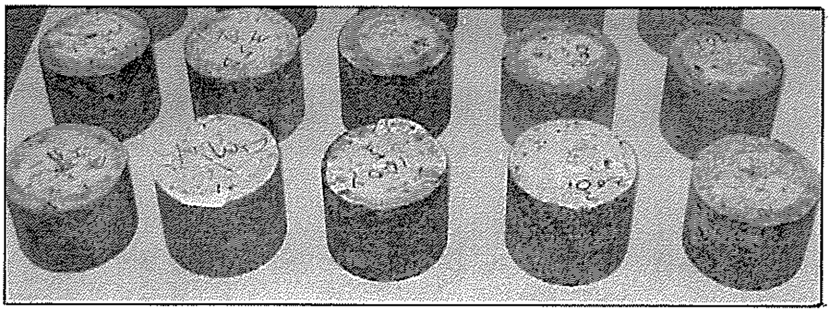

Figure 15.14 Marshall-size specimens prepared using the Gyropac compactor.

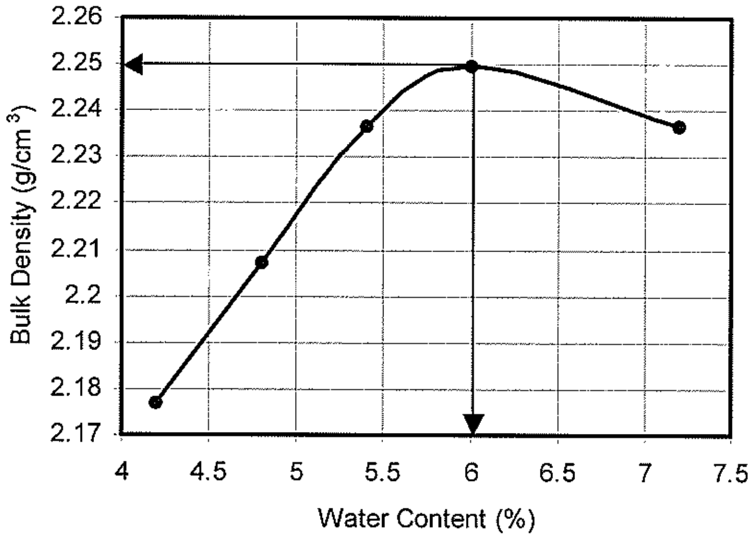

Figure 15.15 Relationship between mix density and water content for specimens containing fly ash and 2% cement as filler (passing 75 μm) and 3.5% foamed bitumen.

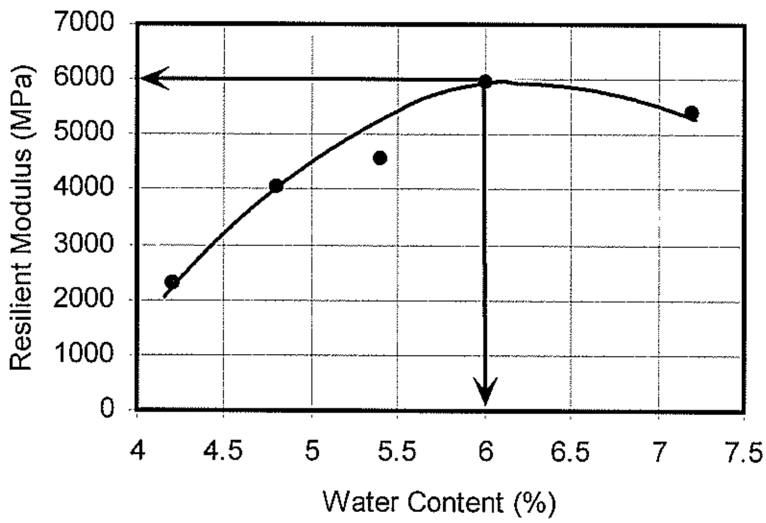

Figure 15.16 Relationship between resilient modulus and water content for specimens containing fly ash and 2% cement as filler (passing 75μm) and 3.5% foamed bitumen, after 7 days curing.

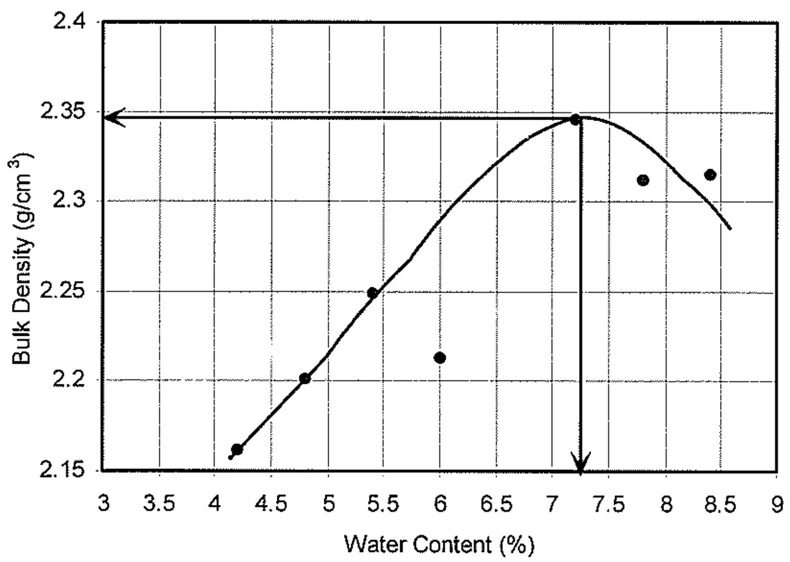

Figure 15.17 Relationship between mix density and water content for specimens containing fly ash as filler (passing 75µm) and 3.5% foam bitumen.

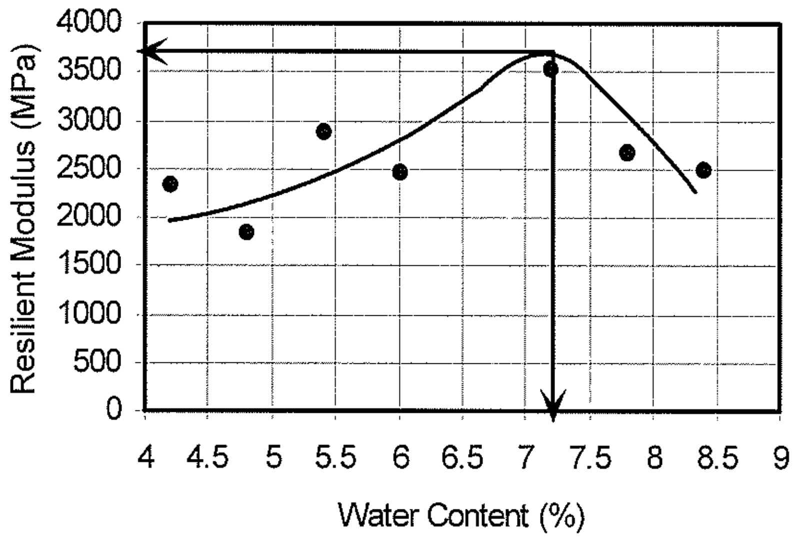

Figure 15.18 Relationship between resilient modulus and water content for specimens containing fly ash as filler (passing 75 µm) and 3.5% foamed bitumen, after 7 days curing.

