Performance Prediction of Cold Foamed Bitumen Mixes

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ABSTRACT: As the use of foamed bitumen mixtures in road pavements continues to grow on a global scale, so the need for performance functions that may be used in the design of pavements incorporating these materials becomes increasingly important. The challenge of modelling the behaviour of these mixes is complicated by the variety of foamed mixes that are produced and the range of properties that prevail.

This paper focuses on the performance of cold foamed bitumen mixes that have low binder contents (generally less than 2,5%) and a model that could be used in pavement design. Mixes with and without active filler (cement) have been considered. Due to the lightly bound nature of these materials, the shear properties of the foamed mix were adjudged to be critical parameters for the prediction of permanent deformation under repeated loading. A model was developed on the basis of triaxial testing carried out on foamed mixes comprising a range of aggregate types, including marginal and recycled materials. This included monotonic and dynamic triaxial testing on large foamed and granular specimens (300mm ϕ x 600mm high) and intermediate specimens (150mm ϕ x 300mm high). The model was then validated using accelerated pavement tests (APT) with the Model Mobile Load Simulator MMLS on a layer treated with foamed bitumen using Cold in-place Recycling (CIPR).

Through triaxial testing, the performance properties of the foamed mixes in terms of permanent deformation under repeated loading, can be related to the shear properties of the same material. This provides a link between laboratory mix design and field performance. Stress dependent models incorporating shear parameters, can be used to define the resilient foamed mix behaviour under dynamic loading. The ratio of deviator stress at failure under monotonic loading and the deviator stress in the pavement structure has been identified as a critical parameter for rut prediction. Using finite element methods that incorporate non-linear elements, these models can be applied in pavement analysis. The applicability of the rutting performance model has been verified using the NOLIP finite element analysis programme and guidelines for (lightly bound) foamed mix layer design now exist.

1 INTRODUCTION

The use of foamed bitumen technology in the rehabilitation of road pavements, as well as new pavement construction, was given a boost in the 1990s through the incorporation of the technology on cold recycling machines. The cold in situ recycling process (CIPR) is eminently suited for the reworking of the upper pavement layers of distressed roads to depths of 300mm and as a result has gained popularity over the last few years. Over the past 5 years, the number of recyclers in use in Southern Africa has increased sharply, see Figure 1. Such recycling machines have different capacities, some with the capability of emulsion and cement stabilization and others with the added capability of in situ foamed bitumen stabilization.

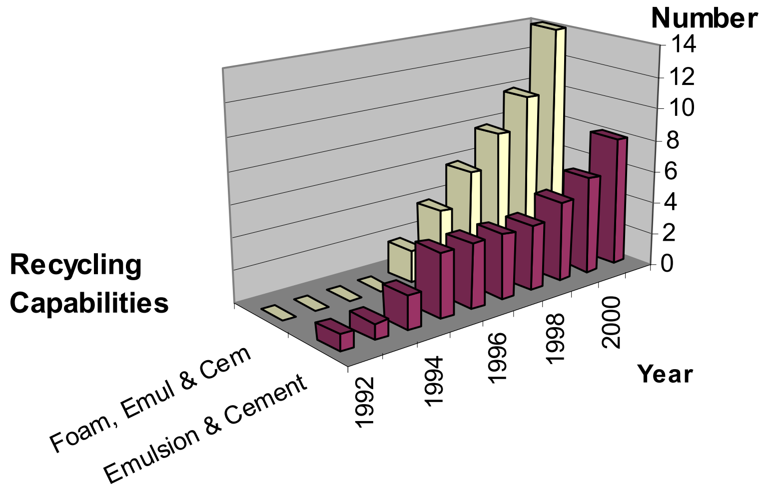


Figure 1. Number of In situ Recyclers in Southern Africa

There are many reasons why the in situ recycling process has gained such acceptance in the road pavements industry, including economic, technical and environmental advantages over alternative road rehabilitation options. The benefits of cold recycling have been widely publicized [Jenkins, 1994] and [Jenkins *et al.*, 1995], and do not require further elaboration in this paper. Suffice it to comment that the technical aspects of in situ recycling will be the focus instead.

Linked to the burgeoning use of foamed bitumen technology, is a dearth of performance criteria to assist in predicting the structural capacity of pavement layers incorporating this binder. The quest for development of a methodology for modelling the behaviour of foamed bitumen treated materials and prediction of the performance thereof, is complicated by the diversity of materials that need to be accounted for. Factors such as inclusion versus exclusion of cement, low versus high binder content and cold versus warm aggregate significantly influence the foamed mix properties[3] and hence the need for a range of performance functions. In South Africa, for example, the highveld region of Gauteng has an abundance of pavement structures with good quality gravels graded crushed stone due to its natural resources. The tendency is to minimize stabilizer contents in such cases and produce mixes that resemble weakly bound granular materials. The coastal region of KwaZulu Natal, however, has higher rainfall and weathering indices and follows the higher stabilizer content philosophy with appurtenant strongly bound and more asphaltic mixes. In general, foamed bitumen contents of less than 3% are used on the highveld and greater than 3,5% are commonly used on the coast.

Development of a unified model that satisfies this range of mixes is ambitious and probably unrealistic. With this as background, research was launched into the most cost-effective foamed mixes i.e. those with lower foamed bitumen contents and cement contents of 1% or less.

2 SELECTION OF MIXES

2.1 Gradation

The suitability of continuously graded aggregates for cold treatment with foamed bitumen has been verified in previous research. Aggregates with this type of gradation are also commonly used in pavement structures, either as granular, cemented or asphaltic base or sub-base layers. Although crushed materials are blended to achieve such gradations, weathered gravel that conforms to such a particle-size distribution, is also commonly encountered. Not surprisingly, such materials form a substantial proportion of the mineral aggregates utilised for foamed bitumen treatment, particularly in countries such as South Africa.

Continuously graded aggregates were therefore selected for a focused investigation into performance of cold foamed mixes. Two graded crushed rock samples used in the road industry as unbound base material, were selected for this purpose G1gau (quartzite) and G1eer (hornfels). In addition, a recycled layer with a blend of crushed hornfels and asphalt (23:77) called G2van was analysed. Finally, a mix granulate of crushed concrete and brick (78:22) called MGtud was analysed. The gradation of these four materials is published elsewhere [Jenkins,2000].

2.2 Material Properties & Preparation of Specimens

For purposes of mixing and compaction, characterization of the natural materials is necessary before treatment with foamed bitumen can be carried out. Only salient details of the material characteristics are provided in here, see Table 1, whilst the comprehensive details have been published elsewhere.

