Foamed bitumen mixes = shear performance?

K. J. JENKINS*†, F.M. LONG‡ and L.J. EBELS†

† Civil Engineering Department, Stellenbosch University
Private Bag X1, Matieland, 7602, South Africa
*kjenkins@sun.ac.za
‡ Modelling and Analysis Systems. Formerly of CSIR, South Africa
P.O. Box 882, Cullinan, 1000, South Africa

Road rehabilitation technology is currently under the global spotlight. Considering that cold bituminous binders, *e.g.* emulsion and foamed bitumen, have become more commonly used in cold recycling operations, there is a need to understand the performance properties of these materials. Great strides have been made in the modelling of foamed bitumen treated materials in recent years. Performance functions that may be used in the design of pavements incorporating these materials become increasingly important.

The challenge of modelling the behaviour of these mixtures is complicated by the variety of foamed mixtures that are produced and the range of mix variables that prevail. Binder content, active filler content, parent rock type, aggregate gradation, plasticity, moisture content etc can vary significantly from mix to mix. A unified approach to designing with these materials that accounts for all of these variables is exceptionally challenging.

This publication investigates the shear and dynamic properties of foamed bitumen stabilised mixes and their role in the performance modelling of these materials. Latest research findings are synthesized with the relevant aspects of the mix evaluation and classification procedures. Correlation of different testing protocols and mix compositions is made with a view to providing synergy to the research results and direction to their application in mix design and pavement design.

Keywords: Foamed bitumen, cold mix, triaxial testing, permanent deformation, shear properties, resilient modulus

1 Introduction

1.1 Background

The reasons for acceptance of cold in place recycling (CIPR) technology as a road rehabilitation process are numerous, including economic, technical and environmental advantages over conventional road rehabilitation alternatives. The benefits of cold recycling have been widely publicised (Jenkins 1994 and Jenkins *et al.* 1995) and do not require further elaboration. Suffice it to comment that the technical material characteristics associated with situ recycling will be the focus of this paper.

The ever burgeoning use of CIPR, with exponential increase in the number of recycler machines in the global roads industry, and associated increase in application of foamed bitumen technology, has resulted in a need for a fundamental understanding of performance criteria to assist in "predicting" the structural capacity of pavement layers incorporating this foamed binder. The quest for development of a behavioural and performance model for foamed bitumen treated materials is complicated by the diversity of materials that need to be incorporated in the model. Factors such as inclusion versus exclusion of cement or other active filler, low versus high binder content and cold versus warm aggregate significantly influence the foamed mix properties (Jenkins 2000) 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 and graded crushed stone, due to its natural resources. The tendency is to minimize stabilizer contents (both viscous and hydraulic) in such cases and produce mixes that resemble weakly bound granular materials. The coastal region of KwaZulu Natal, however, has higher rainfall and rock-weathering indices resulting in poorer quality materials; consequently, the region follows the higher stabilizer content philosophy with appurtenant strongly bound and more asphaltic mixes. In general, foamed bitumen contents commonly less than 2,5% are used on the and greater than 3,5% are commonly used on the coast. Higher bitumen contents result in greater flexibility of mixes but lower resistance to permanent deformation.

The guideline manual TG2 for the design and use of foamed bitumen treated materials (Asphalt Academy 2002) provides a conceptual two dimensional matrix to describe the interdependent influence of bituminous and hydraulic binders in cold mixtures, see Figure 1. This provides a useful background for the understanding of the need for the differentiation between performance models applicable to different types of cold mixes. This concept is further developed in this publication.

'[Insert Figure 1 about here]'

Given the wide range of mixes with different component variables, the development of a unified model that is applicable to all mixtures is ambitious and probably unrealistic. Notwithstanding this, recent research has continued to populate the matrix of different mix types, providing additional pieces to the jigsaw puzzle, thus assisting in forming a recognisable picture. In particular, the shear properties of foamed mixes are considered to be the keystone to the performance modelling. This publication aims to develop that framework further.

2 Background to Shear Properties

2.1 Philosophy

The spatial composition of foamed mixes, with varying proportions of aggregates, binders and moisture as well as different levels of compaction and curing, results in a range of behavioural characteristics and performance. As indicated in Figure 1, foamed bitumen mixes can vary from stress-dependent, to brittle or flexible. This gives rise to a variety of distress mechanisms *i.e.* shear deformation through to fatigue behaviour, making mix characterisation a challenge.

Researchers have generally agreed that, due to the predominantly granular nature of foamed bitumen stabilised mixes, the testing of the shear properties of the mix provides a sound basis for characterisation of the mix. Shackel *et al.*([1974) were some of the first to investigate the shear characteristics of foamed mix, carrying out numerous triaxial tests in Australia on Sydney breccia treated with foamed bitumen. These tests were run in the static and dynamic mode and a good correlation was found between these two modes in terms of permanent deformation.

Shackel *et al.* also established that resistance to permanent deformation is a function of the binder content and the degree of saturation (% voids filled with water by volume) of a foam treated material. The ratio of the axial strain to the peak axial strain ($\varepsilon_{axial}/\varepsilon_{peak\ axial}$) decreases with increasing binder content and degree of saturation. In addition, these researchers found that the rate of accumulation of axial strain is a function of the binder content (BC). The relationship of plastic strain with BC follows an inverted parabola with a minimum point at a given binder content, and increasing rates of deformation at either side of this minimum.

2.2 Stress Ratio Concept

Some of the preliminary modelling of foamed bitumen mixes was strongly influenced by previous models developed for lightly cement stabilised materials. The CSIR (Theyse 1998) reviewed some of the foamed bitumen treated materials that had been constructed in Kwa-Zulu in the early 1990s and used Dynamic Cone Penetrometer (DCP) results to develop preliminary structural models based on previous structural DCP models for cemented materials. These materials generally included more than 3% bitumen and only 1% lime.

Research that was running concurrently (Jenkins, 2000) focussed more on of the shear properties of foamed bitumen treated materials tested using the triaxial apparatus, for modelling performance. Jenkins followed guidelines established by Maree (1979) who tested granular materials in the triaxial apparatus, using three different types of triaxial tests:

- Monotonic triaxial tests to obtain shear parameters e.g. Cohesion (C) and Friction Angle (ϕ)
- Resilient Modulus dynamic triaxial tests of short duration to obtain resilient properties, and
- Permanent deformation ε_p (dynamic) triaxial tests of long duration.

The monotonic triaxial tests provide the fundamental shear parameters of the material. This defines the failure state and conditions of the material and can be used as a benchmark to analyse relative damage that will result from repeated load applications at lower stress conditions than the monotonic failure conditions. The stress ratio as defined by Equation 1, is the parameter used to express the applied deviator stress relative to the maximum deviator stress that the material can withstand. Jenkins (2000) found this parameter to work for foamed bitumen treated materials, in the same way that Maree (1979) and van Niekerk (2000) found this parameter to define the extent of shear deformation that will result in a granular material. This work was extended by CSIR (Long and Ventura 2003).

$$SR = \frac{\sigma_d}{\sigma_{d,f}} = \frac{\sigma_1 - \sigma_3}{\sigma_{1,f} - \sigma_3} = \frac{\sigma_1 - \sigma_3}{\sigma_3\left(\tan^2\left(45^\circ + \varphi/2\right) - 1\right) + 2C\tan\left(45^\circ + \varphi/2\right)} \quad \textbf{Equation 1}$$

Where,

σ_1 = applied major principal stress
$\sigma_{1,f}$ = maximum allowable major principal stress
σ_3 = minor principal stress
C = cohesion
ϕ = friction angle

The deviator stress ratio and not the major principal stress ratio, was found by Jenkins (2000) to be a fundamental performance parameter. For the stress state given as example in Figure 2, a significant difference is noted between the $\sigma_1/\sigma_{1,f}$ term and the $\sigma_d/\sigma_{d,f}$ term. As ϕ decreases, which is a tendency with foamed mixes relative to their equivalent granular materials, the differences in the two stress ratios becomes more significant. In the extreme (bound) case with ϕ = 0º, the $\sigma_1/\sigma_{1,f}$ term is affected by changes in σ_3 of a stress state, whilst $\sigma_d/\sigma_{d,f}$ is not influenced by confinement, thus making the latter a preferable ratio. Where σ_3 is a tensile stress, the disparity between the two stress ratios becomes greater, once again showing the $\sigma_d/\sigma_{d,f}$ to be preferable.