15.6 Curing

Different curing methods have been suggested in the literature. Most of these methods allow for a one-day curing at room temperature, followed by 72 hours of oven curing at 40°C or 60°C. Oven curing is used to represent long-term curing in the field by accelerating water loss. Oven curing is unsuitable in the case of cement modified mixes because it will stop cement hydration and seriously affect the resulting strength of the mix. Oven curing thus does not accurately simulate field conditions in the case of cement-modified mixes. This is not the case however for mixes modified by fly ash filler.

Figure 15.19 shows the relationship between resilient modulus and curing time for the first group of mixes (containing fly ash and 2% cement) prepared at the optimum moisture content. The foam-stabilised mixes obviously have a rapid increase in the resilient modulus with time. In about four days (about 100 hours) at room temperature, the resilient modulus increases by 3 to 4 times its initial value that was recorded after a 24-hour curing period. After six days (about 150 hours) the gain in resilient modulus decreased considerably. To calculate the percentage increase in strength, the maximum strength was assumed to have been gained after one week of room temperature curing. Thus, about 80% of the maximum strength was achieved after about 4 days (80 hours).

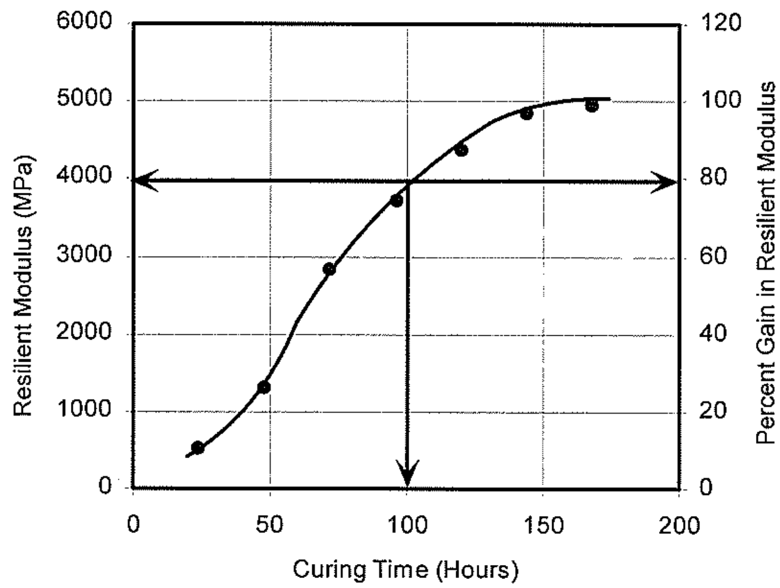

Figure 15.19 Relationship between resilient modulus and curing time for specimens containing fly ash and 2.0% cement as filler (passing 75 μm) and 3.5% foamed bitumen.

For group 2 specimens (without cement), Figure 15.20 shows a similar trend of the resilient modulus with the curing time. A fast increase in the resilient modulus is obvious for the first 100 hours of curing, then the rate of increase of the resilient modulus slows down. For this mix, the maximum strength was determined after drying to constant weight at 40°C. After 100 hours of curing, a 60% gain in the resilient modulus had occurred and a modulus value of about 2500 MPa was obtained, compared to an 80% gain and a resilient modulus of 4000 MPa for the first group. Adding 2% cement has a significant effect on the value and rate of increase of the resilient modulus.

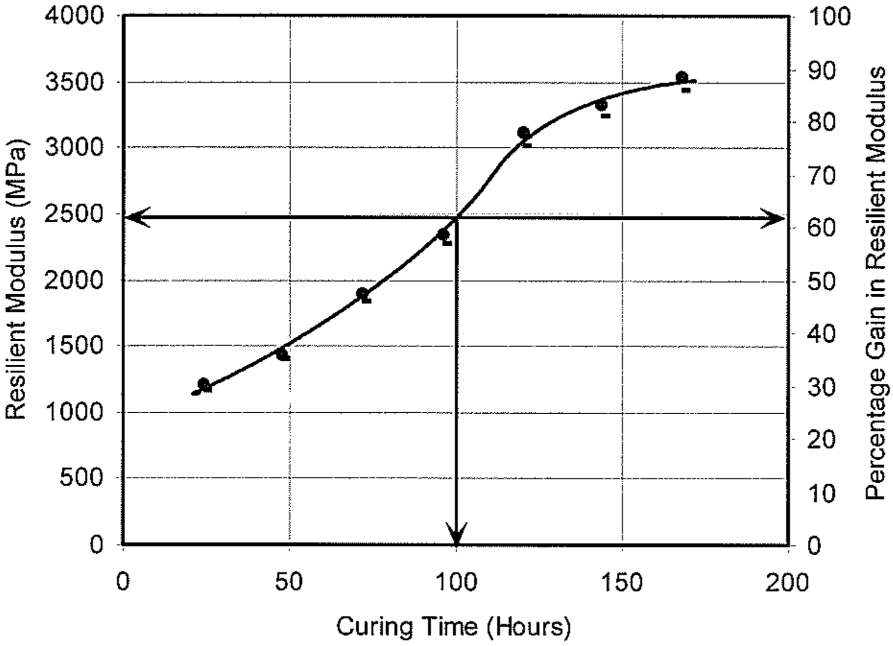

Figure 15.20 Relationship between resilient modulus and curing time for specimens containing fly ash as filler (passing 75μm) and 3.5% foamed bitumen.

For both groups of mixes, the resilient modulus after 24 hours ranges from 400 to 1200 MPa which is good enough to sustain the traffic loads without significant problems, thus allowing an early opening of the road to traffic.