Table 1. Details of Granular and Foamed Mixes for Triaxial Test Specimens

Material	Optimum Moisture Cont. (%)	Binder Content (%)	Cement Content (%)	Compaction No. of Gyrations	Modified AASHTO dens. (%)*	Reference for detailed characteristics
G1gau	5.8	0	0	vibratory	103.9 (1.90)	[Van de Ven *et al.,* 1997]
$G1gau_2$	5.8	2	0	147	105.5 (0.79)	[Jenkins, 2000]
$G1eer_1$	6.8	1	0	233	104.8 (0.71)	[Jenkins, 2000]
$G1eer_2$	6.8	2	0	233	102.7 (0.94)	[Jenkins, 2000]
$G1eer_{2c}$	6.8	2	1	233	100.9 (0.68)	[Jenkins, 2000]
$G1eer_4$	6.8	4	0	233	101.2 (1.35)	[Jenkins, 2000]
$G2van_{1.5}$	6.8	1.5	2	200	97.8 (1.07)	[Jenkins & van de Ven, 1999]
MGtud	12.0	0	0	vibratory	100.3 (0.9)	[Saleh, 2000]
$MGtud_2$	12.0	2	0	vibratory	100.1 (0.3)	[Saleh, 2000]

* Average values are provided with standard deviation in brackets

The materials with G prefix originate from South Africa and the MGtud material from the Netherlands. The materials were all tested in their country of origin. The Wirtgen WLB10 ® laboratory foaming plant was used throughout, with untreated bitumen (without foamants) having an Expansion Ratio = 19, Half-life = 35 seconds and Foam Index = 533 for the G1eer mix. The remaining mixes all utilised bitumen with an Expansion Ratio = 15, Half-life = 15 seconds and Foam Index = 199 [Jenkins,2000]. Compaction of specimens for triaxial testing was carried out using a superpave gyratory compactor with settings of 1,25° angle of gyration and 600kPa vertical pressure. The void contents of 5,8% to 9,9% were achieved for all mixes except $G2van_{1.5}$, Mgtud and $Mgtud_2$. The latter mixes had higher void contents.

Following the manufacture of the foamed mixes, curing was carried out to simulate a medium-term cure for a moderate climate at 50ºC for 72 hours. The MGtud mix was cured at ambient temperature (20-23ºC) for 7 days because an oven large enough to accommodate the specimen could not be procured. The $G2van_{1.5}$ mix was cured at 25ºC for 24 hours, simulating an initial cure equivalent to early trafficking conditions that were investigated with accelerated pavement testing.

3 TRIAXIAL TESTING

3.1 Monotonic Triaxial Tests

From monotonic failure tests, the angle of internal friction ϕ and cohesion C of a material can be obtained using the Mohr-Coloumb model. The ratio of stresses within a granular material to the failure stresses has been shown to relate closely to the response of the material in terms of resilient and permanent strains [Huurman, 1997]. This model is considered to be applicable to foamed mixes that exhibit granular-type behaviour, as is verified through this research.

The triaxial testing facilities used for the investigation provide different set-up configurations. Delft University of Technology (TU) utilises specimens with a diameter of 300mm and a height of 600mm, making it eminently suited for analysis of coarse-grained materials. The University of Stellenbosch (US) triaxial set-up utilises specimens of 150mm diameter and 300mm height, thus limiting its suitability for coarse aggregates considering a desirable maximum particle size to diameter ratio of less than 1:8.

Displacement-controlled triaxial tests provide relatively uniform relationships between deviator stress and displacement. In the case of the TU set-up a displacement rate of 1mm/sec was used (strain rate of 10% per minute) was used and confining pressures of 12, 36 and 72kPa, whilst the US set-up utilised a displacement rate of 6.25mm/min (strain rate of 2,1% per minute) and confining pressures of 50, 100 and 200kPa.

Table 2. Summary of Shear Failure Parameters C and ϕ for Granular and Equivalent Foamed Mixes

Material	Type	C (MPa)	ϕ(°)	R^2
G1gau	Granular	0.082	53.0	*
$G1gau_2$	Foamed	0.166	44.7	0.99
$G1eer_1$	Foamed	0.162	45.8	0.95
$G1eer_2$	Foamed	0.156	45.9	0.92
$G1eer_{2c}$	Foamed	1.137	0.0	*
$G1eer_4$	Foamed	0.280	29.9	0.96
$G2van_{1.5}$	Foamed	0.821	0.0	*
MGtud	Granular	0.158	45.3	1.00
$MGtud_2$	Foamed	0.331	36.0	0.99

* Only 2 test results (other R^2 values indicate the fit of a linear failure envelope to 3 tests)

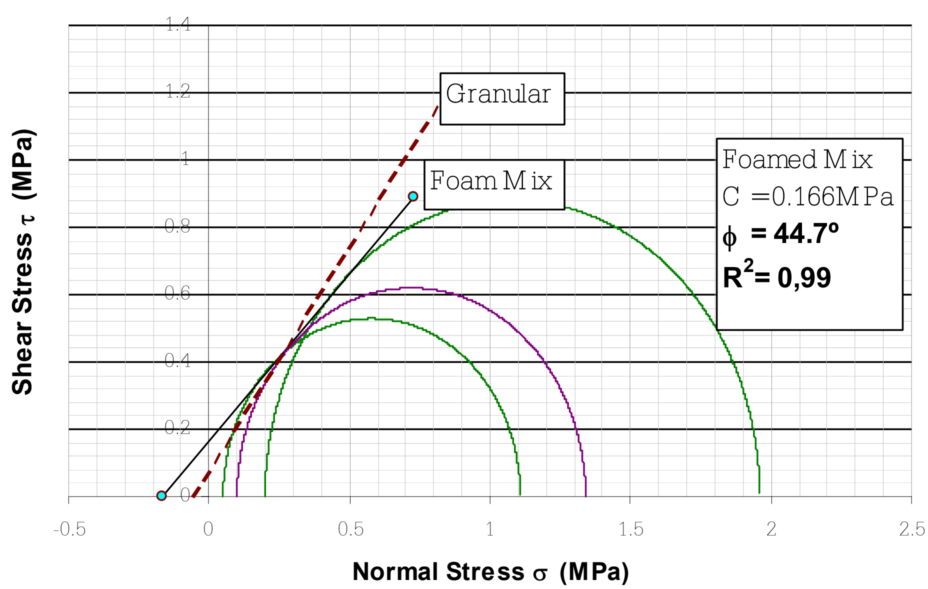


Figure 2. Mohr-Coloumb Circles for $G1gau_2$ Foamed Mix with Failure Envelopes for G1gau

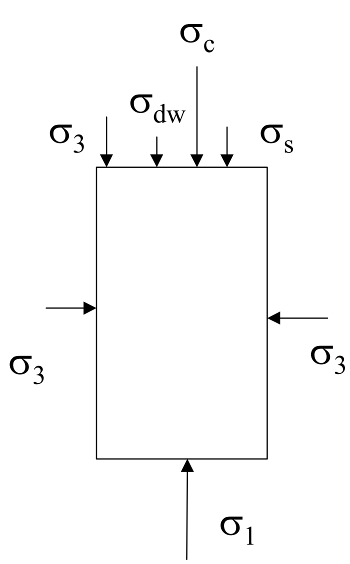
Mohr-Coloumb analysis of the results of the monotonic triaxial tests conducted on the granular materials and their equivalent cold foamed bitumen mixes provides a clearer insight into the function of the bitumen binder. According to the summary of the monotonic triaxial test in Table 2 and typical result in Figure 2, the friction angle ϕ decreases whilst the cohesion of the mix increases with the inclusion of foamed bitumen in a cold mix. This shift in the failure envelope leads to an increase in the maximum principal stress $\sigma_{1,f}$ at low confining stress σ_3 through the addition of foamed bitumen. Depending on the confining stress, $\sigma_{1,f}$ can increase by up to 100% relative to the granular material through the incorporation of 2% foamed bitumen.

