FOAMED BITUMEN STABILISATION IN NEW ZEALAND – A PERFORMANCE REVIEW AND COMPARISON WITH AUSTRALIAN AND SOUTH AFRICAN DESIGN PHILOSOPHY

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ABSTRACT

When foamed bitumen was introduced to New Zealand in 2004, overseas guidelines such as the South Africa Asphalt Academy TG2 and the Wirtgen Cold Recycling Manual formed the basis of developing best practice. While seven years is a relatively short period when compared to typical 25 to 30 year design life, the performance of these Foamed Bitumen Stabilised (FBS) pavements has been exemplary. Quality assurance and post construction evaluation to date suggests that design expectations are being comfortably met and maintained.

Throughout the last few years New Zealand stabilisation specifications, design guides and technical notes have been developed and adopted. It is expected that these documents will continue to be refined to reflect the extensive ongoing performance monitoring and research. Comparison with overseas philosophy and specifications has demonstrated some distinct differences for FBS. South Africa have recently adopted the Knowledge Based (Empirical) design method, while the Australian approach recommends mechanistic design using asphalt fatigue criteria. This results in markedly different thickness and material requirements for FBS design depending on which country in which it is evaluated.

This paper will expand on current FBS design guidelines and performance in New Zealand and evaluate key points of difference with current overseas design philosophy.

INTRODUCTION

The last seven years has seen a significant quantity of Foamed Bitumen Stabilisation (FBS) undertaken in New Zealand, typically for rehabilitated pavements, but also for greenfields projects and maintenance. For a typical design life of 25 to 30 years a high level of confidence is required that failure will not occur later in pavement life. Research, quality assurance and post construction evaluation to date suggests, at the least, a continued achievement of design expectations.

Extensive research has been undertaken in New Zealand on testing protocols, refining mix designs, curing/hydration times and sensitivity to different types and/or proportions of reagents to laboratory failure mode(s). A wide variety of materials and treatment constraints have also been encountered and mitigated with some interesting outcomes. As a result of this experience, valuable lessons have been learnt regarding the reliability of FBS testing, laboratory representation of field performance, mix optimization for pavement requirements and materials sensitivity/rate of strength gain.

When compared to conventional granular pavements, the benefits of FBS for New Zealand modeling such as eradication of sublayering and higher resilient modulus, can be attractive to the designer – especially where undercuts can be avoided and existing pavement materials recycled. This is particularly so in cases where the finished level is constrained, precluding overlay, a scenario commonly associated with urban rehabilitations. This can offset the increased materials cost incurred by the incorporation of bitumen. It is important, however, that these ‘benefits’ are achieved through dependable performance.
This paper will expand on developments and experience gained through foamed bitumen research and current foamed bitumen design guidelines and performance to date. This paper will also evaluate key points of difference with overseas design philosophies. A comparative modeling exercise using the same base pre-treatment profile demonstrates a significant increase in FBS thickness required for Australian practice, and to a lesser degree South African practice, relative to that required by the recommended New Zealand design process.

**FBS IN NEW ZEALAND/AUSTRALIA/SOUTH AFRICA**

**Background**

When foamed bitumen was introduced to New Zealand, overseas guidelines such as the South African Asphalt Academy *Interim Technical Guideline: The Design and Use of Foamed Bitumen Treated Materials* TG2 (September 2002) and the Wirtgen Cold Recycling Manual (2nd Edition November 2004) formed the basis for developing best practice. However, the FBS response to New Zealand’s unique materials (geologically young, heterogeneous and marginal quality when compared to overseas aggregates) and construction constraints (narrow pavements with little or no opportunity for detour or contra-flow closure) FBS design has been refined to suit ‘local conditions’.

Through the last few years, New Zealand stabilisation specifications, design guides and technical notes have been developed by cross-party (Client, Consultant, Contractor and Supplier) working groups which have been adopted for industry utilisation. At the same time, TG2 has been significantly revised with the release of the TG2 Technical Guideline: Bitumen Stabilised Materials (Second Edition May 2009), while the Wirtgen Cold Recycling Technology (3rd Edition 2010) has superseded the Wirtgen Cold Recycling Manual.

New Zealand and Australia both ‘operate’ under the auspices of the Austroads Guide to Pavement Technology, yet both countries are developing National Supplements (New Zealand) or Interim Design Procedures (Australia) which will be expanded on later in this paper. Evaluation of the Austroads *Review of Foamed Bitumen Stabilisation Mix Design Methods* (December 2010) has also demonstrated some distinct differences in methodology and philosophy.