15.7 Optimum Foamed Bitumen Content

After the determination of the optimum moisture content for mixing and compaction of the foam-stabilised mixes, the optimum foam content to maximise a certain mechanical property is required. In this investigation, the optimum foam content is defined as the percentage of foam by the dry weight of aggregate required to maximise the resilient modulus. Other researchers used the index of retained strength (after soaking in water) as the property which needs to be maximised. However, the principles of determining the optimum foam content remain the same.

Mixes of group 1 (containing fly ash and cement) were used. Five 2500 g batches of aggregate, each complying with the midpoint gradation of the ideal zone, were prepared. The water content was fixed at 6.0% while the foam content was set to 2.0%, 2.5%, 3.0%, 3.5%, 4.0% of the dry weight of aggregates. Figure 15.21 shows the relationship between the foam content and the resilient modulus after 7 days curing at room temperature. These results clearly show that the optimum foam content is 3.0%. Tables A2 and A4 in the Appendix contain the resilient modulus values and bulk density for the two groups at different foam contents and curing times.

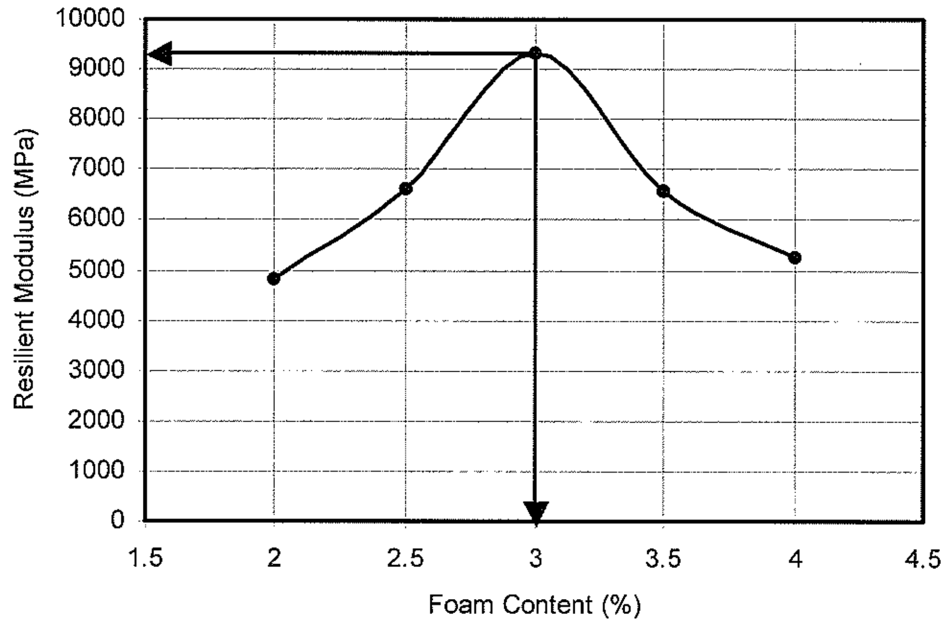

Figure 15.21 Relationship between resilient modulus and foam content at 6.0% water content, after 7 days curing at room temperature, for group 1 specimens.

15.8 Determination of Optimum Foamed Bitumen and Water Contents

Determination of the optimum water and foamed bitumen contents from individual graphs does not provide the complete picture of the different alternatives which could also yield the same maximum mechanical property. Therefore, the research team developed a new technique where the optimum content for both water and foam can be determined simultaneously. Graphs of resilient modulus and bulk density contours were created as shown in Figures 15.22 and 15.23. The figures show that resilient modulus and bulk density are affected by both foamed bitumen and water content during mixing. For example, if a specific value of resilient modulus is desired, several combinations of foamed bitumen and water contents can be used. Also, the figures provide guidelines on how the mix ingredients can be adjusted to improve the properties of a specific mix. Both Figures 15.22 and 15.23 provide the same optimum quantities for foam and water contents. Clearly 3.0% foam and 6.0% water content maximise both resilient modulus and bulk density.

Optimum water and bitumen contents were determined in a similar fashion for the group 2 mixes (i.e. without cement). Results are shown in Figures 15.24 to 15.26.

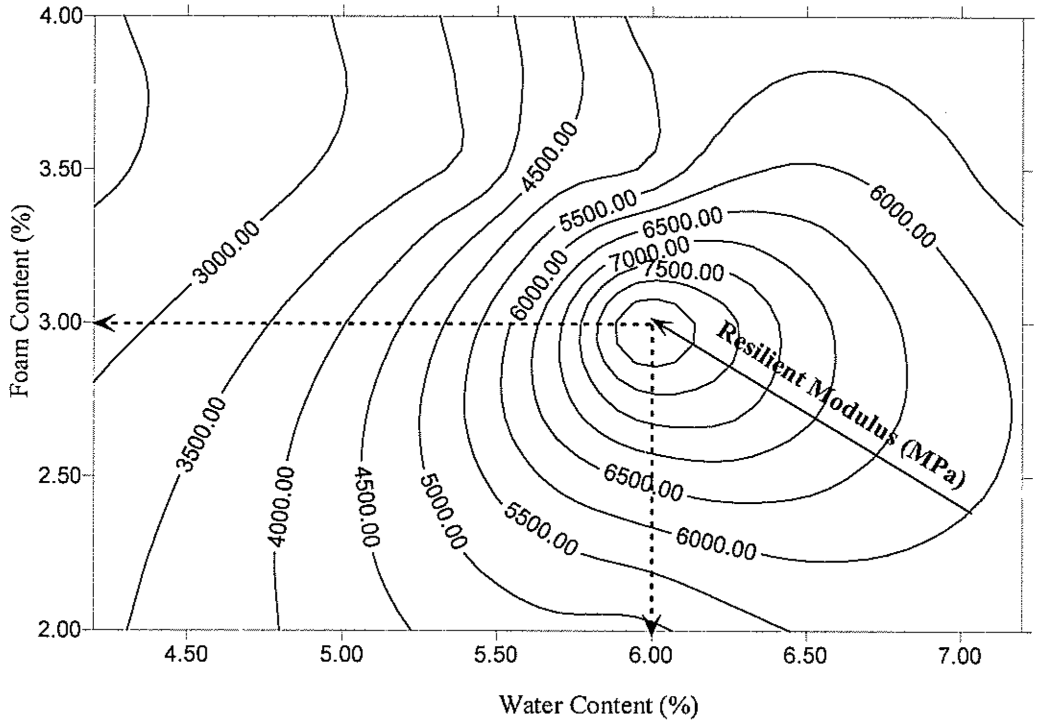

Figure 15.22 Relationship between foam content, water content and resilient modulus, after 7 days curing at room temperature, for group 1 specimens (fly ash and cement as filler).