Within the range of 0% to 4% binder, the cohesion value increases with a higher binder content. This trend is valid for all the mixes tested using the triaxial apparatus. With the incorporation of only foamed bitumen into a mix, the shear parameters continue to exhibit granular behaviour of the material. However, for the limited tests carried out with the inclusion of 1% of cement or more (in two mixes), the cold foamed mix obtains a marked increase in cohesion with reduction of the value of the internal friction to approximately 0º. This indicates that stress dependent behaviour i.e. non-linear mechanical behaviour becomes less applicable to foamed mixes with the addition of cement (unless light cement bonds are broken under traffic loading).

3.2 Resilient Deformation Behaviour

The resilient behaviour of granular and foamed bitumen treated materials can be tested in the triaxial set-up by applying relatively low stresses creating low strains, so that the elastic range of the particular material is not exceeded. It is assumed that within this elastic range, the stress history does not affect the material response. The selection of a range of stress magnitudes, in terms of a combination of deviator and confining stresses, allows the non-linear resilient deformation behaviour to be analysed on one specimen. A further condition for this test is that the number of load repetitions is limited i.e. permanent deformation is restricted.

The resilient modulus versus bulk stress M_r-θ tests were carried out at 25ºC at a frequency of 2Hz at US and 1Hz at TU. Different ratios of main principal stress to minor stress σ_1/σ_3 were used. The granular M_r-θ tests on the TU set-up use deviator to confining stress ratios of $\sigma_d/\sigma_3 = (\sigma_1-\sigma_3)/\sigma_3 \cong 2,3,4$ to 8 and levels of σ_3 of 12,24,36, 48, 60 and 72kPa. The layout of the stresses applied to the specimen in the TU triaxial set-up are shown in Figure 3.



where:

σ_1 = main principal stress [kPa]

σ_3 = minor principal (confining) stress [kPa]

σ_c = cyclic axial stress [kPa]

σ_s = static axial stress [= 12 kPa]

$\sigma_{d.w.}$ = dead weight stress [= 7 kPa]

$\sigma_1 = \sigma_c + \sigma_s + \sigma_3 + \sigma_{dw}$

Figure 3. Stresses Applied in Triaxial Test at TU [van Niekerk *et al.*, 2000a]

A variety of methods exist for the modelling of the stress dependency of M_r and ν [van Niekerk and Huurman, 1995]. Several of these models are applicable to the material behaviour demonstrated by the foamed bitumen mixes. Although a linear relationship between M_r and θ has been found applicable in a few circumstances, an exponential relationship is commonly used. Equation 1 provides the relationship for the **M_r-θ** model.

$$M_r = k_1\left(\frac{\theta}{\theta_o}\right)^{k_2} \tag{1}$$

In exceptional cases a linear relationship may be found between the total stress-state and the resilient modulus as shown in the **Linear Model** below.

$$M_r = k_3\,\theta + k_4 \tag{2}$$

In order to account for the decrease in resilient stiffness noticed as the vertical stress ratio $\sigma_1/\sigma_{1,f}$ approaches a critical value, the relationship in Equation 3 was developed by van Niekerk and Huurman [1995] (**M_r- σ_3-$\sigma_1/\sigma_{1,f}$ Model**).

$$M_r = k_5\left(\frac{\sigma_3}{\sigma_{30}}\right)^{k_6}\left(1 - k_7\left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{k_8}\right) \tag{3}$$

The selection of the primary variable as the bulk stress rather than the confining stress distinguishes the **M_r- θ-$\sigma_1/\sigma_{1,f}$ Model** in Equation 4 from the previous model. The latter model accounts for increase in material stiffness at the same confining stress but higher bulk stress.

$$M_r = k_5\left(\frac{\theta}{\theta_0}\right)^{k_6}\left(1 - k_7\left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{k_8}\right) \tag{4}$$

Where,

M_r	= Resilient Modulus (MPa)
θ	= sum of principal stresses (kPa)
	= $\sigma_1 + 2.\sigma_3 = \sigma_c + \sigma_s + 3.\sigma_3 + \sigma_{d.w.}$
σ_3	= minor principal stress (kPa)
σ_d	= deviator stress (σ_1 - σ_3) (kPa)
θ_0, $\sigma_{3,0}$,$\sigma_{d,0}$	= reference values (= 1 kPa)
k_1, k_3, k_5	= regression coefficients (MPa)
k_2, k_4, k_{6-8}	= regression coefficients (-)

The M_r- θ-$\sigma_d/\sigma_{d,f}$ Model utilises the same format as the M_r- θ-$\sigma_1/\sigma_{1,f}$ Model with the only difference being the use of the term $\sigma_d/\sigma_{d,f}$ to describe the stress ratio rather than $\sigma_1/\sigma_{1,f}$. Models used for application to the M_r-θ data depend on the form of the relationship between stiffness and stress, as analysed graphically. The results obtained for the materials detailed in this project have been analysed using the most appropriate model to provide their coefficients, see Table 3.

Table 3. Examples of M_r Model Coefficients for Granular Materials and Equivalent Foamed Bitumen Mixes

Material	Model	K_1	K_2	R^2		
G1gau	M_r-θ	55.6	0.207	0.73		
G2van$_{1.5}$	M_r-θ	48.0	0.330	0.86		
MGtud$_2$	M_r-θ	132.5	0.319	0.94		
Material	Model	K_3	K_4	R^2		
MGtud$_2$	Linear	0.721	569.35	0.97		
Material	Model	K_5	K_6	K_7	K_8	R^2
G2van$_{1.5}$	M_r-σ_3-$\sigma_1/\sigma_{1,f}$	273.2	-0.040	-2.801	0.917	0.99
MGtud	M_r-σ_3-$\sigma_1/\sigma_{1,f}$	350.0	0.300	0.600	0.100	0.69
MGtud$_2$	M_r-θ-$\sigma_1/\sigma_{1,f}$	30.0	0.600	0.700	1.000	0.89
MGtud$_2$	M_r-θ-$\sigma_d/\sigma_{d,f}$	48.0	0.500	0.500	1.200	0.90