'[Insert Figure 2 about here]'

The failure envelope defined by C and ϕ in a Mohr-Coulomb plot is generally recognised as non-linear and therefore not entirely accurate in representing the shear properties of material. Nevertheless, provided that the stress applied to a material is not very low, the Mohr-Coulomb space remains sufficiently accurate for material modelling purposes.

Analysis of the triaxial test data that was available by 2000 provided indications that a critical stress ratio $\sigma_d/\sigma_{d,f}$ of between 40% and 45% is applicable to coarse grained granular materials in order to achieve $\varepsilon_p < 5\%$ at $N = 10^6$ (van Niekerk 2002) and between 50% and 55% for foamed mixes (Jenkins 2000), as reflected in Figure 3.

Subsequently, significantly more triaxial testing has been carried out, in all three modes of test protocols. The purpose of this publication is to synthesize the research results of foamed mixes of the period 2000 to 2005 and to highlight the new material performance characteristics that are becoming evident, in the context of the models that have already been established.

3 Fundamental Shear Parameters

3.1 Materials

3.1.1 Background. The two institutions in South Africa where the monotonic triaxial tests reported here, were carried out, are the CSIR Transportek and Stellenbosch University (SU). Foamed bitumen treated materials that have been tested at these two institutions during the past five years is reported here. The road building materials tested are categorised into three general groups as follows:

- Crushed rock
- Blend of sand and calcrete
- Ferricrete

In the crushed rock category, three different aggregate samples were tested. These graded crushed rock samples are commonly used in the road industry as unbound base material. The following were selected:

- Quartzite from a quarry in the Gauteng region (G1gau)
- Hornfels from the Eersterivier quarry in the Cape region (G1eer)
- Hornfels originally from the Contermanskloof quarry in the Cape region (N7)

Crushed rock samples were tested at SU (G1gau and G1eer) and reported in detail by Jenkins (2000). The details of the N7 samples are discussed elsewhere (Long and Ventura 2003). The 50/50 blend sand and calcrete was sourced from the R22 (P423) in the KwaZulu-Natal region (sandmix). The tests on this blend were performed by the CSIR and the aggregate properties are provided in (Long and Theyse 2005). The Ferricrete material was sourced from road P243 nearby Vereeniging (P243). This material was also tested by the CSIR and is detailed elsewhere (Theyse and Mancotywa 2001).

3.1.2 Grading. The gradings are discussed in detail in the various references listed above. For ease of reference they are summarised in Figure 4 and Table 1.

'[Insert Figure 4 and Table 1 about here]'

It can be seen from Figure 4 and Table 1 above that the gradings vary significantly. The grading of the sand-calcrete blend Sandmix is the only uniformly graded material and has a very fine grading. All the other mixes are continuously graded. Of the continuously graded materials, the crushed rock sample G1gau is the coarsest on the larger sieves. At the same time, this mix is fairly fine at the finer sieve sizes of the grading curve. The P243 Ferricrete material has the finest grading of the continuously graded mixes.

3.2 Monotonic Triaxial Tests

3.2.1 Specimen preparation. The test specimens of all granular materials were compacted on a vibratory table with dead weight loading. Specimens were tested at a range of densities and moisture contents. The relative dry densities of all the mixes discussed in this paper range from 90.7% to 105.5% of Modified AASHTO density (average of 98.6% with a standard deviation of 3.8%). The ratio of actual moisture content during testing over optimum moisture content (relative moisture content) of the material ranges from 18.8% up to 104.9% (average of 56.9% with a standard deviation of 22.4%).The range in relative density and relative moisture content is relatively wide, because this was used as a variable to establish the influence of the degree of compaction and saturation.

All specimens were prepared with a specimen height of 300mm. This results in a height : diameter ratio of 2.0. The specimens were tested at various foamed bitumen binder contents. An active filler in the form of cement was added to selected mixes at various percentages. To some of the Sandmix mixes lime was also used as active filler.

3.2.2 Curing. The curing protocols adopted for the various mixes vary considerably. The specimens at the CSIR were cured out of the mould at controlled room temperature for 28 days. The specimens at the SU were subjected to accelerated oven curing. A few different protocols were adopted, which are detailed by Jenkins (2000).

3.2.3 Test set-up and parameters. The triaxial test set-ups at the CSIR and the SU are fairly similar. Both institutions use hydraulic testing systems with closed-loop feedback. The loading capacity of both systems is equal at 100 kN. Deformation in the monotonic tri-axial tests is measured by means of the displacement on the actuator. At the CSIR confining pressures of 0, 100 and 200 kPa are used. At the SU a confining pressures of 50 kPa instead of 0 kPa is used, with the remaining confining pressures identical.

The rate of displacement of the monotonic tri-axial tests differs between institutions. At the CSIR a displacement rate of 0.67% strain/minute is applied. The tests at the SU were performed with a rate of 2.1% strain/minute. The difference in loading rate may be cause for differences in the results, especially at higher binder contents. This, however, falls outside the scope of this paper and is not further discussed here.

With regard to climatic control, the temperature during testing at the CSIR is carried out at room temperature. The testing at the SU has been carried out at 25ºC.

3.2.4 Results. In order to be able to compare the results of the monotonic tri-axial testing, specimens with similar relative densities and relative moisture contents were selected, see Table 2. The friction angle and cohesion at various combinations of foamed bitumen content and active filler content of the three general material groups are summarised in Figure 5 to Figure 7.

'[Insert Table 2 about here]'

'[Insert Figure 5 - Figure 7 about here]'

3.2.5 Discussion of results and trends.

Friction angle:
The friction angle of the untreated graded crushed rock is approximately 52º , the highest of the three material groups. The friction angle of the sand-calcrete blend and Ferricrete material are approximately 40º. The higher friction angle may be attributed to the grading of the material. The crushed rock is a continuously graded material, while the sand-calcrete blend is a uniformly graded material (see Figure 4 and Table 1). However, the grading is not the only parameter influencing the friction angle. Other parameters, such as angularity and surface texture, were not analysed here.

In Figure 5 and Figure 6 it can be seen that there appears to be a dependency of the friction angle on both the foamed bitumen content as well as the active filler content. The friction angle reduces with an increase in foamed bitumen content. At the same foamed bitumen content, the addition of active filler appears to increase the friction angle.

The reduction in friction angle at increased foamed bitumen contents can be explained by the fact that the added foamed bitumen acts as a lubricant, which reduces internal friction. The addition of active filler results in the formation of cementitious bonds. These bonds increase the internal friction.

The effect of the active filler on the friction angle seems to be more pronounced for the graded crushed rock samples. The increase in friction angle by adding active filler to the sand-calcrete blend is small.

The results of the Ferricrete material (Figure 7) are only available at a limited number of combinations of foamed bitumen content and active filler, which makes trends difficult to discern.

Cohesion:
The cohesion appears to be more strongly dependent on the amount of active filler than on the foamed bitumen binder content. Addition of active filler results in a considerable increase in cohesion. This can again be explained by the fact that cementitious bonds are created.