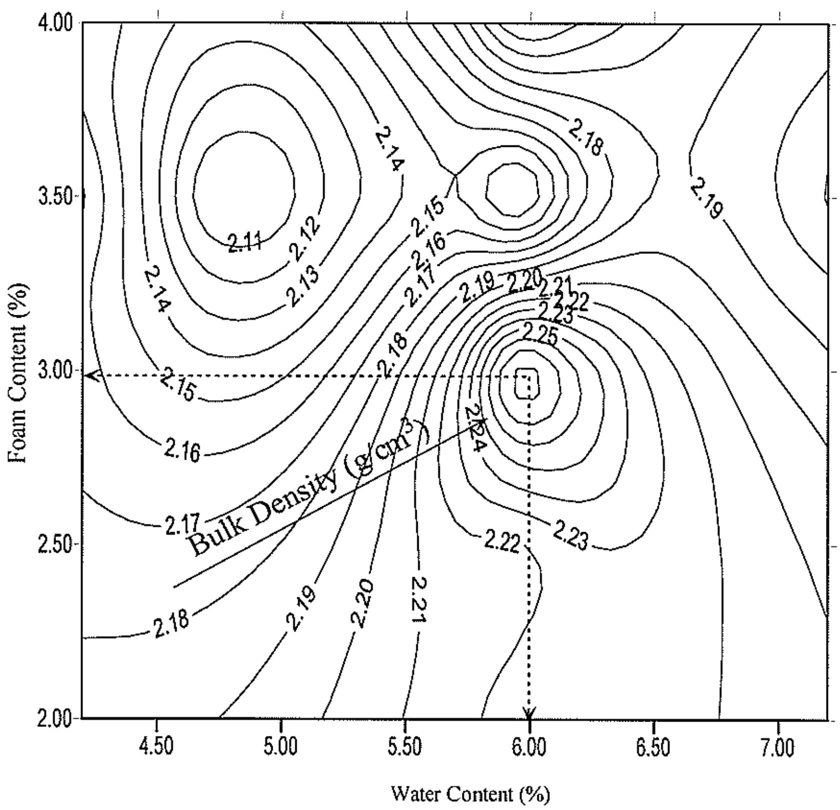

Figure 15.23 Relationship between foam content, water content and bulk density for group 1 specimens (fly ash and cement as filler).

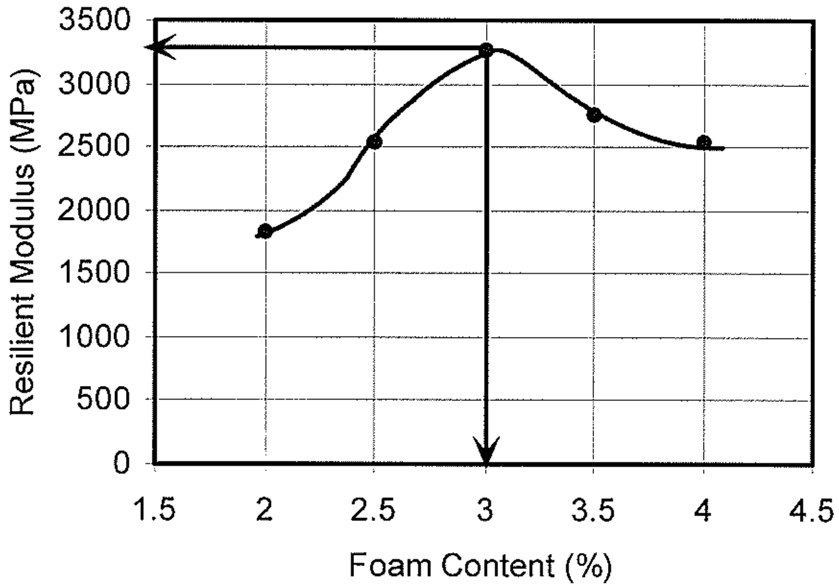

Figure 15.24 Relationship between resilient modulus and foam content at 6.0% water content, after 7 days curing at room temperature, for group 2 specimens.

15.9 Effect of Bitumen Source on Resilient Modulus

To study the effect of source and grade on the characteristics of the foam mixes, four replicates were prepared using the same aggregate gradation as for Sources I & II specimens but with 1.0% cement as a partial replacement to the fly ash. All specimens were mixed with 6.0% water and foam at 3.5% and compacted by the gyratory compactor for the same compaction effort as before. Note that these values for foam and water contents are not the optimum values for these mixes and an approach similar to the one discussed in Section 15.2 of this report, should be used if the maximum mechanical properties are needed.

The resilient modulus was measured at three different curing times, one day, three days, and one week (168 h), at room temperature (about 19°C). Comparing the results in Table 15.4 and Figure 15.27, Source I provides higher resilient modulus values than Source II, although the foamability characteristics of the foam produced from Source I are inferior to those of Source II (Table 15.3). These results cast doubt on the usefulness of the current system of characterising foam quality, and designing a new characterisation technique is considered imperative in order to better classify the quality of the foamed bitumen. Currently, the research team is investigating a new system which will be discussed in the second phase of this project.