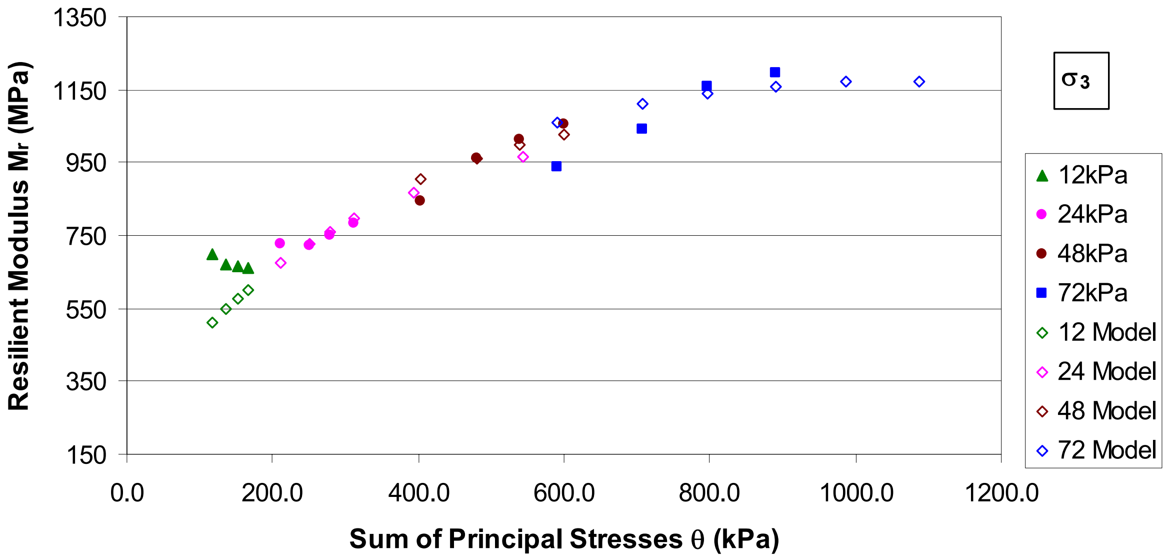


Figure 4. Resilient Modulus as a Function of Total Stress from Triaxial Tests on MGtud$_2$ Foamed Mix with 2% Binder after Conditioning with 10 000 Load Pulses at $\sigma_d/\sigma_{d,f}$ of 40%

The justification for stress-dependent behaviour is apparent when considering a typical relationship between resilient modulus and bulk stress, as shown in Figure 4. The model shown in this case is M_r-θ-$\sigma_d/\sigma_{d,f}$ and its output is shown relative to the acual results in the figure.

Poisson's Ratio can be calculated from the results of the triaxial tests using the radial and axial resilient deformation readings i.e. using the TU set-up. With both the resilient stiffness and Poisson Ratio measurements, the influence of conditioning on the results is significant. Initial measurements, taken on specimens before 10000 conditioning pulses have been exercised, show bound material behaviour; but this soon degenerates to unbound-type behaviour. Before conditioning, low Poisson Ratios and a high initial stiffnesses are evident, with the former averaging 0.18, a value that is commonly associated with cemented materials. The Poisson Ratio increases and M_r decreases to realistic values after conditioning, which is an important consideration for laboratory testing of these mixes.

Models for the change in Poisson's Ratio with variation in the stress condition have also been established by van Niekerk *et al.*(2000a) for granular materials. The models that were found to be suited for the modelling of the granular and foamed mixes are given below.

$$\nu = a + b.\left(\frac{\sigma_1}{\sigma_3}\right) \tag{5}$$

$$\nu = c.\left(\frac{\sigma_1}{\sigma_3}\right)^d \left(\frac{\sigma_3}{\sigma_{3,0}}\right)^e \tag{6}$$

After conditioning, the Poisson Ratios develop dependence on the σ_1/σ_3 stress ratio, see Figure 5. Compliance with this model is also indicative of granular material behaviour, as reported by van Niekerk *et al.* [2000a]. As with the M_r after conditioning, the Poisson Ratio relationship indicates disturbance of bonds during conditioning of the specimen. This procedure is therefore necessary for closer representation of field behaviour.

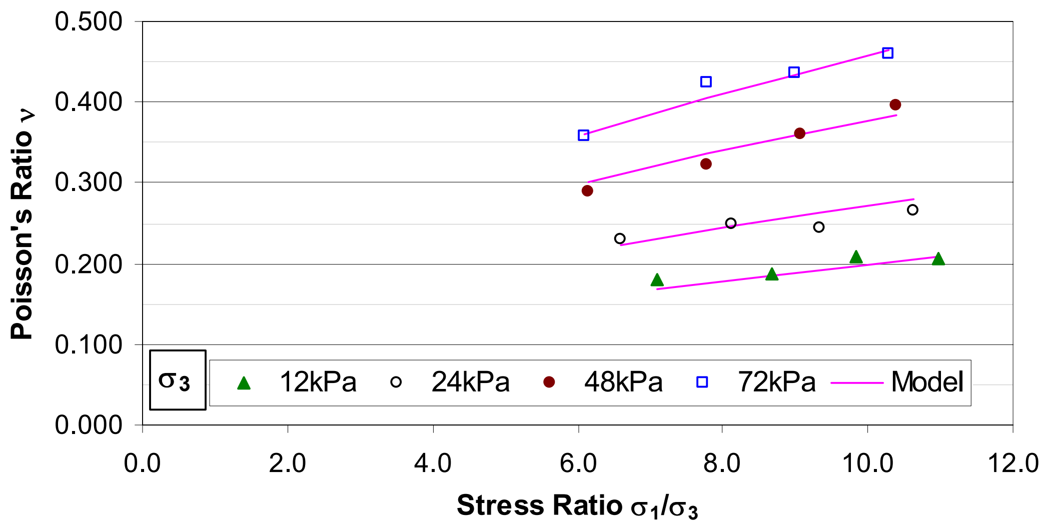


Figure 5. Poisson's Ratio as a Function of Stress Ratio σ_1/σ_3 from Triaxial Tests on $MGtud_2$ Foamed Mix after 10 000 Conditioning Cycles at $\sigma_d/\sigma_{d,f}$=40%

For the purpose of modelling foamed bitumen material behaviour in a pavement structure, relationships require establishment for the Poisson Ratio as a function of the stress. The results of regression coefficients for the Poisson Ratio relationships are provided in Table 4 for the granular material and its equivalent foamed mix with 2% binder. With bound mixes, jumps in the Poisson Ratio can occur through damage to a specimen during testing. This can be limited through reduction in the maximum σ_1/σ_3 ratio utilised.

Table 4. Summary of Coefficients for Poisson Ratio Models

Material	Model	a	b	R^2	
Mgtud	Equation 5	0.06279	0.05057	0.65	
Material	Model	c	d	e	R^2
$Mgtud_2$	Equation 6	0.01885	0.48607	0.483312	0.99

3.3 Permanent Deformation Behaviour

Permanent Deformation (ε_p) Tests are performed in the triaxial set-up by means of repeated load applications for a controlled stress ratio. Tests are carried out on virgin specimens at different stress ratios to establish the stress dependency of the permanent deformation behaviour. At least three but preferably four specimens require testing as part of a sensitivity analysis for each mix and set of test conditions, for a suitable model to be established. The typical results of permanent deformation tests for one material viz. G1eer$_2$ are provided in Figure 6, where the influence of the deviator stress applied : maximum deviator stress is apparent.