The apparent cohesion of untreated materials does not exceed 100 to 150 kPa. Untreated granular materials should have no true cohesion. The apparent cohesion shown is likely to be the result of suction since the specimens were partly saturated. The cohesion values shown for the treated materials were however measured on specimens with the same degree of saturation. The difference in cohesion values between the treated and untreated specimens could therefore be attributed to the effect of the binders added.

No clear difference in cohesion of the untreated material can be established between the three material groups. However, when the material is treated the cohesion increases with the addition of active filler. The graded crushed rock has the highest increase in cohesion as a result of the addition of active filler. Cohesion values of up to 400 kPa are observed. The non-linearity of the Mohr-Coulomb failure envelope, although not investigated, is likely to have an influence here.

The sand-calcrete blend has the lowest cohesion values after treatment. The addition of active filler does not have a great effect on the cohesion.

It can furthermore be seen that the addition of lime does not increase the cohesion as much as the addition of cement. In fact, that addition of 1% lime reduced the cohesion of the sand-calcrete blend. The addition of additional active filler (increase in active filler content from 1% to 2%) continues to increase the cohesion.

Effect of relative density and relative moisture content:
As mentioned in the section on specimen preparation, specimens of the three materials were prepared at a wide range of relative densities and relative moisture contents. The results are discussed in detail by (Long and Ventura 2003) and (Long and Theyse 2005). The most important findings are that:

- an increase in density results in higher friction angles and cohesion values;
- an increase in moisture content appears to have a small effect on the friction angle, but reduces cohesion. This holds especially for untreated material. The effect is reduced for higher foamed bitumen and active filler contents.

From the above it can be concluded that increased density (better compaction) reduces the risk of shear failure at similar load levels. Improved compaction thus increases the pavement life. It is important to prevent moisture ingress into the pavement layers (especially untreated material or material with low foamed bitumen and active filler content), because the wet material has a lower resistance to shear failure.

4 Resilient Modulus

4.1 Field versus Laboratory Measurements

The mechanistic analysis of pavements incorporating foamed bitumen treated layers requires the selection of resilient modulus values. The Conference on Asphalt Pavements for Southern Africa CAPSA 2004, highlighted disparities between resilient moduli of materials measured during laboratory triaxial tests, field measurement after APT and as part of LTPP evaluation. Jenkins (2000) showed that the resilient modulus of foamed mixes can exhibit strongly stress dependent behaviour when measured during triaxial tests, except for cemented type foamed bitumen mixtures. Long and Theyse (2004) report that use of the Heavy Vehicle Simulator on a foamed bitumen stabilised base results in an asymptotic reduction in resilient modulus with load repetitions. A typical example is provided in Figure 8 showing the application of different levels of wheel load.

‘[Insert Figure 8 about here]’

In contrast, the results of FWD measurements (Loizos *et al.* 2004) on a Greek Highway with a foamed bitumen stabilised base layer, show a reduction in deflection with time and traffic, see Figure 9. This would imply an increase in the back-calculated modulus value for the foamed mix layer. The effects of curing in this layer both in terms of strengthening and stiffening with moisture loss, appear to far outweigh the resilient stiffness reduction observed during HVS tests. The important question that arises is "What Resilient Modulus value is applicable for a foamed bitumen stabilised material when analysing a pavement structure?". The issue of resilient modulus selection is explored further in the next section.

['[Insert Figure 9 about here]'

4.2 Dynamic Modulus Selection

4.2.1 Introduction. In the resilient modulus dynamic triaxial test, the resilient modulus is measured over a range of confining and vertical stress levels, to enable the resilient modulus to be determined as a function of the stress condition. It is then possible to fit models to allow the resilient modulus to be predicted at any imposed stress condition within the tested range of stress conditions. However, the test is relatively expensive to perform, and so in some cases it is not included in a test program and only the static and permanent deformation dynamic triaxial tests are performed. It is possible to estimate a resilient modulus from the stress and strain measurements taken during a permanent deformation triaxial test. This method does not however, allow for the stress dependency of the resilient modulus to be determined. To estimate the resilient modulus from the permanent deformation test, the average of the values is calculated over a range of load repetitions where the moduli remain fairly constant.

In this section of the paper, resilient moduli determined from the permanent deformation test are compared with those determined using the resilient modulus test. The purpose of this is to check the accuracy and appropriateness of using the permanent deformation results to calculate the resilient modulus.

The dynamic triaxial tests used to perform the comparisons in this paper were done at Stellenbosch University (SU) and Delft University of Technology (TUD). In the SU triaxial testing device, during the permanent deformation test the specimen deformation is only measured using the LVDT placed in the loading ram of the testing device (called the machine-LVDT). During the resilient modulus test, the deformation is measured with LVDTs placed on the specimen and with the machine-LVDT. The resilient moduli determined based on the on-specimen LVDTs' data is consistently larger than those determined based on the machine-LVDT data. The permanent deformation tests performed at the CSIR measure the specimen LVDT using both on-specimen LVDTs and the machine-LVDT. Resilient moduli determined by these tests also consistently show the machine-LVDT determined moduli to be lower than the on-specimen LVDT determined moduli [Long and Theyse, 2005]. Both the SU and CSIR triaxial testing devices are well calibrated, and it is therefore surmised that this trend is an artefact of the testing machines and the position of the LVDT in the loading ram. The specimens tested using the TUD testing device are larger in size than the other specimens, *i.e.* 300 mm in diameter and 600 mm in height, as opposed to 150 mm diameter and 300 mm high specimens. In addition, TUD uses specimen mounted LVDTs for permanent deformation triaxial tests.

4.2.2 Results. Three materials were used to compare the resilient moduli determined from the resilient moduli and permanent deformation dynamic triaxial tests: A graded crushed stone from Eersterivier and another from Vanguard Drive (SU tests) and a mixed granulate (tested at TUD). The results are shown in Figure 10 to Figure 12 for the three materials. The Eersterivier material was tested using four combinations of foamed bitumen and cement. as shown in Figure 10. The Vanguard Drive material was tested at 1.5% foamed bitumen and 2% cement only, and the Mixed Granulate was tested with 2% binder and no cement. In the figures, the markers indicate the resilient moduli at the various bulk stress levels tested during the resilient modulus dynamic triaxial test as well as the initial stiffness during the permanent deformation tests at several stress ratios. The SU results from the 1990s provide consistently lower stiffness values than TUD; nevertheless, the trends rather than the absolute values are the main focus of these investigations.

‘[Insert Figure 10 - Figure 12 about here]’

The resilient moduli from the short duration triaxial test show a stress dependency, with the modulus increasing as the bulk stress increases. This is expected behaviour for granular type foamed bitumen treated materials. The resilient moduli from the permanent deformation test do not show a consistent trend with the stress ratio. This was also found with much of the CSIR data where the stress dependency of the resilient moduli from the permanent deformation test is not apparent (Long and Theyse 2005). Therefore it would appear that this test is not suitable to investigate the stress dependent behaviour of a material. Furthermore, one has to take into account that the range of resilient moduli determined in short duration triaxial test are from the same specimen measured during a single test. The long duration permanent deformation triaxial test, however, is carried out on a different specimen for every different stress ratio tested. This would result in inherent variability in the resilient moduli as a result of specimen variations.

Comparisons of the resilient moduli from all mixes of both materials do not show a consistent trend. In some cases the permanent deformation test predicts a higher resilient modulus, in some cases the values are in the same range, and in one case the resilient modulus is slightly lower than the values from the short duration dynamic test. It should be noted that the results shown from the permanent deformation test are from the machine-LVDT and are therefore conservative estimates of the resilient modulus. When this is considered, it is reasonable to conclude that, for the mixes shown, the permanent deformation test predicts a resilient modulus that is similar or higher than the resilient modulus test. In the situation where a higher resilient modulus is predicted, this means that using the permanent deformation test to predict the resilient modulus will typically result in a non-conservative estimate. The resilient modulus is a major parameter in mechanistic-empirical design, and a non-conservative estimate could result in fairly significant over estimation of pavement bearing capacity.