**Australia, South Africa & New Zealand key distinctions**

It is difficult to document a current comparative evaluation due to the ongoing changes in approach for all three countries. However, it is clear that there are major differences in design philosophy and modeling convention. This environment of refinement and change is a function of striving for best practice and recognition that further improvements are required to provide a laboratory testing and design process that closely matches observed field performance.

Capturing some of the current key distinctions between New Zealand, South Africa and Australian foamed bitumen stabilisation design and construction (at the time of writing this paper) reveals the following (refer Table 1).

<table>
<thead>
<tr>
<th>Element</th>
<th>New Zealand specification</th>
<th>Australian specification</th>
<th>South African specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design philosophy</td>
<td>Equivalent granular state (phase 2) mechanistic design</td>
<td>Effective fatigue phase – Austroads asphalt criteria (phase 1)</td>
<td>Knowledge based method: pavement number empirical OR mechanistic design</td>
</tr>
<tr>
<td>Expansion/half life requirement</td>
<td>Minimum of: 10 times expansion &amp; 6 seconds half life</td>
<td>Minimum of: 15 times expansion &amp; 30 seconds half life</td>
<td>Min. of: 8 times expansion (for 10 – 25 °C) &amp; 6 sec. half life</td>
</tr>
</tbody>
</table>

Table 1: New Zealand, Australia and South Africa key distinctions of FBS practice
<table>
<thead>
<tr>
<th>Element</th>
<th>New Zealand specification</th>
<th>Australian specification</th>
<th>South African specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foaming agent</td>
<td>Not used for construction. Some labs use for mix design</td>
<td>Teric 311 foaming agent for design and construction</td>
<td>Not used</td>
</tr>
<tr>
<td>Percentage by mass of active filler</td>
<td>≤ 1.5% cement (Lime Oxide or KOBM for pretreatment)</td>
<td>≤ 2.5% Hydrated Lime (Hydrated Lime for pretreatment)</td>
<td>≤ 1% Cement or lime (lime or other active filler for pretreatment)</td>
</tr>
<tr>
<td>Tensile test loading rate</td>
<td>1 mm/minute recently proposed for ITS. Prev 50.8 mm/min(^2) to Wirtgen A2.4.1</td>
<td>3000 ms test pulse with 40 ms rise time for ITSM testing (to AS2891.13.1)</td>
<td>50.8 mm/min for ITS testing. (ITS only used for low volume roads)</td>
</tr>
<tr>
<td>Base aggregate</td>
<td>Single specification. Focus on grading and plasticity</td>
<td>Single specification. Focus on grading and plasticity</td>
<td>BSM1 (high strength) &amp; BSM2 (med strength)</td>
</tr>
<tr>
<td>Characteristic design modulus</td>
<td>800 MPa soaked (phase 2)</td>
<td>3000 to 4000 MPa Dry &amp; 1800-2000 soaked</td>
<td>BSM1 600 MPa max BSM2 450 MPa max(^3) Linear Elastic may be 600 to 1200 MPa(^4)</td>
</tr>
<tr>
<td>Initial modulus /preseal stiffness</td>
<td>Not specified. Able to be trafficked without rut. Clegg Impact Value 45° may be checked</td>
<td>700 MPa (3 hours curing)(^5). Ball Embedment testing may be undertaken</td>
<td>Not specified. Upper 100 mm of BSM layer to reduce to 50% OMC prior to sealing</td>
</tr>
<tr>
<td>Rut resistance</td>
<td>Repeat Load Triaxial (NZTA T/15) ≤1.0 mm rutting (ideally 0.5 mm) /million ESA</td>
<td>Max rut depth at 2000 Cycles to be 5-7 mm</td>
<td>Cat A – 10 mm rut at end of service life. Maximize density &amp; minimise moisture if early loading</td>
</tr>
<tr>
<td>Characteristic bitumen content (by mass)</td>
<td>2.7% to 3.5% Typically 2.7% to 3%</td>
<td>Typically 3.0% to 4.0%</td>
<td>Typically 1.7% to 2.5% (lower binder for high RAP(^6) mixes)</td>
</tr>
</tbody>
</table>

Notes:
1 There is no reference to the use of foaming agent in TG2 2009. Material temperature is limited to 15 °C before treatment to avoid the use of foaming agents.
2 Some New Zealand laboratories utilize 50.8 mm/min for Indirect Tensile Stiffness (ITS) testing. Specified loading rate is currently under review.
3 Empirical values for PN determination for roads with structural capacity <10 million ESA.
4 A resilient modulus of 400 to 900 MPa for bitumen treated graded crushed stone (GCS) material is referenced for mechanistic design. For more than 10 MESA, linear elastic modelling is typically used to determine critical parameters (i.e. stress ratio for BSM’s).
5 Minimum ‘early’ stiffness values are targeted to minimise the risk of early rutting.
6 Recycled Asphalt Product (RAP) is typically coarse and therefore typical foamed bitumen application rate reduces to the range of 1.8% to 2.2%.