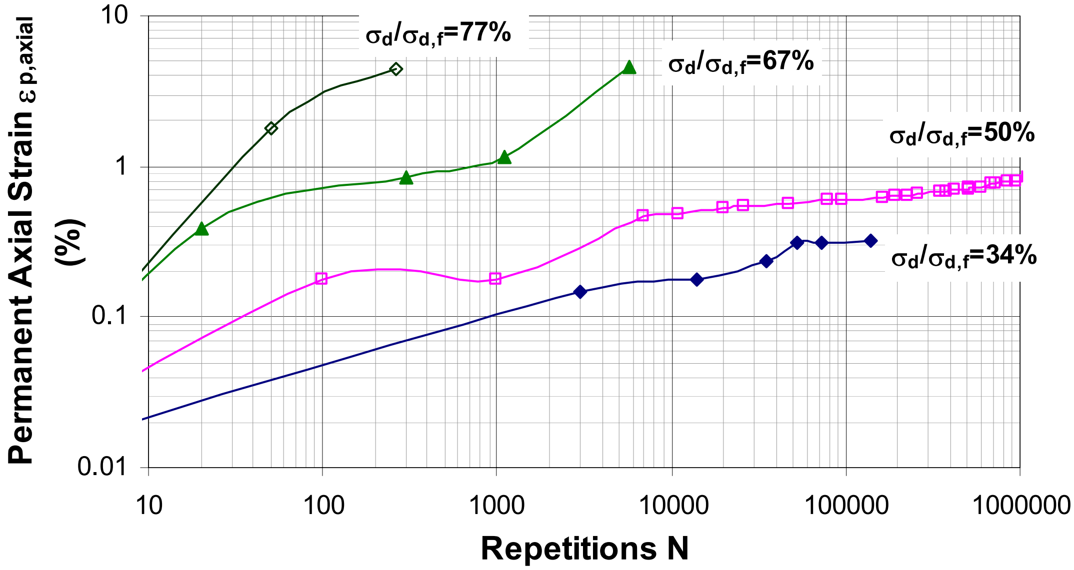


Figure 6. Permanent Axial Deformation versus Load Repetitions for G1eer$_2$ Foamed Mix with 2% Binder, Tested in US Triaxial with σ_3 = 50kPa

Relationships utilised for modelling of permanent deformation data, require account to be taken of the stress level at which the triaxial test is performed. Van Niekerk *et al*. [2000a] utilize the formula provided in Equation 7 for granular materials.

$$\varepsilon_p = A \cdot \left(\frac{N}{1000}\right)^B + C \cdot \left(e^{D \cdot \frac{N}{1000}} - 1\right) \tag{7}$$

Where,

$$A = a_1 \cdot \left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{a_2}; B = b_1 \cdot \left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{b_2}; C = c_1 \cdot \left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{c_2}; D = d_1 \cdot \left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{d_2} \tag{8}$$

As with granular materials, a critical stress ratio defines the boundary between stable ε_p growth and accelerated ε_p growth under repeated loading up to 10^6 cycles. As with the static triaxial tests and resilient deformation tests, a differentiation is necessary between stress dependent behaviour i.e. typical of granular materials, and strongly bound behaviour (including cement). The results of the ε_p tests for the range of materials, as shown for example in Figure 6, provide templates of deformation at different stress levels that are divided into foamed materials with and without cement to achieve this distinction. These results can be combined into one unified template for foamed mixes that exhibit stress dependent behaviour, see Figure 7.

As with the stiffness function, the deviator stress σ_d has been used in the equations in the place of the major principal stress σ_1. According to the average template, a ratio of $\sigma_d/\sigma_{d,f}$ = 55% defines this critical boundary for foamed treated materials (with 4% or less binder and without cement), see

Figure 7. Below a ratio of $\sigma_d/\sigma_{d,f} = 55\%$, less than 2% axial strain is observed in the foamed treated material after 10^6 load repetitions. For foamed mixes with 1% cement (or 2% cement in an early cure state), a ratio of $\sigma_d/\sigma_{d,f} = 52\%$ defines this critical boundary.

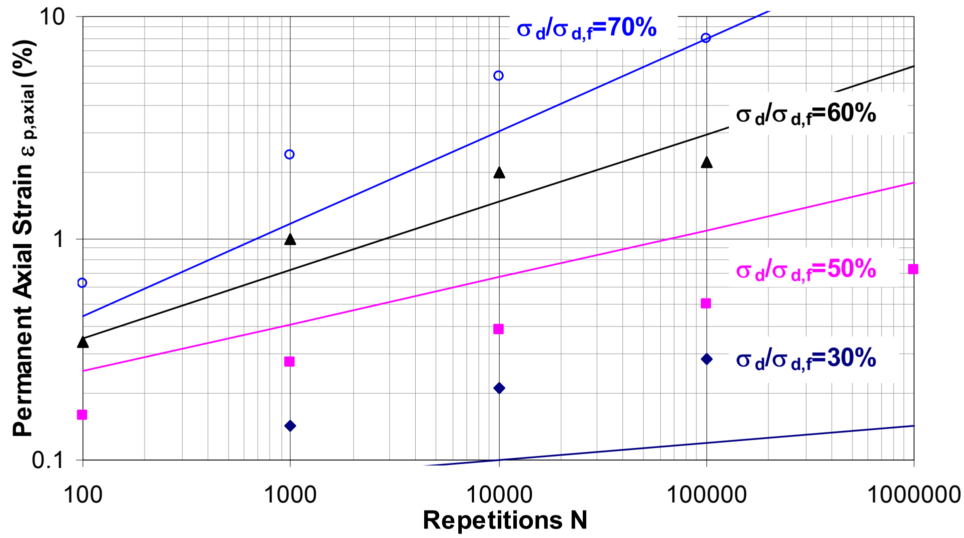


Figure 7. Template for Permanent Deformation Modelling of Foamed Mixes with <4% Binder and Without Cement based on Averaged Triaxial Results

The coefficients for the relationships applicable used for ε_p models are detailed in Table 5. Coefficients of 0 for c_{1d} and d_{1d} indicate an inactive second term in the Equation 7, which specifies that accelerated deformation will not occur at a high number of load repetitions to the already log-linear functions.

Table 5. Summary of Coefficients for $\varepsilon_{p,axial}$ Models Applicable to Foamed Bitumen Mixtures

Model	Mix	a_{1d}	a_{2d}	b_{1d}	b_{2d}	c_{1d}	c_{2d}	d_{1d}	d_{2d}	R^2
Equation 7 with $\sigma_d/\sigma_{d,f}$	Foamed Mix No cement	3.5	3.1	0.85	2	0	1	0	1	0.90
	Foamed Mix with cement	3.8	3.5	0.9	2	0	1	0	1	0.88

4 MODEL VALIDATION ON RECYCLED FOAMED MIX LAYER

4.1 Accelerated Pavement Testing

In order to test the validity of the permanent deformation models established, the influences of traffic on a recycled layer comprising G2van$_{1.5}$ foamed bitumen mix were simulated for analysis using a Model Mobile Load Simulator MMLS Mk3. The aim of the accelerated pavement testing was the comparison of field deformation and modelled deformation based on material properties from triaxial tests.