4.3 Stiffness Changes during Permanent Deformation Dynamic Triaxial Tests

In the Interim Technical Guideline for the Design and Use of Foamed Bitumen Treated Materials the first phase of the material behaviour in the structural design models is a stiffness reduction phase (Asphalt Academy 2002). This was included in the design model because this behaviour is consistently observed in Heavy Vehicle Simulator testing.

In permanent deformation dynamic triaxial tests performed on 150 mm diameter and 300 mm high specimens, no consistent trend in the resilient modulus as a function of the load repetitions has been observed (Long and Theyse 2005). In some cases the resilient moduli are relatively constant throughout the test, whereas in other cases it either decreases or increases at stages during the test. However, examination of a limited amount of triaxial data obtained from the 300 mm diameter and 600 mm high specimens (Jenkins 2000) indicates that the resilient modulus does appear to decrease with increasing load repetitions, until the modulus stabilises for a period and then begins to increase in value. This trend is illustrated in Figure 13 for four specimens tested at TUD. The resilient moduli are plotted as a function of the load repetitions, which are shown using a log scale to allow the stiffness reduction in the early part of the test to be easily observable. One of the specimens, with a stress ratio of 67% failed fairly early and so the stabilisation and subsequent increase in stiffness are not observed. For the other three specimens, the reduction in resilient modulus prevails until approximately 10 000 repetitions, after which it is relatively stable until approximately 60 000 repetitions at which stage it begins to increase.

Whilst this is a small sample size, and only one material, these results provide some interesting observations. It is postulated that the decrease in modulus observed is a result of small microscopic bonds in the foamed bitumen bound material fracturing under the action of loading. The increase in modulus after the 60 000 load repetitions is possibly due to subsequent densification of the specimen. Some permanent deformation dynamic triaxial tests are not run for more than 50 000 load repetitions because of the time required to run the test. That means that the increase in resilient modulus would not typically be observed. Finally, it may be that the boundary conditions in the smaller triaxial tests, and small amounts of deformations observed preclude the observation of the reduction in modulus as observed with the larger specimens. These observations and postulations need to be confirmed with more tests. It is important to note that the extent of reduction in the resilient modulus is significantly less for permanent deformation triaxial tests than for APT testing. Considering the findings of stiffness increase during LTPP testing, this type of triaxial test may be providing more realistic resilient modulus results.

5 Permanent Deformation

5.1 Test Protocols

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 three sources of permanent deformation triaxial results that are discussed in this paper have been tested using different protocols and are outlined in Table 3. In addition, the mathematical functions that are used by the various research institutions differ, as shown. The equations take the form of:

- hyperbolic linear (HL), which is applicable to permanent deformation results with a two-phase structure i.e. primary phase with bedding in at a fast rate followed by a secondary phase of stabilised deformation (either plateau or constant rate)
- double exponential (DE), which includes the same two phases as HL but is followed by tertiary flow defined by accelerated deformation.

Relationships used by SU and TUD utilised for modelling of permanent deformation data, require account to be taken of the stress level at which the triaxial test is performed. The DE formula utilised by van Niekerk *et al*. (2000) for granular materials, shown in Equation 2, was found to be applicable to foamed bitumen mixes too.

$$\varepsilon_p = A \cdot \left(\frac{N}{1000}\right)^B + C \cdot \left(e^{D \cdot \frac{N}{1000}} - 1\right) \qquad \textbf{Equation 2}$$

Where,

$$A = a_1 \cdot \left(\frac{\sigma_1}{\sigma_{1,f}} \right)^{a_2} ; B = b_1 \cdot \left(\frac{\sigma_1}{\sigma_{1,f}} \right)^{b_2} ; C = c_1 \cdot \left(\frac{\sigma_1}{\sigma_{1,f}} \right)^{c_2} ; D = d_1 \cdot \left(\frac{\sigma_1}{\sigma_{1,f}} \right)^{d_2}$$ **Equation 3**

Regarding the test protocols, the results of Stellenbosch University (SU) and Delft University of Technology (TUD) (Jenkins 2000) utilise data from up to one million load repetitions and apply a double exponential DE equation to model the permanent deformation measurements. CSIR results (Long and Theyse 2005) were tested to 50 000 repetitions and, with very few exceptions, do not exhibit tertiary flow behaviour. For this reason the hyperbolic linear HL modelling functions were applied in the vast majority of cases.

'[Insert Table 3 about here]'

5.2 Stress Ratio Synthesis

5.2.1 Introduction. Numerous types of foamed bitumen mixtures have been investigated using triaxial testing, as noted in Section 3. Due to the diversity of variables incorporated in these mixes, it is necessary to first classify the mixes in terms of their aggregates, before relationships between stress ratios and permanent deformation can be compared. For this purpose, as with the shear parameter classification, three classes have been identified namely graded crushed rock, gravels and sand. Sand materials are not discussed in this section.

5.2.2 Graded Crushed Rock. Long and Ventura (2003) provide analyses of repeated load triaxial tests of N7 and present the number of load repetitions required to achieve a certain level of plastic strain given variation in the stress ratio, see Figure 14. This analysis is based on almost exclusive HL function fitted to tests terminated at 50 000 repetitions and then extrapolated to establish the data for the graph. The distribution of results does not indicate significant differences in foamed bitumen mixes with different binder contents. The inclusion of 2% cement in the mix results in a notable increase in sensitivity of the material to higher stress ratios.

'[Insert Figure 14 about here]'

During extensive PD dynamic triaxial tests van Niekerk (2002) published numerous results that show accelerated permanent deformation after 100 000 load cycles and even after 400 000 load cycles in coarse granular materials, refer Figure 15. This occurs at stress ratios generally, but not exclusively, in excess of 40%. The results highlight the need to extend the PD triaxial tests to beyond 400 000 cycles for granular and possibly foamed bitumen materials. This phenomenon was investigated further using the diverse array of tests carried out on foamed bitumen mixes and is discussed in this section.

[Insert Figure 15 about here]'

The effects of density and saturation have also been included in the analysis of PD triaxial tests (Long and Ventura 2003). It is apparent from Figure 16 that extension of the PD triaxial tests has a significant influence on the stress ratio function that is developed. A critical stress ratio of approximately $\sigma_d/\sigma_{d,f}$ = 55% defines the deviation point in the data sets for granular crushed stone materials. At stress ratios SRs below 55% there is good correlation between the collective data. At SRs above 55% the short duration PD tests become un-conservative. This deviation point is somewhat higher than the critical deviator stress in the large triaxial setup of TUD , which is approximately $\sigma_d/\sigma_{d,f}$ = 40% to 45% for granular materials.

‘[Insert Figure 16 about here]’

Extending the analysis to foamed bitumen mixes results in the same trend, refer Figure 17. The critical stress ratio of $\sigma_d/\sigma_{d,f}$ = 52% to 55% identified by Jenkins (2000) fits with the collective data. Once gain, below the critical stress ratio, reasonable correlation between the collective data is apparent. Dominance of neither density nor saturation in the permanent deformation behaviour is apparent from this data.

‘[Insert Figure 17 about here]’

Similar trends are applicable to foamed mixes that are loaded repeatedly until 17% plastic strain levels, refer Figure 18. Once again, a critical stress ratio of $\sigma_d/\sigma_{d,f}$ = 55% up to approximately 60% is evident. Where higher foamed bitumen contents are applied in the absence of cement, limited data indicates that higher load repetitions can be withstood at the lower stress ratio levels.