MODELING PHILOSOPHIES FOR FOAMED BITUMEN

The basic premise of pavement design is to generate a profile of material layers that both protect the subgrade from strains that will induce permanent deformation, and to generate independent pavement layer properties that provide appropriate stiffness and durability for the position in the pavement system. Foamed bitumen is typically utilised for the basecourse (or upper pavement) layer – which is typically protected from traffic stresses only by a sprayed seal and/or thin asphalt surfacing.

In New Zealand and South Africa where foamed bitumen stabilised basecourse is modeled as a modified granular material (i.e. flexible and non-continuously bound), it is critical that the foamed bitumen basecourse can accommodate design loads without rutting and shoving due to
inadequate shear resistance, or deformation through shear and densification – and finally disintegration through breakdown or unraveling.

New Zealand modelling of foamed bitumen

The New Zealand modeling approach utilizes the Austroads Guide to Pavement Technology principles. However, guidance for the recommended approach to designing FBS materials is provided by the New Zealand Transport Agency (NZTA) Supplement to Austroads (2007). The FBS basecourse is not modeled with a performance criteria (as per asphalt, bound materials or subgrades), but rather is modeled as a modified granular material with unique parameters. The phase one elastic modulus is not typically used for design (although NZTA note the possibility of Austroads hot mix asphalt performance criterion for this phase), rather the phase two (steady state) elastic modulus or ‘equivalent granular state’ is instead utilised for modeling as follows:

- Elastic (or Resilient) Modulus $E = 800 \text{ MPa}$
- Poisson’s Ratio $= 0.3$
- Anisotropic Layer $(H = 0.5V)$ – but no sublayering.

Also noted is “Care should be taken to ensure that cracking is not a primary mode of failure by limiting the application of cementitious additives”.

The assumption of no sub-layering for the FBS layer could be considered unconservative. This is subject to limiting the modular ratio of the FBS layer to no more than 5 x modulus of the underlying layer (note: Austroads recommend 2 x modulus for typical unbound materials). On this basis, an underlying granular subbase layer with an elastic modulus of at least 175 MPa is required to permit the FBS layer to be modeled with no sub-layering and comply with modular ratio recommendations. Designers sometimes elect to divide the FBS layer into two sub-layers with a 400 MPa base sublayer where subbase modular ratio is marginal or inadequate.

This modeling approach results in a ‘thin’ pavement when compared to South African and Australian pavement design for a similar system. However, research and as-built pavement monitoring to date suggests that where an adequate substrate / modular ratio is achieved, and cement content is limited, this approach is providing consistently robust performance.

Australian modelling of foamed bitumen

In Australia, foamed bitumen has properties relatively close to that of asphalt and is modeled as a continuously bound layer (using Austroads Guide to Pavement Technology principles) subject to the common modes of distress for asphalt layers such as rutting and/or shoving due to insufficient resistance to pavement deformation, and cracking due to fatigue.

The recent Austroads Report TT1358 (Gonzalez 2011) evaluates a number of different design procedures from Australia, New Zealand, South Africa and the United Kingdom. The recommendation is that the proposed interim design procedure outlined in the report be adopted for design alongside the Austroads Guide to Pavement Technology – Part 2 (Austroads 2008a). On this basis FBS is to be designed as a bound pavement layer with asphalt fatigue criteria.

The characteristic design modulus of 3,000 to 4,000 MPa (dry) and 1,800 to 2,000 MPa (soaked) is significantly higher than that typically used / specified in New Zealand and South Africa. This resilient modulus is used with the Austroads asphalt fatigue criterion (Eqn 1) thus:

$$N = RF \left[ \frac{6918(0.856V_b + 1.08)}{S_{mix}^{0.36} \mu} \right]^5$$

where
\[ N = \text{allowable number of repetitions of the load} \]
\[ \mu_{\varepsilon} = \text{tensile strain produced by the load (microstrain)} \]
\[ V_b = \text{percentage by volume of bitumen in the asphalt (\%)} \]
\[ S_{\text{mix}} = \text{asphalt resilient modulus (MPa)} \]
\[ RF = \text{reliability factor for asphalt fatigue}. \]

With this high characteristic design modulus and specified properties the requirement for high bitumen content and the utilisation of foaming agents is likely to be sustained.