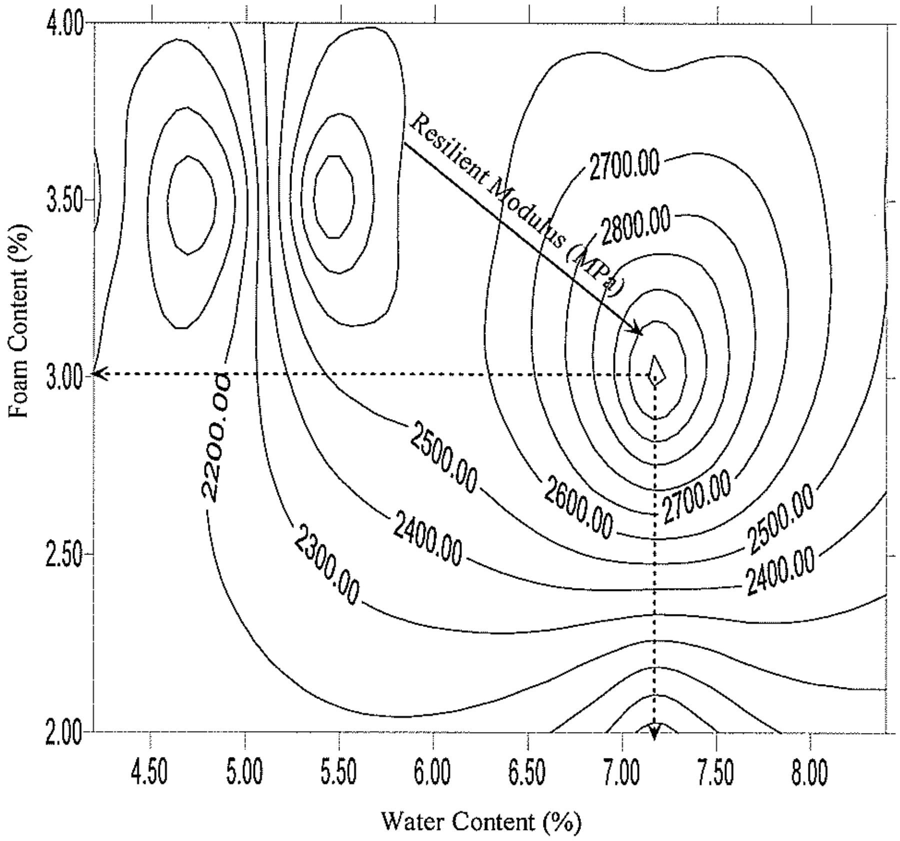

Figure 15.25 **Relationship between foam content, water content and resilient modulus, after 7 days curing at room temperature, for group 2 specimens (fly ash as filler).**

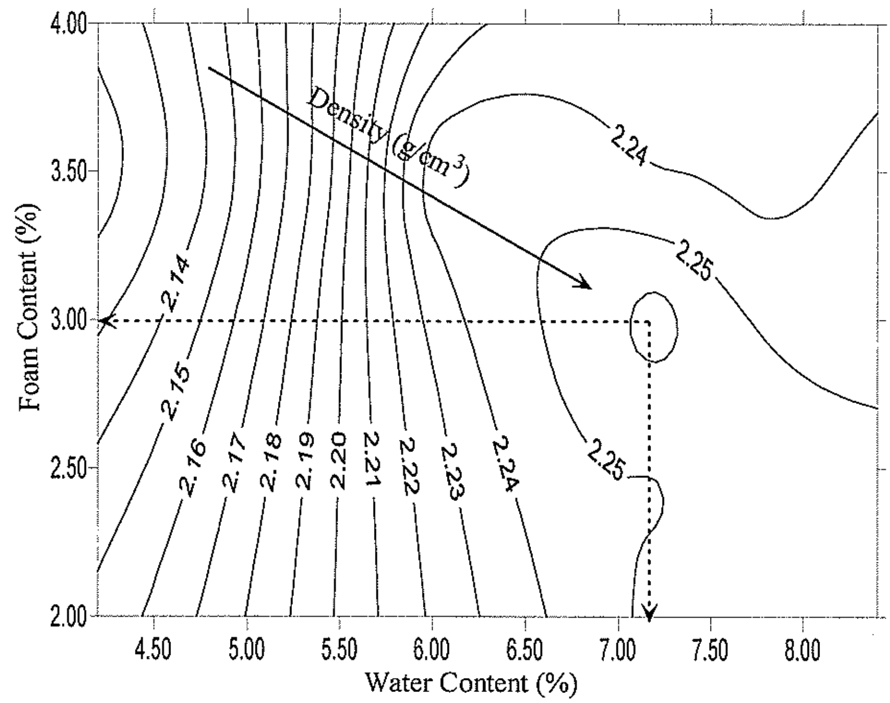

Figure 15.26 **Relationship between foam content, water content and density for group 2 specimens (fly ash as filler).**

Also, Table 15.4 and Figure 15.27 show that, within each source of bitumen, the softer 180/200 grade provided better results regarding the resilient modulus.

Table 15.4 Resilient modulus (MPa) for different sources, grades and curing times.

Curing Time (hours)	Source I		Source II	
	80/100	180/200	80/100	180/200
24	–	909	385	688
72	1956	2884	1365	1575
168	4414	4830	2727	3739

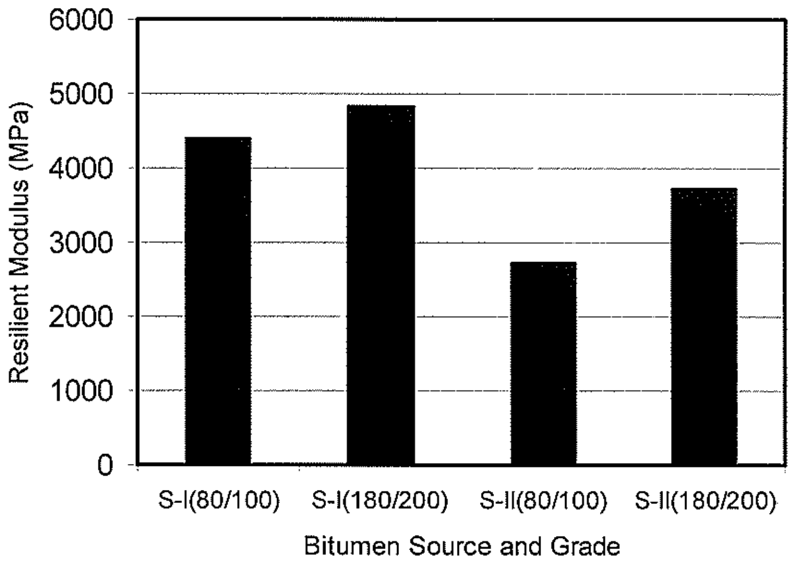

Figure 15.27 Effect of bitumen source and grade on resilient moduli (after one week or 168 h curing at room temperature).

16. Economic Evaluation of Stabilisation Alternatives

The cost effectiveness of foamed bitumen stabilisation in New Zealand compared to other stabilisation techniques will depend, to some degree, on specific projects. In general terms, binder costs are typically half of the total project costs (Australian Stabilisation Industry Association (ASIA) 2000). Given that the cost of bitumen is typically 3 to 4 times that of cement or lime, then on that basis (assuming comparable lifetimes) lime or cement stabilisation would be favoured.

However, one of the key advantages of foamed bitumen stabilisation, in contrast to lime- or cement-stabilised pavements, is that they can be trafficked almost immediately after compaction. On highly trafficked urban roads and motorways, the requirement to minimise traffic delays governs the construction process, and prolonged road closures (with the associated costs to road users) cannot be tolerated. On highly trafficked roads, the cost of traffic management is about 30% of the total capital cost of the project. In these instances, the use of cement or lime stabilisation could be disadvantageous.