‘[Insert Figure 18 about here]’

Superimposition of the collective data for foamed mixes produced using graded crushed rock is illustrated in Figure 19 and Figure 20. For each of these figures a proposed relationship between the stress ratio of the foamed mix $\sigma_d/\sigma_{d,f}$ and the number of load repetitions to the relevant plastic strain level, is provided. This performance function ignores the data to the right of the critical stress ratio (the zone of concern), which is considered to estimate un-conservatively high allowable repetitions due to extrapolation of short duration test results.

‘[Insert Figure 19 and Figure 20 about here]’

5.2.3 Gravel Material. Although the mix granulate from Delft (MGtud) is not weathered gravel, its characteristics indicate that it would be best classified in this category. The P243 foamed mix results of PD triaxial tests performed by CSIR are compared with those of mixed granulate of TUD foamed mix MGtud. The former material tests used 150mm ϕ x 300mm high specimens whilst the latter used 300mm ϕ x 600mm high specimens. van Niekerk (2002) showed that specimen size as a variable introduces shifts in the results of triaxial tests if coarse material gradations are used. Generally, unless the gradations are scaled down, the smaller specimen size provided lower resilient modulus values.

The extent of the influence of specimen size on the shift in results could assist in providing some explanation for the shift in permanent deformation results below the critical stress ratios for gravel materials, shown in Figure 21. However, the dramatic decline in the number of load repetitions required to produce 4% plastic strain for the MGtud mixes, once again differs dramatically from the P243 results that were extrapolated from PD triaxial tests with a maximum of 50 000 cycles. The same trend is evident when 17% plastic strain is utilised as the terminal deformation value highlighting the need to treat the data in the “zone of concern” with caution.

‘[Insert Figure 21 about here]’

5.2.4 General. Notwithstanding the issues around the “zone of concern” identified, the collective results for permanent deformation of foamed mixes based on the stress ratio as the critical variable appear to provide a sound basis for performance analysis and pavement life estimation. The variability of the PD results with regard to types of foamed mixes appears to preclude classification of foamed mixes on this basis. It could be argued that foamed mix type is inherently included in the stress ratio $\sigma_d/\sigma_{d,f}$ as it defines $\sigma_{d,f}$ through the shear parameters of the mix.

6 CONCLUSIONS

6.1 Shear Parameters from Monotonic Triaxials

From the collective results of monotonic triaxial tests on foamed mixes, it can be concluded that:

- The continuously graded crushed rock tested has higher angle of internal friction than the more uniformly graded material tested (Ferricrete and blend of sand and calcrete);
- The friction angle reduces with increasing foamed bitumen contents;
- At constant foamed bitumen content the addition of active filler increase the friction angle. This effect is more pronounced for the continuously graded material;
- The cohesion is to a large extent controlled by the active filler content;
- There is little difference in the cohesion of the untreated graded crushed rock, sand-calcrete blend and Ferricrete. However, the increase in cohesion by adding active filler is the largest with the graded crushed rock;
- The addition of lime as active filler results in a similar effect as the addition of cement, but of much lower magnitude;
- Higher density results in increased cohesion values and friction angles;
- An increase in moisture content reduces cohesion.

6.2 Resilient Modulus

Based on the results of short duration dynamic triaxial tests, it is concluded that:

- Estimating the resilient modulus from the permanent deformation dynamic triaxial test does not always provide a reasonable estimate of the true resilient modulus as measured using the resilient modulus from the resilient modulus triaxial test.
- These estimates should therefore be used with care, and where possible, the resilient modulus test should be run to obtain a more reliable result and to capture the stress dependency of the material.
- Some decline in resilient modulus can be expected with repeated load application without change in the curing regime of a foamed mix.

6.3 Permanent Deformation

The synthesis of collective results of extensive permanent deformation tests carried out at CSIR, SU and TUD have enabled the following conclusions to be drawn:

- The stress ratio $\sigma_d/\sigma_{d,f}$ is a critical parameter that defines the permanent deformation behaviour of foamed bitumen stabilised mixes.
- It is necessary to continue PD triaxial tests to beyond 250 000 repetitions in order to ensure that any tertiary flow behaviour is captured in the measurements and hence the modelling.
- A critical stress ratio value of conservatively $\sigma_d/\sigma_{d,f}$ = 50% to 55% defines the boundary between a stable rate of deformation under repeated loading and an accelerated rate of permanent deformation for foamed mixes.

- The variability of the PD results with regard to types of foamed mixes appears to preclude classification of foamed mixes on this basis. Nevertheless, foamed mix type is inherently included in the stress ratio $\sigma_d/\sigma_{d,f}$ as it defines $\sigma_{d,f}$ through the shear parameters of the mix.
- Where higher foamed bitumen contents are applied in the absence of cement, limited data indicates that higher load repetitions can be withstood at the lower stress ratio levels.
- Dominance of neither density nor saturation is apparent in the permanent deformation behaviour of foamed mixes.

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Table 1. Grading properties

Material	$P_{0.075}$[a]	$P_{4.75}$[a]	D_{max}[b]	Grading modulus[c]	Grading coefficient[d]
$G1_{eer}$	8.3	54	19	2.38	34
$G1_{gau}$	5.2	37	37.5	2.54	23
N7	7.9	37	53.0	2.56	24
$Sand_{mix}$	9.6	99	13.2	1.05	1.6
P243	12.6	56.6	37.5	2.14	30

[a]P_n indicates the percentage passing the sieve size n
[b]Dmax is the maximum particle size (smallest sieve through which 100% passes)
[c]The grading modulus is defined as $(R_{2.0} + R_{0.425} + R_{0.075}) / 100$, where R_n is the percentage retained on sieve size n
[d]The grading coefficient is defined as $(P_{26.5} – P_{2.0}) \times P_{4.75} / 100$

Table 2. Average relative density and relative moisture content (standard deviations between brackets) of selected specimens

Material	Relative density (% Mod AASHTO)	Relative moisture content (% OMC)
Graded crushed rock	101.0 (2.9)	36.3 (8.1)
Sand-calcrete blend	99.8 (1.8)	35.4 (7.2)
Ferricrete	99.0 (1.0)	49.8 (5.2)

Table 3. Triaxial Settings for Permanent Deformation Tests

Research Institution :	SU	TUD	CSIR
Confining Stress σ_3 [kPa]	50	12	80 or 140
Frequency [Hz]	2	5	2.5
Temperature [ºC]	25	25	25
Specimen size [mm]	150ϕ x 300	300ϕ x 600	150ϕ x 300
Termination repetitions	Up to 10^6	Up to 10^6	50 000
Model type	DE	DE	HL DE