**South African modelling of foamed bitumen**

The South African modeling approach was traditionally based upon the Interim Technical Guidelines (TG2 2002) which classified the treated pavement materials and modelled on a 2-phase mechanistic – empirical structural design. The primary effective fatigue phase evaluated the horizontal tensile strain at the bottom of the layer against dependable flexural modulus of the material from flexural beam testing – similar to modeling asphalt. The secondary equivalent granular phase commences when the applied loading reduces the layer stiffness to that of a ‘good quality’ unbound granular material and permanent deformation becomes the critical response parameter.

While mechanistic modeling is still undertaken, the revised TG2 (Second Edition May 2009) now recommends a “Pavement Number Structural Design Method” approach where the design traffic is more than 1 million equivalent standard axles (ESA). The bitumen stabilised material’s (BSM’s) are categorised according to their dependable properties and integrated into a pavement system that employs a controlled increase in strength through subsequent layers and properties that are based on observed pavement behavior and performance.

Some key elements of this design method are:

- Empirical effective long term stiffness (ELTS) properties for BSM layer modulus are limited to 600 MPa (BSM1) or 450 MPa (BSM2) rather than tested properties and are for use in the pavement number (PN) design method – and not for modelling or analytical use. This averages out the effects of seasonal / long term stiffness reduction rather than specifically defined ‘current’ properties.
- Modular ratio limit is 3.0 (for BSM1) or 2 (BSM2). Other layers also have modular ratio limits depending on the material classification.
- Limited to design traffic of more than 30 million ESA, layers with lenses of weaker material or low shear strength interfaces, or a subgrade CBR of less than 3%.
- Note that the data used for the ‘frontier curve’ still exhibits no failures and this suggests the upper limit may be closer to 40 million ESA.

Where the pavement structural capacity is more than 10 million ESA mechanistic modeling may be used with primary material input modulus of 800 to 1200 MPa for a good quality material. The deviator stress ratio is the critical indicator for performance with transfer function based on repeated load triaxial testing (ref Wirtgen Cold Recycling Manual 2010).

**Impact of modelling philosophies**

The differences in modeling philosophy between the three countries is significant, and this will have a profound impact on where FBS is deemed to be appropriate for a given pavement.
system or rehabilitation, where it is specified, what construction costs are and how this base layer interacts with the pavement system it becomes an integral part of.

To provide some context for comparison, an identical medium level pavement structure requiring rehabilitation has been selected, and modeled in the following section, using each of the approaches outlined above to evaluate the resulting thickness of FBS basecourse required.

**COMPARISON OF MODELLED FOAMED BITUMEN PAVEMENT SYSTEMS**

Preparing comparative models for FBS basecourse in Australia, South Africa and New Zealand shows some significantly variable treatment depths due to the markedly different approaches and typical modelled parameters currently recommended.

The dependable pavement profile used for this exercise comprises a sprayed seal surfacing overlying 450 mm of aggregate (150 mm of aged basecourse over 290 mm of aged subbase) that will respond positively to FBS. The aggregate overlies a subgrade with dependable resilient modulus of 50 MPa. This profile is represented in Table 2.

**Table 2: Existing pavement structural properties**

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness (mm)</th>
<th>Existing vertical modulus (MPa)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seal surfacing</td>
<td>10</td>
<td>N/A</td>
<td>Grade 4 chipseal</td>
</tr>
<tr>
<td>Basecourse</td>
<td>150</td>
<td>300 MPa (top sublayer)</td>
<td>Aged 40 mm all-in basecourse</td>
</tr>
<tr>
<td>Subbase</td>
<td>290</td>
<td>210 MPa (top sub-layer)</td>
<td>Aged 65 mm all-in basecourse</td>
</tr>
<tr>
<td>Subgrade</td>
<td>Semi-infinite</td>
<td>50 MPa</td>
<td>Cohesive sandy clayey silt soils</td>
</tr>
</tbody>
</table>

This pavement system was evaluated for achieving a load capacity of 5 million in the first instance, and then subsequently for 10 million equivalent standard axles (ESA).