The MMLS Mk3 is an accelerated pavement testing tool that includes four pneumatic-tyred wheels that cycle in a closed loop, trafficking a trial section in a single direction, see Figure 8. The wheels are 300 mm in diameter and 70 mm wide. Each test was terminated at 100 000 to 150 000 axle-repetitions. Testing was carried out at ambient temperature, with air temperature ranging between 17ºC and 27ºC.

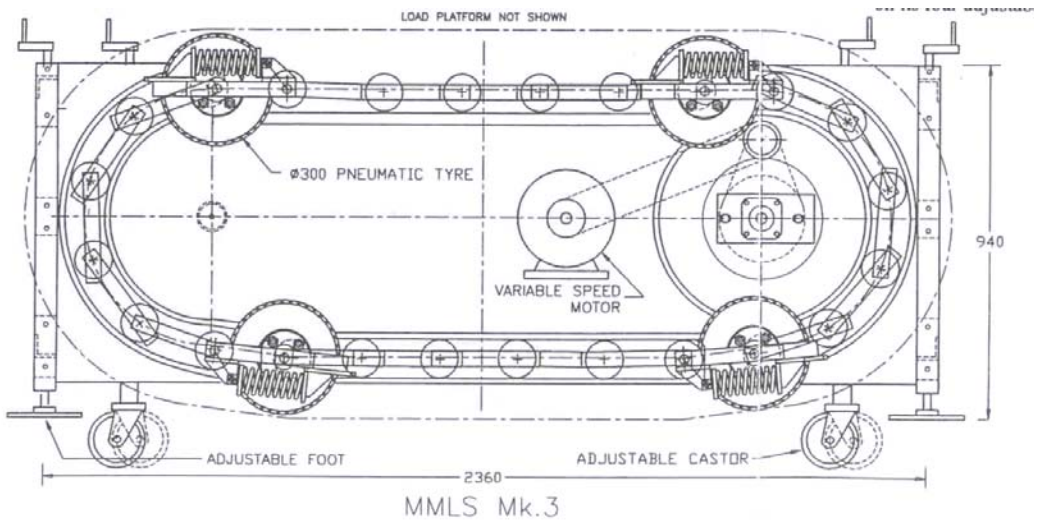


Figure 8. General Configuration of MMLS Mk3 Accelerated Pavement Testing

The proportion of ravelled material obtained from the APT trial was used to differentiate between rutting and ravelling, see Figure 9. Cold foamed mix can be susceptible to ravelling [Jenkins and van de Ven, 1999] and cognisance of this form of failure requires consideration in APT. In this way, a rutting profile caused solely by permanent deformation behaviour could be established.

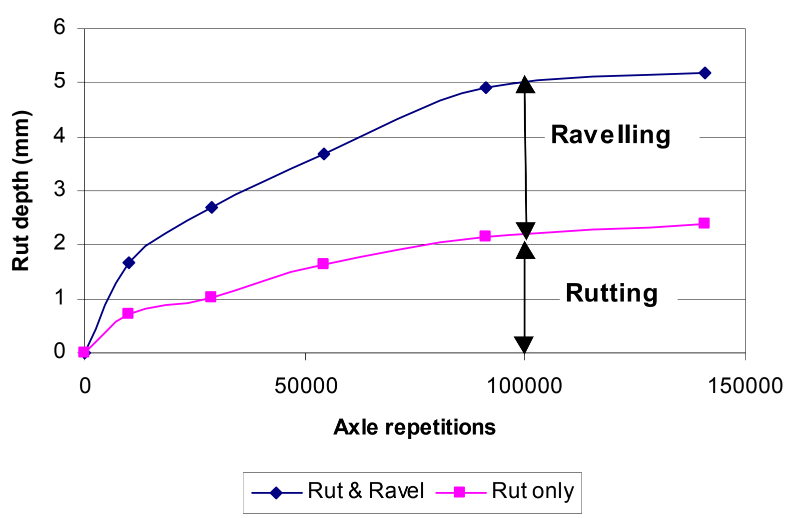


Figure 9. Deformation Profile for the Foamed Bitumen Section Tested 3 days after Compaction with MMLS Mk3

4.2 Finite ElementModelling

The NOLIP programme is an axial symmetric non-linear finite element model developed by Huurman [1997]. NOLIP is suited to modelling of non-linear materials as the programme determines the resilient modulus together with the stresses for each element in an iterative procedure as well as the displacements at each node, based on the applied wheel load. The resilient modulus and Poisson's Ratio are adjusted after every iteration based on the stresses calculated during the iteration. The iterative procedure continues until convergence is achieved, measured as the maximum difference in successive resilient moduli and Poisson's Ratios complying with a threshold limit.

Shear properties for the G2van$_{1.5}$ foamed mix are provided in Table 2. These values are representative of the material after a 24hour cure at ambient temperature. In addition, the M_r-σ_3-$\sigma_1/\sigma_{1,f}$ model for this material is included in Table 3. Utilising these material properties for the 300mm recycled, underlain by a 200mm ferricrete sub-base layer (M_r=200MPa and ν=0.35) and soil sub-grade (M_r=200MPa and ν=0.35) in NOLIP, non-linear modelling may be carried out using the MMLS wheel load configuration.

Analysis of the distribution of major and minor principal stress with depth in the pavement incorporating the recycled foamed mix layer, see Figure 10 and Figure 11, enables the deviator stress with depth to be assessed as shown in Figure 13. This deviator stress is the primary variable in determining the resultant permanent deformation in the pavement.

The non-linear behaviour of the material is evident from the M_r distribution with depth and lateral offset, see Figure 12. Due to stress dissipation, the material experiences a significant reduction of resilient modulus with depth i.e from 500MPa to 100MPa under the wheel.