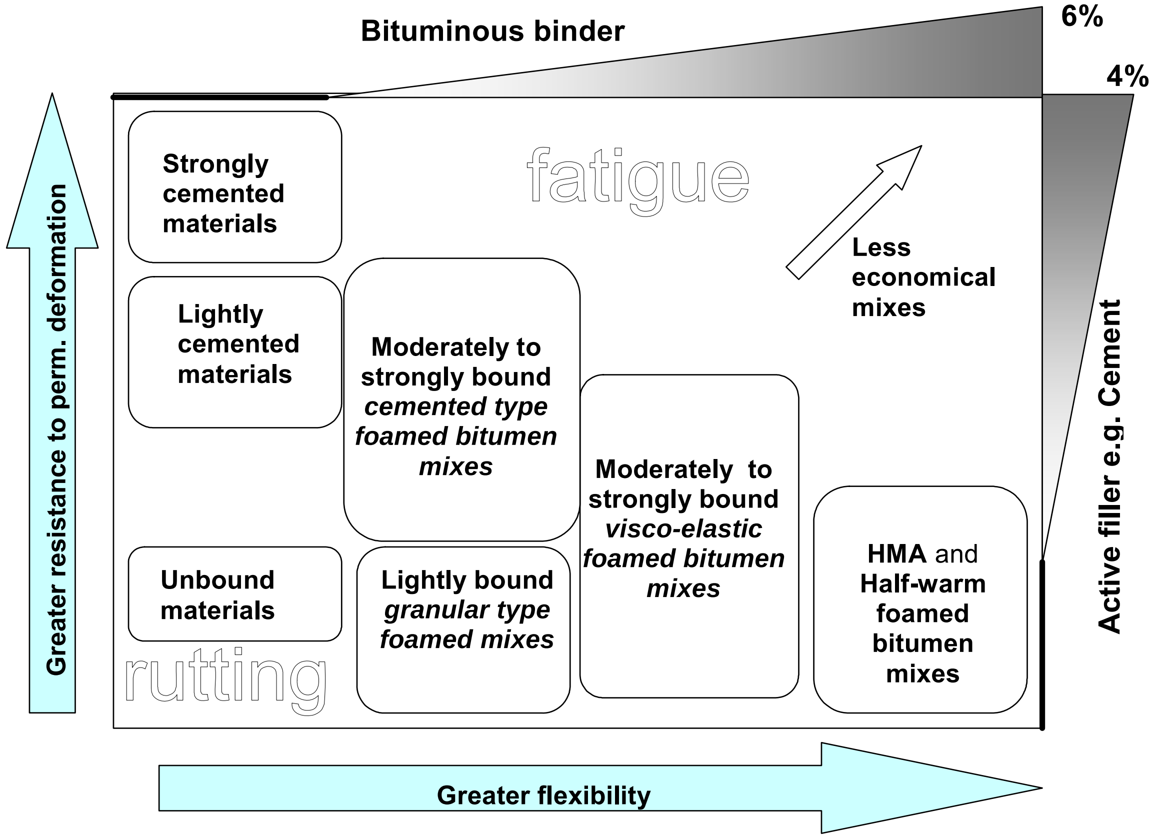


Figure 1. Types of Foamed Bitumen Mixes, after Asphalt Academy (2002)

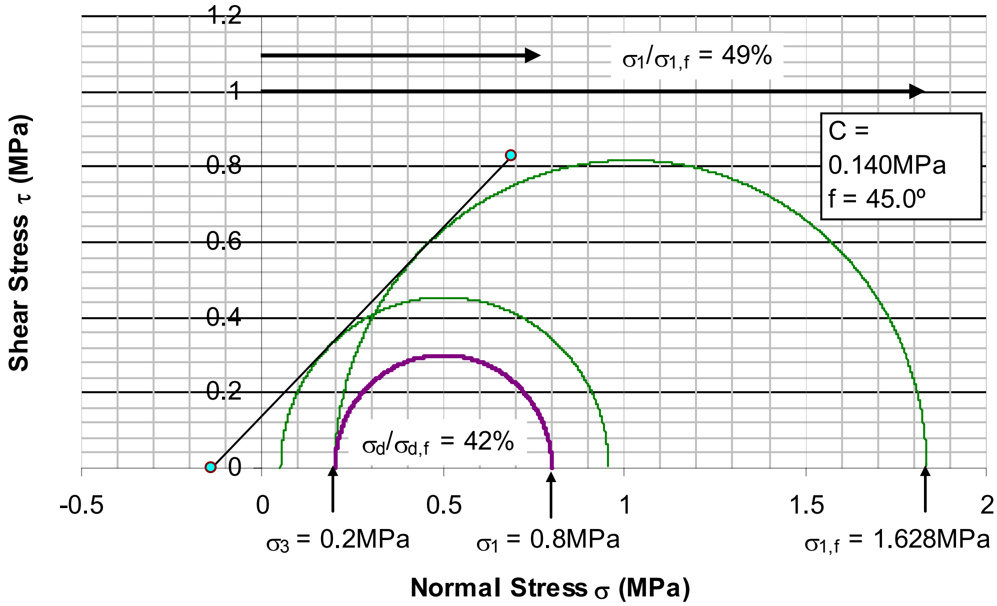


Figure 2. Graphical Illustration of Stress Ratios in the Mohr-Coulomb Space

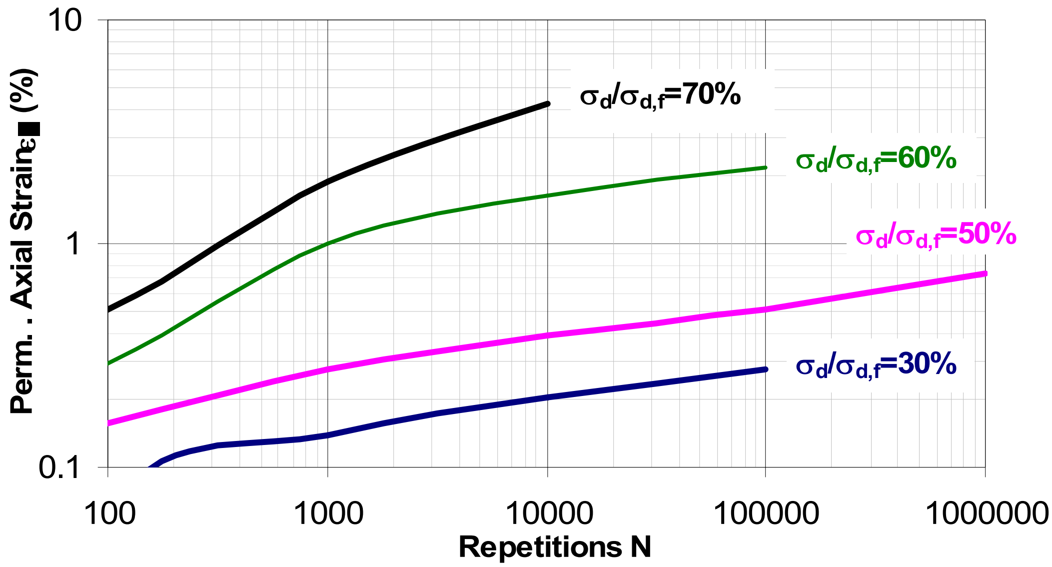


Figure 3. Permanent Deformation of Foamed Mixes, Average of All Mixes (Jenkins 2000)