**Pavement systems for design traffic of 5 million ESA**

**New Zealand FBS design example**

This pavement configuration provides a critical damage factor (CDF) of 0.65 for the subgrade where less than 1.0 is required to meet design loadings without failure. Basecourse and subbase are not modeled for fatigue and merely provide a stress spreading mechanism for subgrade (unlike bound layers that employ a fatigue criteria) and rely on specifications for nominal properties.

**Table 3: New Zealand NZTA supplement to Austroads Design**

<table>
<thead>
<tr>
<th>Layer</th>
<th>Material</th>
<th>Thickness (mm)</th>
<th>Modulus (MPa)</th>
<th>Sub-layering?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surfacing</td>
<td>2-coat chip seal</td>
<td>10</td>
<td>0</td>
<td>N/A</td>
</tr>
<tr>
<td>Basecourse</td>
<td>Foamed bitumen</td>
<td>150</td>
<td>800</td>
<td>No</td>
</tr>
<tr>
<td>Subbase</td>
<td>Existing subbase</td>
<td>300</td>
<td>210</td>
<td>Yes</td>
</tr>
<tr>
<td>Subgrade</td>
<td>Sandy clayey silt</td>
<td>Semi-infinite</td>
<td>50</td>
<td>N/A</td>
</tr>
</tbody>
</table>

A traffic multiplier of 1.2 was applied to the subgrade, while the foamed bitumen layer was modeled with no sublayering (as the modular ratio was less than 5). The subbase modulus was constrained by the modular ratio of 2.0/125 mm of aggregate thickness overlying subgrade as...
required by Austroads (Section 8.2.3). The use of (Austroads integrated) mechanistic design software ‘Circly 5’ automatically constrains the subbase modulus to that permitted by Austroads modular ratio convention regardless of the layer property used for the model. The sprayed seal surfacing does not contribute to structural capacity.

**Australia FBS design example**

This pavement configuration utilising 320 mm FBS provides a critical damage factor (CDF) of 0.92 for the basecourse and negligible CDF for the subgrade (3.3E-04) where < 1.0 is required.

**Table 4: Australian interim design procedure for foamed bitumen pavements**

<table>
<thead>
<tr>
<th>Layer</th>
<th>Material</th>
<th>Thickness (mm)</th>
<th>Modulus (MPa)</th>
<th>Sub-layering?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surfacing</td>
<td>2-coat chip seal</td>
<td>10</td>
<td>0</td>
<td>N/A</td>
</tr>
<tr>
<td>Basecourse</td>
<td>Foamed bitumen</td>
<td>320</td>
<td>1960</td>
<td>No</td>
</tr>
<tr>
<td>Subbase</td>
<td>Untreated existing subbase</td>
<td>130</td>
<td>103</td>
<td>Yes (5 x 26 mm)</td>
</tr>
<tr>
<td>Subgrade</td>
<td>Sandy clayey silt</td>
<td>Semi-infinite</td>
<td>50</td>
<td>N/A</td>
</tr>
</tbody>
</table>

The subbase top sublayer resilient modulus is constrained to 103 MPa as per Austroads sublayering convention. A traffic multiplier of 1.2 was applied to the subgrade and the foamed bitumen layer was modeled with Queensland parameters for the interim design procedure with soaked laboratory indirect tensile resilient modulus of 2470 MPa adjusted with WMAPT of 32 °C, traffic speed of 90 km/hr to provide 1,960 MPa resilient modulus. The assumed volume of bitumen to derive fatigue criteria (as per Equation 1) was 7%.

The sprayed seal surfacing does not contribute to structural capacity. It is likely that the FBS basecourse would need to be treated and compacted in two layers significantly adding to time/costs. A 40 mm to 50 mm asphalt wearing course would reduce FBS base to a layer thickness of 240 to 270 mm which could be undertaken in a single layer.

**South Africa FBS design example**

The empirical PN method was used. However, with the existing pavement being relatively thin, a Category A type of pavement is not considered appropriate in terms of existing and rehabilitated structural capacity and a category B pavement was determined. A sprayed seal surfacing could not provide an adequate structural number without the use of 300 mm or more of BSM1 basecourse. An adequate pavement number (i.e. 17.9 or more) and corresponding capacity for 5 million ESA design loading was achieved by employing a BSM1 basecourse layer thickness of 175 mm with 25 mm thickness of asphalt surfacing. Note that the empirical PN method is considered conservative and mechanistic modelling will provide a ‘leaner’ structure.