In addition to the reduction in traffic delay, foamed bitumen stabilisation is likely to become more common in the future as sources of good quality aggregates (particularly in the Auckland area) become scarcer and attention turns to the use of more marginal materials.

Both cement and lime can contaminate the ground water by raising the pH value, and this is a problem in urban areas. Contrary to lime and cement, foamed bitumen is environmentally friendly and it does not cause contamination to the surrounding area.

Furthermore, comparing foamed bitumen to cutbacks or even emulsions, foamed bitumen is a clean alternative and does not cause leakage of kerosene or emulsifying agents into the surrounding environment. Such environmental advantages are vital but hard to quantify in the economic analysis.

To perform an economic analysis to compare different stabilisation methods including lime, cement, and foamed bitumen stabilisation, the resilient moduli results from previous research conducted at the University of Canterbury were utilised (Byers et al. 2003). Table 16.1 shows the resilient moduli for the nine stabilisation techniques. For comparative purposes, data for the foam-stabilised greywacke obtained during the present work are presented in Table 16.1. In this analysis, the Multi-Layer Linear Elastic computer program CIRCLY was used for the pavement design (Wardle 1977). The Austroads mechanistic pavement design method was used in this pavement design exercise, in which the subgrade CBR was assumed to be 5.0% (i.e. $M_r = 50$ MPa).

Table 16.1 Typical resilient moduli for various stabilisation methods.

Mix	Average M_r (MPa)
Slag	420
Basalt	340
Greywacke	380
1.5% Emulsion	337
2% Lime	484
4% Lime	600
2% Cement	900
*Greywacke stabilised with Foam Bitumen	9000
*Hot Mix Asphalt	3000

from Byers et al. 2003;
* denotes current research by authors of this report.

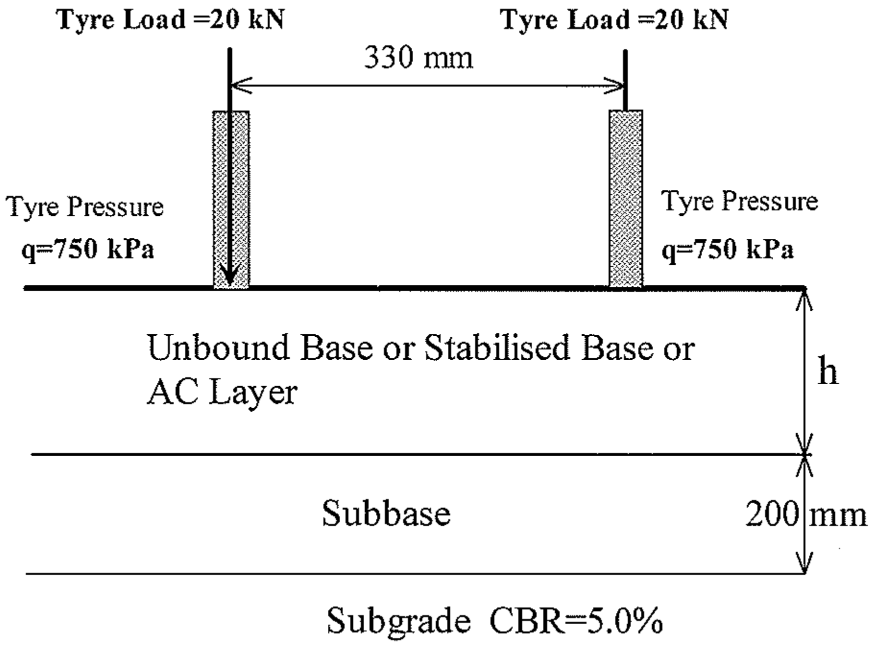

Figure 16.1 Pavement thickness and applied loads used in the pavement design method for comparing different stabilisation methods.

Figure 16.1 shows dual tyres of the standard axle load (80 kN), each tyre carrying a 20 kN load with a tyre pressure of 750 kPa, and a distance between the two tyres of 330 mm. The ESAL (equivalent single axle loads) during the design period were assumed to be $5\text{x}10^6$ ESAL. In all the design alternatives, a sub-base course of marginal materials with a resilient modulus of value 200 MPa was used. The thickness of the pavement structure, h, was calculated according to the Austroads design method using each of the materials listed in Table 16.1. Table 16.2 shows the required thickness of the pavement structure. The transfer functions used in the analysis are shown below (Austroads Design Guide 2002).

Fatigue Life Transfer Functions for Asphalt and Foamed-Stabilised Mixes

$$N_f = \left[\frac{6918*(0.856*V_b+1.08)}{S_{mix}^{0.36}*\mu\varepsilon}\right]^5 \tag{19}$$

where:

V_b = Percentage of bitumen by volume, and can be determined from this equation

$$\%V_b = \frac{\%P_b * G_{mb}}{G_b} \tag{20}$$

where:

P_b = Percentage of bitumen by total weight of mix
G_{mb} = Bulk Specific Gravity of the compacted mix
G_b = Specific gravity of bitumen
S_{mix} = Stiffness of the mix (MPa)
$\mu\varepsilon$ = Tensile strain at the bottom of the HMA (microstrains)

Permanent Deformation of Subgrade

$$N_f = \left[\frac{8511}{\mu\varepsilon}\right]^{7.14} \ldots \tag{21}$$

These assumptions have been made in the analysis:

- P_b for hot mix asphalt design is 5.5% and G_{mb} = 2.3, therefore, V_b = 12.4%
- P_b for Foam Bitumen was assumed 3.5% and G_{mb} = 2.3, therefore, V_b = 7.9%

Regarding the cost analysis, these assumptions have been made based on prices given by local contractors in both Christchurch and Auckland.

- The cost of HMA at 100 mm depth or more and an area of 1000 m^2 (within 10 km from the mixing plant and assuming no traffic control), is $135.00/tonne including supply, laydown and compaction.
- For unbound AP-20 material at 100 mm depth and an area of 1000 m^2 (within 10 km from the depot and assuming no traffic control), the cost is $40.00/tonne including supply, laydown and compaction.
- For lime-stabilised AP-20 (same details as above), the cost is $45.00/tonne for 2% lime and $47 for 4% lime, including supply, laydown and compaction.
- For cement-stabilised AP-20 (same details as above), the cost is $52.00/tonne including supply, laydown and compaction.
- For foamed bitumen-stabilised AP-20 or recycled asphalt pavement (RAP), the price is $55/tonne including allowance for addition of up to 10% virgin aggregate, fines correction, up to 1% cement, 3.5% foam, and surface preparation for surfacing.
- For marginal aggregates stabilised with about 3.5%, the cost is $50.00/tonne including transportation, laydown and compaction.
- If chipseal surfacing is to be used the cost is $4.00/m^2. The cost for chipsealing a one lane-km is $14,000. This value is added to all alternative costs except the HMA alternative.