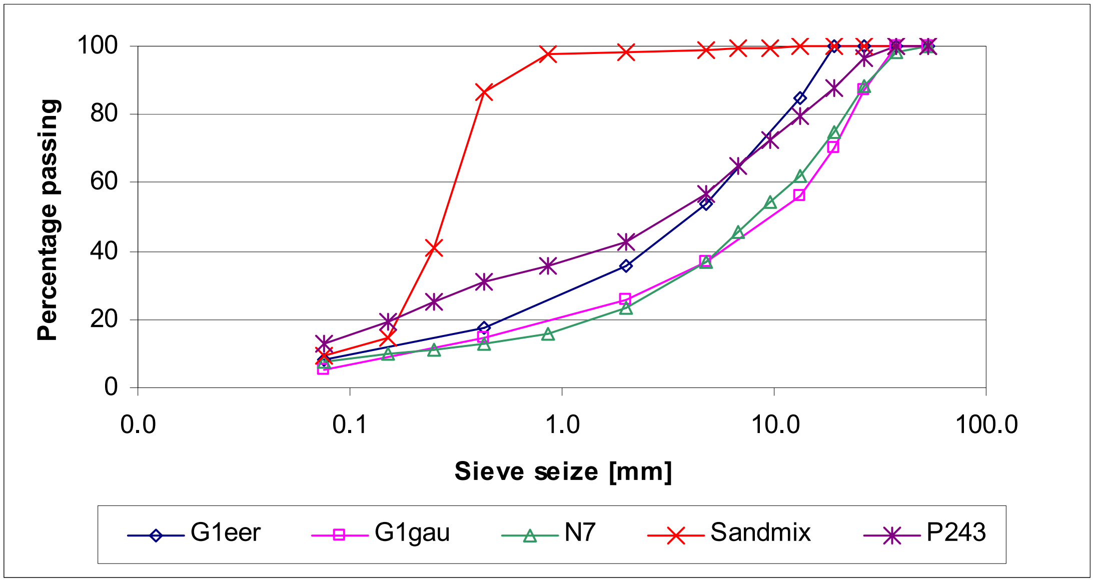


Figure 4. Grading curves of materials tested

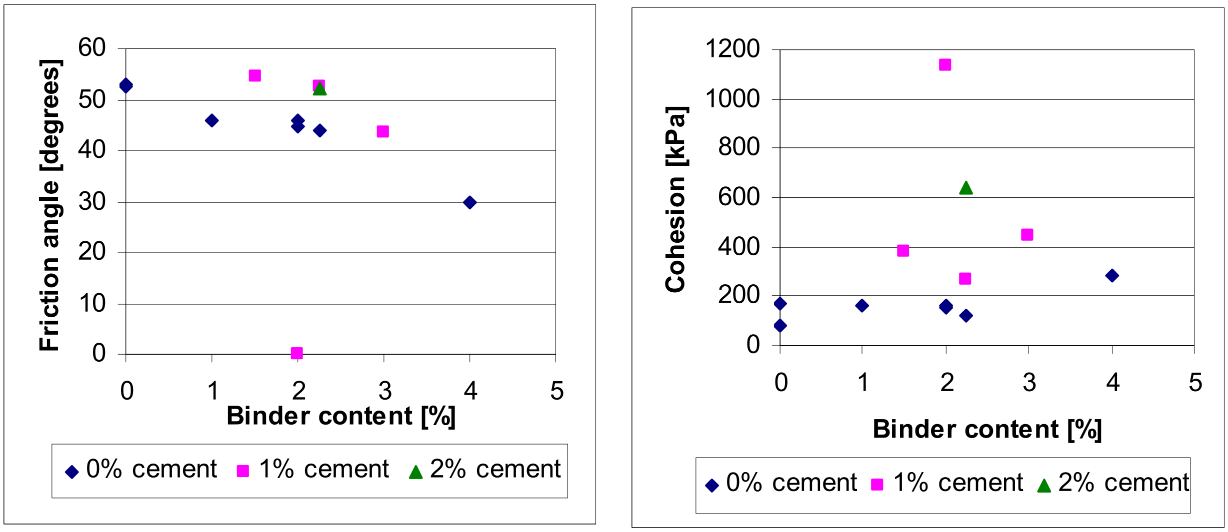


Figure 5. Graded crushed rock; friction angle and cohesion vs. foamed bitumen content

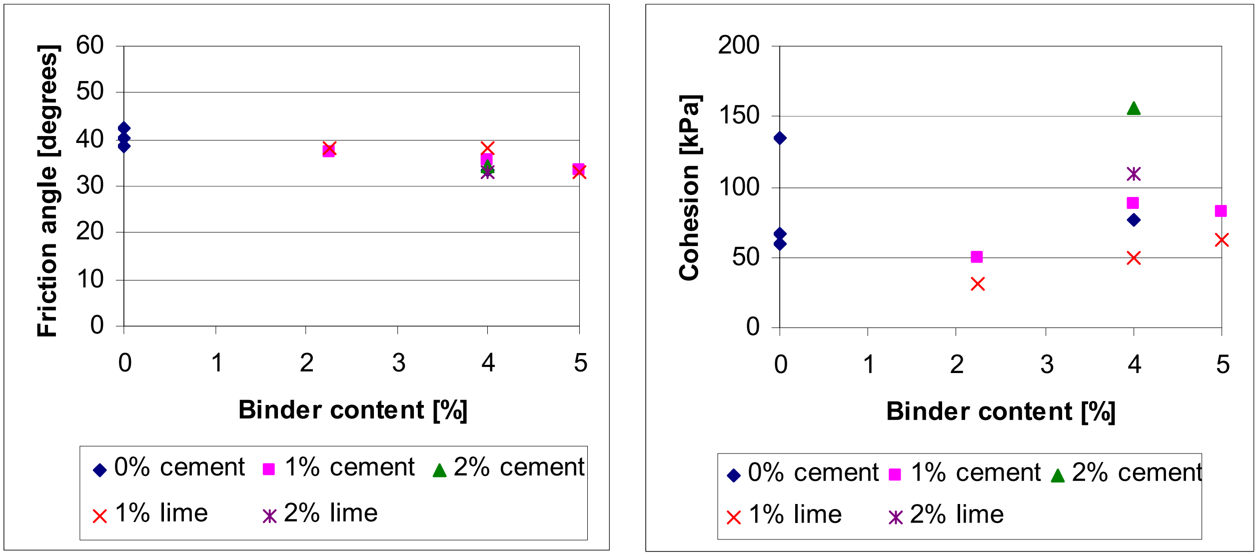


Figure 6. Sand-calcrete blend; friction angle and cohesion vs. foamed bitumen content