**Table 5: South African TG2 (2009) Pavement Number Structural Design**

The BSM1 foamed bitumen base was constrained to an ELTS of 486 MPa due to permissible modular ratio for the underlying subbase and select fill. This modeling methodology is extremely sensitive to the subgrade ELTS which constrains all overlying pavement layer properties,
however, the asphalt surfacing adds significantly to design pavement number. The permissible loading of this pavement structure for a Category A pavement is significantly less, and significant structural improvements would be required to accommodate a Category A pavement for 5 million ESA via an increased BSM1 base and asphalt surfacing.

Summary of design examples

A summary of the relative pavement configurations is as follows:

Figure 1: Comparison of required foamed bitumen pavement configurations (5M ESA)

Pavement systems for design traffic of 10 million ESA

Using the same dependable existing pavement, the exercise was repeated. This time the design loading comprised 10 million ESA to evaluate comparative requirements.

Figure 2: Comparison of required foamed bitumen pavement configurations (10M ESA)

The New Zealand design example required an increase of 35 mm to the thickness of the FBS basecourse (to 185 mm total) to maintain acceptable subgrade strain levels. For a design loading of 10 million ESA it would typically be recommended that thin asphalt surfacing such as open graded porous asphalt or stone mastic asphalt be utilised rather than a sprayed seal surfacing. Typically in New Zealand where a thin asphalt wearing course is employed, it is not modeled with asphalt fatigue properties when modelling with Austroads asphalt fatigue criteria.
This is due to the questionably large ‘theoretical’ structural improvement it provides. Rather, the equivalent thickness of asphalt surfacing is ‘added’ to the modeled thickness of foamed bitumen basecourse.

The Australian design required a nominal increase in FBS basecourse of 35 mm to a total thickness of 355 mm. It is highly likely that this would need to be constructed in two separate layers to permit good mixing and adequate compaction. Again, as for New Zealand approach it is likely that a thin asphalt surfacing would be implemented for a wearing course. As thin surfacing asphalt would be placed over the FBS basecourse modeled with asphalt fatigue criteria it would be modeled structurally with appropriate fatigue properties.

The South African design, if moving to a 40 mm asphalt wearing course, required the foamed bitumen BSM1 base thickness to be increased by 30 mm to 265 mm total. This could probably be achieved by bulking of the available 250 mm. However - if the rehabilitation was constrained to existing surface levels, then the pavement would need to be milled to a depth of -55 mm, then foamed bitumen stabilised to 195 mm (compacted) and then surfaced with a 55 mm asphalt wearing course. This suggests that increasing the asphalt surfacing by 15 mm more than doubles the pavement structural capacity. The designer would need to be comfortable that the asphalt surfacing and BSM1 dependable (ELTS) properties would be maintained.

While the New Zealand approach provides a ‘lesser’ thickness of foamed bitumen it is interesting to note that doubling the design traffic provides a similar change in thickness of FBS basecourse for all design approaches. However, it remains that the New Zealand approach results in significantly reduced thickness of FBS basecourse. Some practitioners may suggest that this is unconservative. On this basis is it necessary to review the performance of FBS in New Zealand – particularly those ‘older’ FBS treated pavements that have experienced the most traffic loading.

CHARACTERISTICS OF NEW ZEALAND FOAMED BITUMEN

FBS properties

The addition of foamed bitumen to an aggregate creates a material with unique properties relative to other more conventional treatment processes. Suitable materials are foamed bitumen stabilised (FBS) with bitumen - typically 2.7% to 3.0% by weight for NZ aggregates. This is a similar application rate to typical South African materials with little or no recycled asphalt products (and significantly less than typical Australian mixes). New Zealand practice is to employ a small amount of active filler (typically 1.0 to 1.5% cement by weight) to generate a visco-elastic medium that is strong and rut resistant - yet still flexible with a ductile failure mode.

The practice of adopting 1.0% to 1.5% cement provides good early strength. NZ road narrowness precludes contra-flow closures in most instances and traffic must run on freshly treated FBS basecourse within hours. It is extremely rare to use only bitumen in FBS mixes in NZ. Australia employs no more than 1.0% active filler (typically hydrated lime), while South Africa evaluates performance with no active filler, then if required up to 1% lime or cement is used.