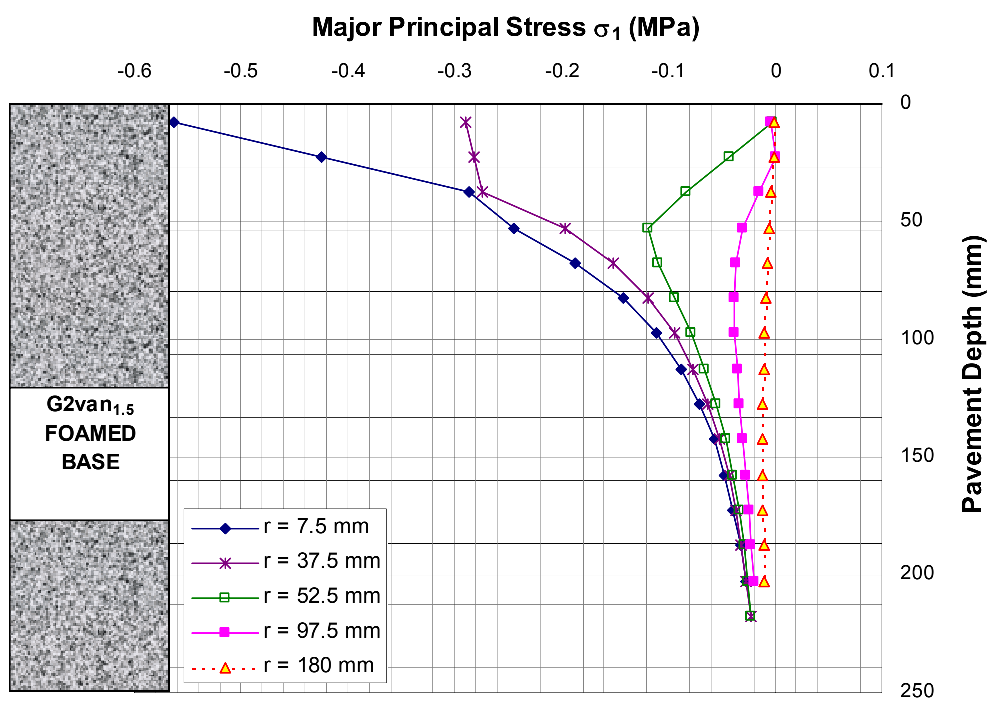


Figure 10. Major Principal Stress σ_1 with Depth in Recycled Foamed Mix under MMLS Mk3 Wheel Load (r = 33.4mm) [negative is compressive]

Note: Radial distance for stresses is measured from the centre of the wheel

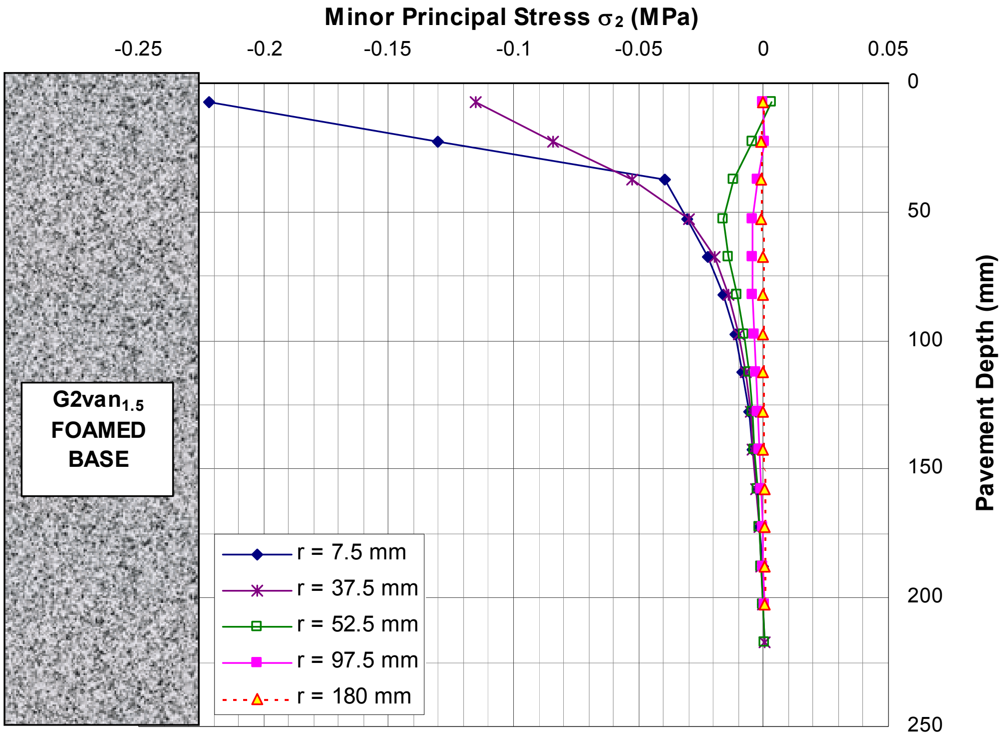


Figure 11. Minor Principal Stress σ_2 with Depth in Recycled Foamed Mix under MMLS Mk3 Wheel Load (r = 33.4mm) [negative is compressive]

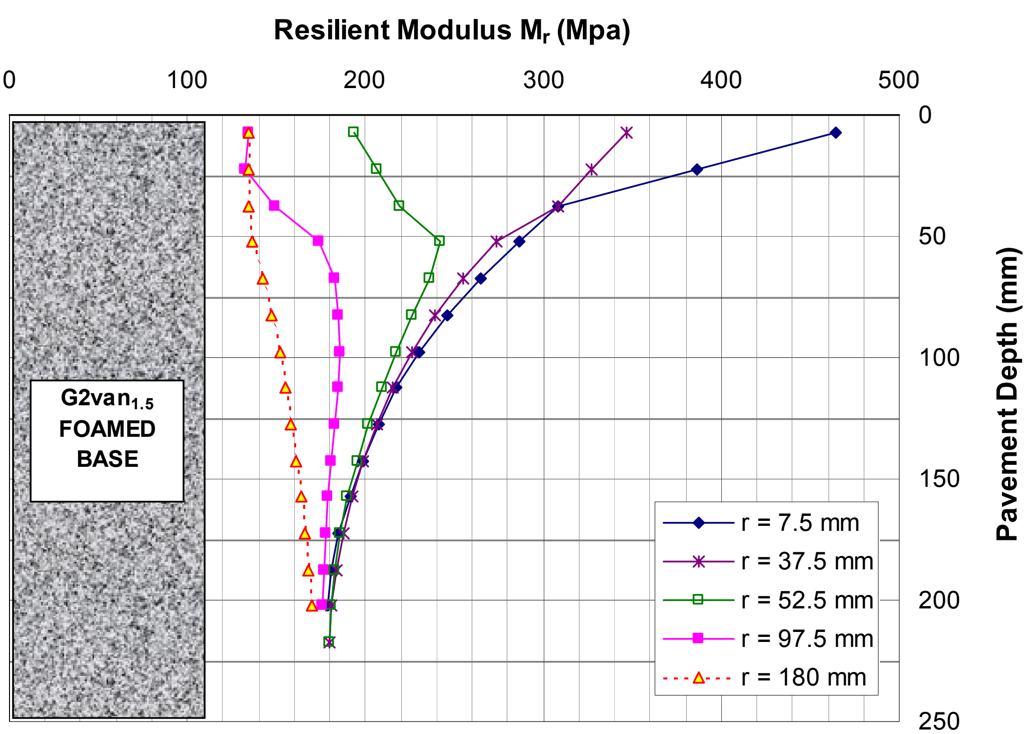


Figure 12. Resilient Modulus M_r with Depth in Recycled Foamed Mix under MMLS Mk3 Wheel Load (r = 33.4mm) at distances from Wheel Centre

The influence of the slushing, which delayed the commencement of accelerated pavement testing, creates softening of the upper layer. Slushing includes wetting of the surface of the recycled layer during final placement and compaction to encourage a tightly knit upper 50mm. Either water or diluted emulsion can be used. The slushing does, however, extends the curing time of the layer and can increase susceptibility to rutting during early trafficking. Account can be taken of the softening through adjustment in the material properties of the upper 60mm during the 3 day cure and remodelling with NOLIP.

Modelling of an upper layer that begins in a saturated state and cures with time, results in a substantial increase in the deviator stress ratio $\sigma_d/\sigma_{d,f}$, see Figure 13. These ratios are representative of the actual stress imparted during accelerated testing with the MMLS Mk3.