- For the sub-base materials, the cost per is $20 including transportation and laydown and compaction. The cost of the sub-base construction is $23,800 per lane-km. This value is added to all alternative costs.
- In the analysis the lane width was assumed to be 3.5 m.

The above prices do not include GST.

Table 16.2 compares the properties and economics of the various methods. Note that the analysis undertaken at this stage is based on the capital cost because of the lack of data about the performance of each method. However, if maintenance and user costs during the life of each method were considered, this would significantly impact the analysis.

The first six methods were designed based on rutting, where the maximum compressive strain on the top of the subgrade layer was calculated and the transfer function given by Equation 21 was used to determine the rutting life of the pavement. However, for foamed bitumen-stabilised mixes and asphaltic concrete materials, the thickness design was based on the fatigue of the asphalt layer where the tensile strain at the bottom of the asphalt layer was calculated and the transfer function given by Equation 19 was used to determine the fatigue life. The assumption that Equation 19 is valid for foamed bitumen-stabilised materials was considered reasonable, however, a full fatigue life study of these materials will form a later phase of the project. In addition to fatigue, the compressive strain at the top of subgrade layer was used to calculate the rutting of the pavement.

In Table 16.2, two methods are given for the foamed bitumen method. The first method has a modulus of 6000 MPa which represents a good quality basecourse modified with the foamed bitumen, and contains about 2.0% cement. Note that 6000 MPa is a conservative estimate for the modulus since values greater than 9000 MPa were obtained in the laboratory. The alternative method (3000 MPa) represents a lower quality basecourse modified with foamed bitumen and about 1.0% cement.

The maximum surface deflection was determined for each alternative as an indicator of the expected performance for each design.

By comparing the cost of the different methods, as depicted in Figure 16.2, the scenario with a foamed bitumen-stabilised basecourse of modulus 6000 MPa is clearly a cost-competitive alternative. Even the lower quality basecourse stabilised with the foamed bitumen looks very competitive to the other stabilisation methods. Also expected is that the cost of foamed bitumen might decrease considerably as more experience is gained in this area, and when more contractors start to adopt this technique.

Also clear is that the maximum surface deflection expected in the foamed-stabilised mixes is less than 50% of that developed with the unbound base, lime stabilisation or cement stabilisation methods, and even less than that developed in the hot mix asphalt. Thus, an excellent performance is expected from using foamed-stabilised mixes and as a result less maintenance costs and lower user delays are expected. Also, this technique is very competitive to hot mix asphalts where a very high performance is expected with its much lower cost.

Table 16.2 Pavement thickness design (costs are $NZ excluding GST).

Design Alternative	Mix	M_r (MPa)	Thickness h (mm)	Max. Deflection (µm)	Quantity per lane - km (m^3)	Unit Weight (t/m^3)	Quantity in per lane – km (tonnes)	Cost per tonne ($NZ)	Cost per lane-km including sub-base course and chipseal surfacing ($NZ)
1	**Slag**	420	225	943	787.5	2.0	1575	40	100,800
2	**Basalt**	340	235	1003	822.5	2.0	1645	40	103,600
3	**Greywacke**	380	230	967	805	2.0	1610	40	102,200
4	**2% Lime**	484	215	914	752.5	2.0	1505	45	105,525
5	**4% Lime**	599	205	873	717.5	2.0	1435	47	105,245
6	**2% Cement**	898	185	821	647.5	2.3	1489	52	115,241
7	**Foam Stabilised**	6000	205	434	717.5	2.3	1650	55	128,564
8	**Foam Stabilised**	3000	240	468	840	2.3	1932	50	134,400
9	**HMA**	3000	165	603	577.5	2.3	1328	135	203,114

In addition to its cost competitiveness, foamed bitumen stabilisation is environmentally friendly with lower emissions produced from the mixing plants than HMA. Moreover, about 25% reduction in energy is reported by previous researchers compared to that achieved in HMA production, because cold aggregates are used.

Preliminary testing on moisture susceptibility showed a high resistance of foamed bitumen mixes to water damage where an index of retained strength of more than 80% was achieved. These mixes were able to keep their structural integrity without significant loss of strength even after soaking in water for four days. The results of moisture susceptibility will be thoroughly discussed in the second phase of this study. Therefore, an excellent performance is expected in areas where high precipitation or a high water table is likely.

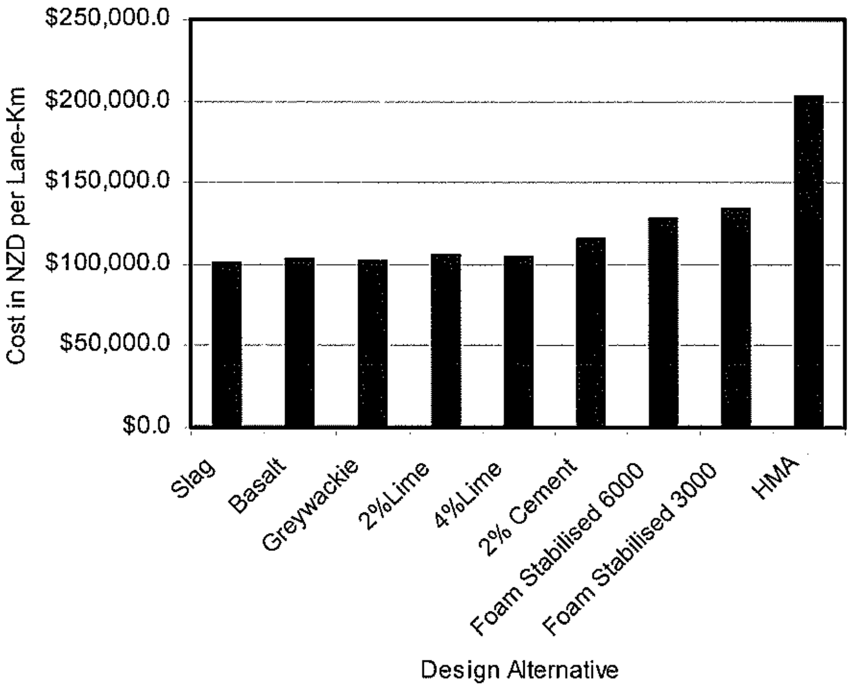

Figure 16.2 Comparison of costs for different stabilisation methods.