A resilient modulus of 800 MPa is the long term (phase 2) baseline target for NZ design. Wet and dry indirect tensile strength (ITS) and unconfined compressive strength (UCS) testing is typically undertaken to interpolate the resilient modulus. For larger projects or ‘higher level’ design, Indirect Tensile Stiffness Modulus, Repeat Load Triaxial and Flexural Beam testing is undertaken to derive more comprehensive mix and FBS design parameters.

For NZ and South Africa FBS it is critical that the quantity of active filler does not move the materials into a ‘bound’ state where the cementitious binder overwhelms the ductile properties of the bitumen treated aggregate and cracking can occur. Different materials will respond differently to the same mix design so extensive testing is recommended for materials where performance history/response to FBS are not well understood.


**Strength relative to flexibility and failure mode**

Recent research and laboratory/field testing has demonstrated that very high compressive/tensile strengths can be achieved for some materials – no doubt partly due to the use of up to 1.5% cement. UCS results of as much as 5.0 to 6.0 MPa have resulted, which are significantly higher than typical UCS values of 1.0 to 2.5 MPa for FBS aggregates. This strength would place the material firmly into the ‘bound’ category for conventional cement stabilising where the risk of shrinkage or fatigue cracking would be considerable. However, extended unconfined compression testing of the FBS samples confirms that the failure mode is ductile with sustained load capacity well beyond 200% strain of the peak load. This suggests that provided the quantity of active filler is controlled to no more than 1.5%, the visco-elastic properties are maintained despite generating very high strength in some instances. Cracking has only been observed where lower pavement strength is significantly under-capacity generating very large strains in the FBS basecourse. Consequently FBS materials do not conveniently fit into conventional Austroads pavement design materials classifications as they are neither unbound nor bound. A wise South African Engineer refers to it as being ‘non-continuously bound’.

A research project (Xu et al, 2011 China) demonstrated that the optimal cement content for foamed bitumen mixes was 1.5% – and this falls into line with findings in New Zealand where this quantity of active filler optimises the rut resistance/early strength/temperature and moisture insensitivity of the mix without introducing the risk of brittleness/crack potential.

**PERFORMANCE OF NEW ZEALAND FBS SITES**

FBS utilising purpose-built equipment is a relatively recent introduction to New Zealand – with the oldest pavements experiencing no more than seven years service life at the time of preparing this paper. In order to provide assurance to industry, extensive post construction monitoring, research and full scale trials have been undertaken. With recognition that the New Zealand modeling approach provides significantly ‘thinner’ FBS layers it becomes increasingly important to ensure that design intent is matched by insitu performance.

Despite the ‘less robust’ design relative to overseas philosophy, the performance of FBS sites across New Zealand suggests that the design process is conservative (i.e. does not overstate performance). This is appropriate for this ‘interim period’ where FBS performance criterion are developed and validated. Testing of constructed pavements to derive remaining life shows that design assumptions have been met or surpassed. Experience suggests that pavements maintaining shape and stiffness for five+ years with no signs of distress are unlikely to suddenly develop severe problems in subsequent years.

**Characterisation and use of stabilised materials in New Zealand**

The characterisation of FBS pavements to provide dependable performance criteria for mechanistic modeling is required by New Zealand industry. It is not considered appropriate to adopt asphalt performance criteria or modified granular parameters as the FBS layers are unique in composition and performance.

To assist in developing a set of performance criteria a research project has recently been completed (Gray et al. 2011) to characterize stabilised basecourse materials properties. The intention was to establish a framework to derive a conceptual performance model for FBS in New Zealand. Due to the limited data pool (11 FBS sites), limited service life and a lack of identified failure modes this could not be developed with confidence. The recommendation was to continue to use the existing NZTA guidelines and also include a check that the modular ratio of the FBS layer to the underlying ‘subbase’ layer modulus is not more than 5.

This research project demonstrated the wide variety of interpolated modulus and tensile strain at the base of the FBS basecourse. While there were several apparent localised points of marginal load capability and remaining life, none of the evaluated pavement sections could be classified as failing. The back analysed FBS modulus was lower than anticipated with some
sites demonstrating a 10%ile modulus of 500 MPa, and 800 MPa being closer to the 20%ile value. This can be perceived as a benefit, as provided shear strength is adequate to resist rutting, shoving and moisture, the ability of the FBS to accommodate higher strains is enhanced.