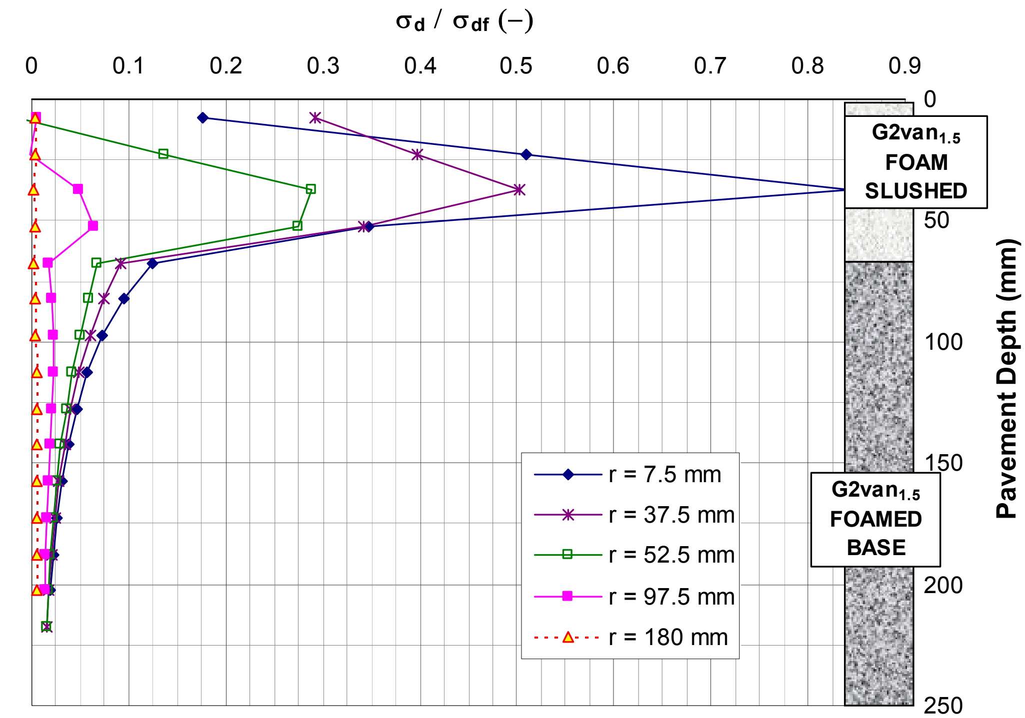


Figure 13. Deviator Stress Failure Ratio $\sigma_d/\sigma_{d,f}$ for Recycled Foamed Mix with Top 60mm Slushed ϕ=50º and C=50kPa, under MMLS Mk3 Wheel Load (r = 33.4mm) at Different Offsets from Wheel Centre

Utilising the stress ratios obtained and the models developed for foamed bitumen materials including cement provided in Table 5, a rutting profile can be determined for the G2van$_{1.5}$ material and compared with the results of the accelerated pavement testing. An acceptable correlation is obtained between the model and actual results, considering that the levels of rutting are reduced by almost an order of magnitude for scaled down APT with the MMLS (30% of size and 9% of surface contact area or footprint of a full-scale wheel).

Figure 14 provides a graphical comparison of measured and modelled rutting. With an additional 48 hours of curing and enhancement of in situ material shear proprerties, a significant reduction in rutting occurs, a phenomenon that is dependent on climate, traffic levels, material composition and slushing procedure. Immediate trafficking of a surface saturated (slushed) layer can result in premature deformation, therefore, if sufficient curing is not allowed subsequently.

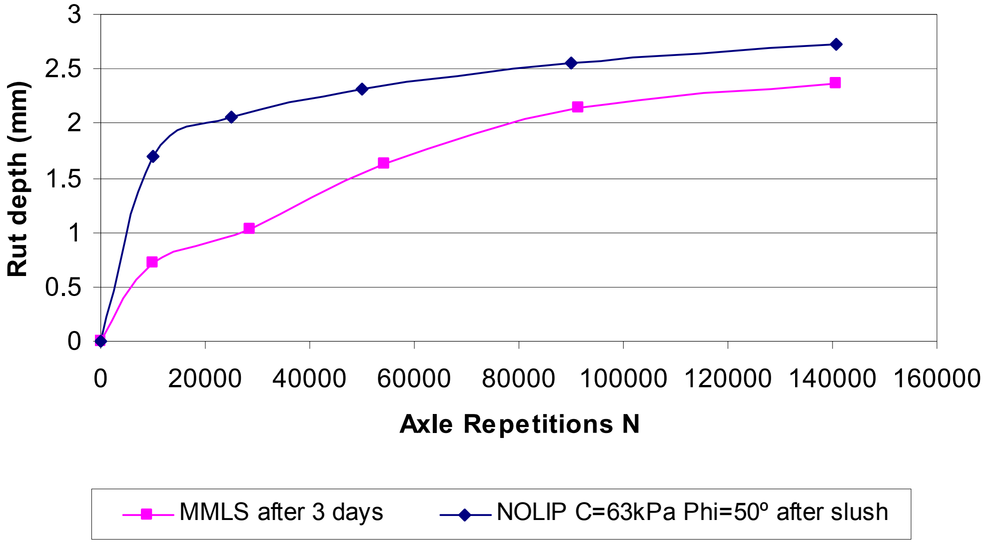


Figure 14. Rutting of G2van$_{1.5}$ Recycled Foamed Mix Layer from APT with MMLS Mk3 and Modelling using Finite Elements (NOLIP) after 3 days Curing

5 CONCLUSIONS

Compared with the equivalent granular material, cold mixing with foamed bitumen results in an increase in cohesion C, to in excess of 100kPa after moderate curing. An associated moderate reduction in the friction angle ϕ of less than 10º occurs after inclusion of the binder.

Certain procedures require consideration when testing foamed bitumen mixes in the laboratory. Conditioning of triaxial specimens before testing for resilient stiffness, for example, has a profound effect on the magnitude of M_r and behaviour at different stress levels. Exposure to 10 000 load pulses changes the resilient deformation behaviour from stress-independent to stress-dependent behaviour. Stress history i.e. the use of conditioning pulses, is necessary to simulate field conditions and thus obtain representative results.

Models used for resilient behaviour of granular materials are applicable to foamed bitumen mixes with less than 4% binder content and no cement. The M_r-θ-$\sigma_d/\sigma_{d,f}$ model and M_r-σ_3-$\sigma_1/\sigma_{1,f}$ models are most applicable.

The shear properties of foamed bitumen mixes with low binder and cement contents provide the basis for models that assist in the prediction of permanent deformation of these mixes under repeated loading. Although one mix of 4% binder was found to exhibit stress dependent behaviour, a limit of binder < 2,5% and cement < 1% is more applicable. Triaxial testing may be used to determine the shear properties that are used in finite element analysis to provide rut predictions for a road pavement incorporating such a foamed bitumen layer.

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7 KEYWORDS

Foamed bitumen, cold mix, triaxial, permanent deformation