17. Conclusions & Recommendations

A review of the literature reveals that foamed bitumen stabilisation is a mature technique gaining wide acceptance in the highway community. However, some aspects of the process, and properties of the resulting mixes, have received relatively little attention. Further research is warranted to facilitate possible improvements to the method and to allow proper comparisons between different techniques.

In the experimental section of this study, two grades of bitumen (180/200 and 80/100) from two sources were used to investigate the feasibility of foam stabilisation in New Zealand. The foaming characteristics of the bitumens and the properties of the stabilised mixes were measured.

The foamability of bitumen obtained from the two different sources differed according to the current system of characterisation which is based on expansion ratio, half-life, and foam index parameters. Also, within each source, softer grades obviously provided better foamability results and better mechanical properties for the foam mixes. However, although Source I bitumen showed inferior properties compared to Source II in terms of its foamability properties, the mechanical properties of the mixes prepared from Source I were better than those obtained from Source II. This indicates that the current system of characterising foam properties may be inadequate and more research in this area is needed.

The resilient modulus tests showed that foamed bitumen-stabilised mixes have high stiffness values when using an AP-20 aggregate gradation (with additional fly ash, or fly ash and cement, as fillers). Moduli increased with increasing curing time at room temperature to values comparable to, or in excess of, that of HMA. Even after only 24 hours the moduli obtained were sufficient to sustain traffic loads without significant problems.

A cost-analysis exercise comparing the cost of nine different stabilisation methods was made. The results of this analysis showed that foamed bitumen stabilisation using high quality aggregates and about 2% cement is very competitive for unbound materials because a reduced thickness of seal is required. Because quantitative data are lacking, environmental benefits, road user effects such as vehicle operating costs, or maintenance costs arising from various stabilisation methods, could not be estimated at this time. From the literature reviewed such factors seem very likely to increase the competitiveness of the foamed bitumen stabilisation method relative to other methods. Because of the limited information about constitutive models and transfer functions of the foamed bitumen-stabilised mixes, the analysis was carried out assuming that the fatigue behaviour of the foamed bitumen material is similar to that of HMA.

Based on the literature discussed in Part I of this report and the results of the feasibility study reported in Part II, foamed bitumen stabilisation appears to be a very cost-competitive alternative to those methods traditionally used in New Zealand. The technique appears to offer significant advantages in terms of speed of construction and disruption to road users during construction.

In the second phase of this study the proposal is to study the properties of foamed bitumen stabilisation in more detail. Particular emphasis will be placed on characterising aspects of the method that are especially relevant to its applications in New Zealand. This work will include investigation of:

- A wider range of different types of bitumen to achieve a better understanding of the variation in foam and foaming properties likely to be experienced in practice.
- A new characterisation system for bitumen foam properties.
- Effect of foaming agents on the properties of foamed bitumen.
- The temperature susceptibility characteristics of foamed bitumen mixes and the likely effects of foam temperature on mixing efficiency.
- The effect of using low penetration bitumens on mix properties.
- The temperature susceptibility of foamed bitumen mixes in comparison to HMA.
- The stress dependence of foamed bitumen mixes and the effect of confining stresses.
- The effect of different aggregate types (selected from around New Zealand) and gradations on mix properties.
- Moisture susceptibility of foamed bitumen-stabilised mixes.
- Volumetric properties of foamed bitumen-stabilised mixes and their effects on performance.
- The fatigue properties of foamed bitumen-stabilised mixes in comparison to those of other stabilisation techniques.

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Appendix

Table A1. Resilient modulus values at different water contents (Wc) and curing times for the first group of mixes (contains 2% cement and fly ash as filler) and 3.5% foam content.

Wc	Density (g/cm^3)	Curing time (hours)							
		24	48	72	96	120	144	168	Completely Dry
		Resilient modulus values (MPa)							
4.2	2.170	393	809.5	1147	1604.5	1789	1988.5	2411.5	NA
4.8	2.090	384.5	978	1569.5	1970.5	2211	2424.5	2851	NA
5.4	2.135	631	903	1945.5	2809.5	3190	3622	3484.5	NA
6.0	2.120	536	1313.5	2831	3709.5	4386	4858.5	4955.5	NA
7.2	2.170	395	1029.5	2603.5	3624.5	4291	4698.5	5240	NA

NA not applicable

Table A2. Resilient modulus values at different foam contents (Fc) different curing times for the first group of mixes (contains 2% cement and fly ash as filler) and 6.0% water content.

Fc	Density (g/cm^3)	Curing time (hours)							
		24	48	72	96	120	144	168	Completely Dry
		Resilient modulus values (MPa)							
2.0	2.28	752.5	1103.5	2089	2843.5	4135	4036.5	4828	NA
2.5	2.28	712.5	1588	3117.5	3940	4661.5	6034.5	6608.5	NA
3.0	2.25	837	1838.5	4328.5	5713	6988	8799.5	9315.5	NA
3.5	2.30	692	1406.5	2725.5	3552	5005.5	5530.5	6576.5	NA
4.0	2.275	719	1402.5	2931	3859	4674	5165.5	5252.5	NA

Table A3. Resilient modulus values at different water (Wc) contents and different curing times for the second group of mixes (contains only fly ash as filler) and 3.5% foam content.

Wc	Density (g/cm^3)	Curing time (hours)							
		24	48	72	96	120	144	168	Completely Dry
		Resilient modulus values (MPa)							
4.2	2.160	643	813	1331	1500	2065	1896	2326	1999
4.8	2.200	544	729	1063	1149	1613	1756	1857	1985
5.4	2.245	973	1127	1651	2057	2525	2827	2896	3538
6.0	2.215	944	1034	1342	1754	2110	2368	2479	2909
7.2	2.350	1204	1439	1890	2343	3113	3333	3543	4117
7.8	2.315	756		1530				2684	3373
8.4	2.315	744		1305				2496	3719

Table A4. Resilient modulus values at different foam contents (Fc) and different curing times for the second group of mixes (contains only fly ash as filler) and 6.0% water content.

Fc	Density (g/cm^3)	Curing time (hours)							
		24	48	72	96	120	144	168	Completely Dry
		Resilient modulus values (MPa)							
2.0	2.320	385.5		918				1837.5	2423.0
2.5	2.280	556.5		1197				2534.5	3263.5
3.0	2.295	871.5		1779.5				3259.5	4106.5
3.5	2.260	817.5		1431				2750.5	3263.5
4.0	2.260	805.5		1352.5				2546.5	3119.5