**Full scale test track accelerated loading FBS experiment**

An earlier research project undertaken in New Zealand comprised a full-scale accelerated load experiment (Gonzalez 2009) of foamed bitumen pavements at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF). In this experiment, the same FBS mix design and aggregate that was comprehensively tested in the laboratory was used to construct six different pavement sections, each with different contents of foamed bitumen and cement. Three were constructed using foamed bitumen contents of 1.2%, 1.4% and 2.8% respectively, plus an active filler content of 1.0% cement. Additional subsections were constructed using cement only (1.0%), foamed bitumen only (2.2%), and also a control section with the untreated unbound material. These pavement sections were constructed directly upon a stiff soil subgrade such that the pavement systems could achieve terminal strains.

This project demonstrated that conventional 8-tonne heavy duty axle (40 kN) loading was not sufficient to introduce rut development to the FBS sections (while the other three subsections ‘failed’ in rutting). This loading was then increased to 60 kN and subsequent to this water was introduced to the pavement via saw-cutting and sprinklers to instigate distress. There was remarkably little difference in rut development for the three FBS sections which demonstrated a rut progression of less than 1 mm/1 million load cycles under conventional loading.

These results demonstrated that the addition of 1.0% cement provided superior rut resistance and fatigue capability for all three foamed bitumen/cement mixes, relative to the performance for mixes with bitumen or cement only. The conclusion was that the addition of cement provides early life strength and improved stiffness / rutting resistance for the FBS basecourse. Furthermore FBS mixes utilising cement are more ‘constructible’ while undertaking rehabilitations of ‘live’ carriageways with the ability to accommodate early trafficking.

**CONCLUSION**

The design process for FBS mixes in New Zealand is currently based upon tensile and compressive strength properties, which is questionable for a pavement layer that is lightly bound due to a preferential (non continuous) distribution of bitumen ‘spot welds’. Pavement design philosophy also adopts an unbound aggregate second phase ‘steady state’ condition which is also contrary to observed behavior. Limiting active filler to no more than 1.5% cement appears to ensure any failure mode is ductile rather than brittle, confirming that a ‘bound’ layer FBS fatigue criterion is not appropriate for New Zealand materials, typical aggregate depth and current design philosophy.

Research, testing and performance to date suggests that the current means of modeling FBS pavements in New Zealand, while not attempting to correctly represent mechanistic properties, does not overstate the fatigue capacity of pavement layers. The recent NZTA report (Gray et al. 2011) recommends continuing the use of guidelines provided in Austroads (2008) and the Transit NZ (now NZTA) Supplement (2007), but additionally limiting the maximum FBS modular ratio to five, rather than two as Austroads would require for unbound aggregates (or three as specified for South Africa BSM1 FBS materials). However, there is still plenty of scope for re-evaluation of FBS modeling and development of a representative failure mode with associated performance criteria for mechanistic modeling.

There are many variations, some subtle and some significant, in the approach to specification, mix design and modeling for FBS pavements in Australia, South Africa and New Zealand. The primary distinction of failure mode and modeling philosophy varies from granular state mechanistic design (with no performance criteria) in New Zealand to effective fatigue state mechanistic design (with asphalt performance criteria) in Australia. The South Africa approach has recently changed to an empirical pavement number structural design approach with
materials classification. This may appear somewhat more conservative than previous, but TG2 2009 notes that this approach has been validated using observed performance data. Ongoing research currently underway will continue to define the most appropriate approach for each country.

Current research in New Zealand and overseas will, in due course, facilitate a means to correlate mix design, modeling and dependable performance to provide the designer a methodology that accurately represents the unique properties of FBS basecourse. The approach to FBS design internationally is surprisingly varied at this point in time. In New Zealand a variety of research projects have been recently completed or are currently underway, working towards development of a dependable performance criterion for FBS mixes. This will be preferable to the current approach of adopting the modified non-sublayered granular properties designed to achieve a nominal 800 MPa for all FBS mixes.

In the meantime, during this period of re-evaluation and continued research, industry (and in particular pavement engineers) needs to continue to closely monitor and evaluate domestic and overseas developments and philosophies.

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Prior to this Allen was a senior geotechnical engineer with Opus Consultants, working primarily in civil engineering projects with a special interest in foundation design and pavement engineering having changed direction from Geologist to Engineer through the mid 90’s.

Recent work for Hiways has involved research, design and construction generally involving stabilising works – with particular interest in the foamed bitumen operations, while also maintaining support for Hiway Geotechnical and Environmental and asphalt / surfacing design and support for Hiway International. Allen chairs the New Zealand National Pavements Technical Group and sits on the Roading New Zealand Pavements Committee, and is also an active participant on several other industry working groups.

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