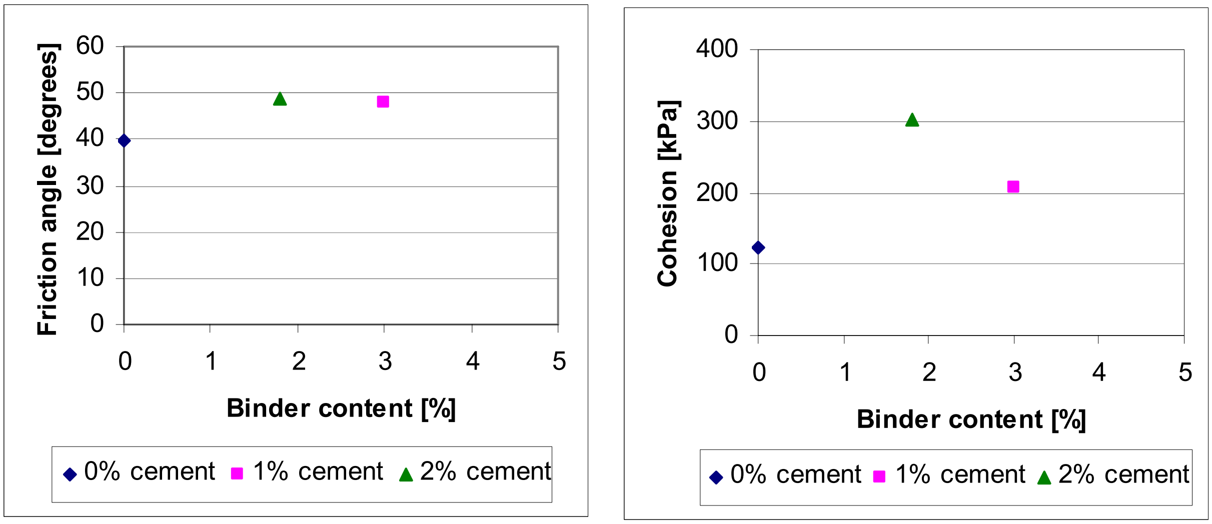


Figure 7. Ferricrete; friction angle and cohesion vs. foamed bitumen content

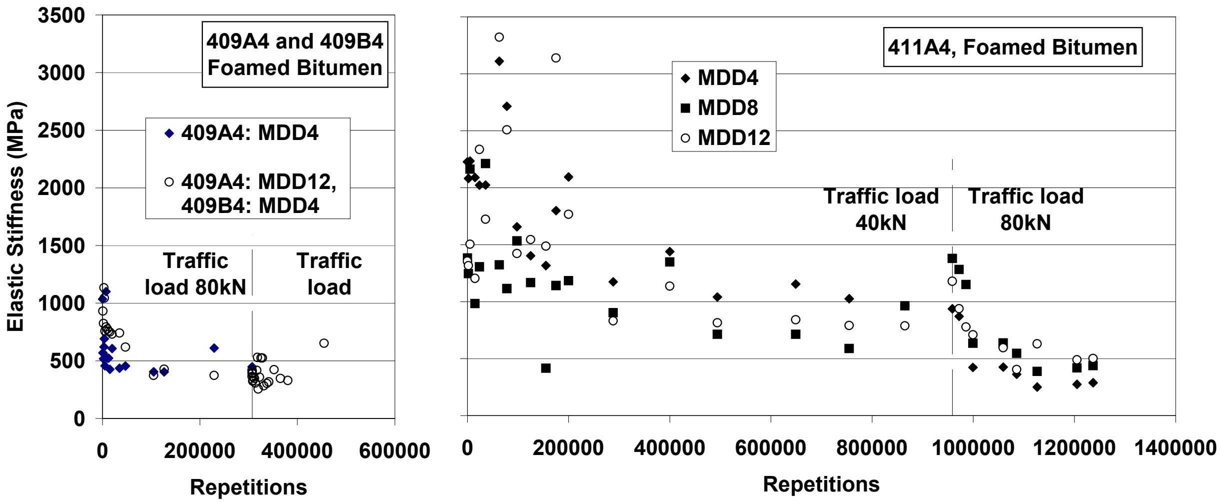


Figure 8. Resilient Modulus of Foamed Bitumen Treated Bases with 2.3% binder and 1% cement (Long and Theyse 2004)

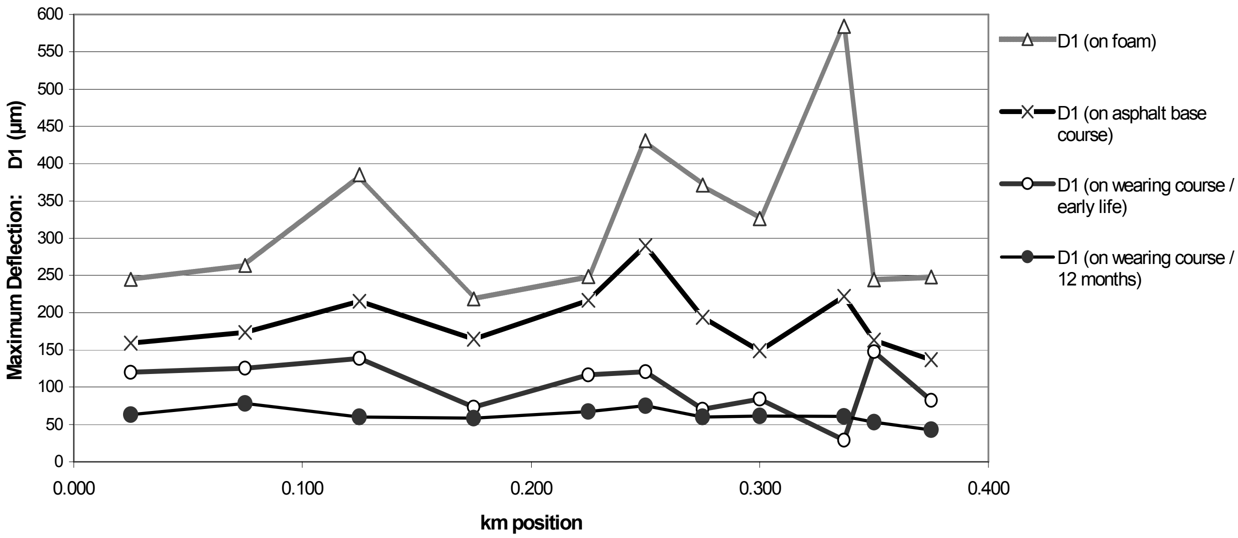
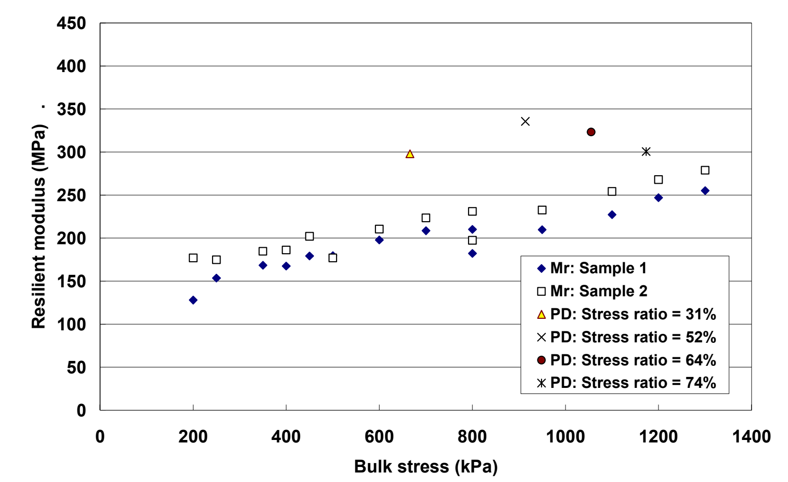
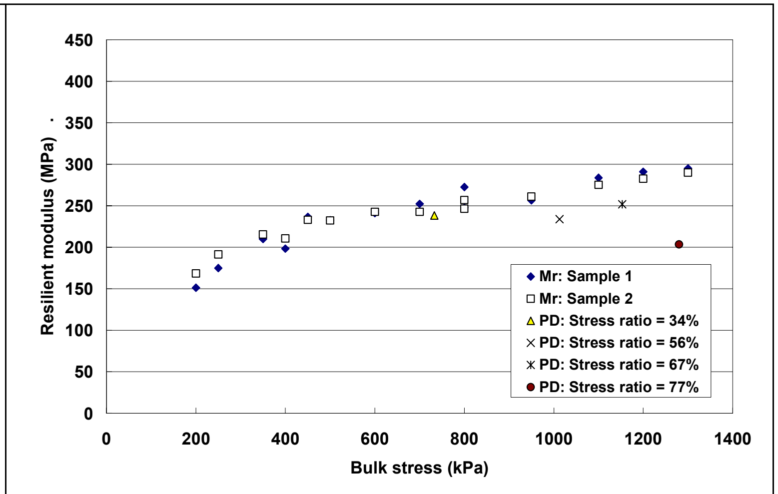


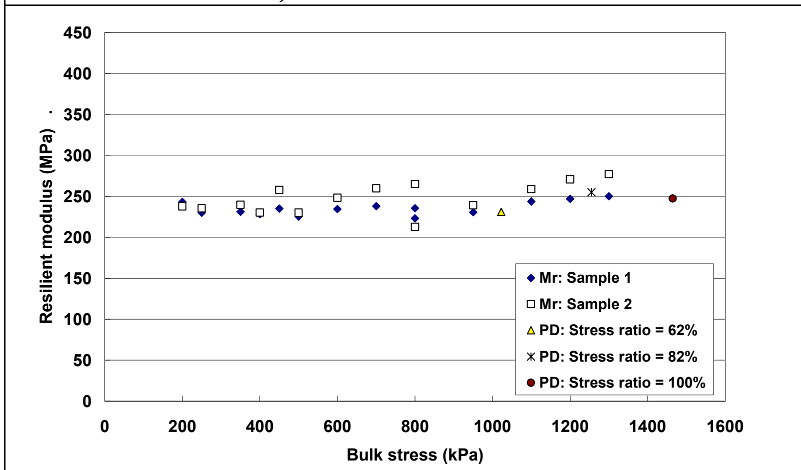
Figure 9. FWD results on a Greek Highway with 250 mm Foamed Bitumen Stabilised Base (Loizos *et al*. 2004)



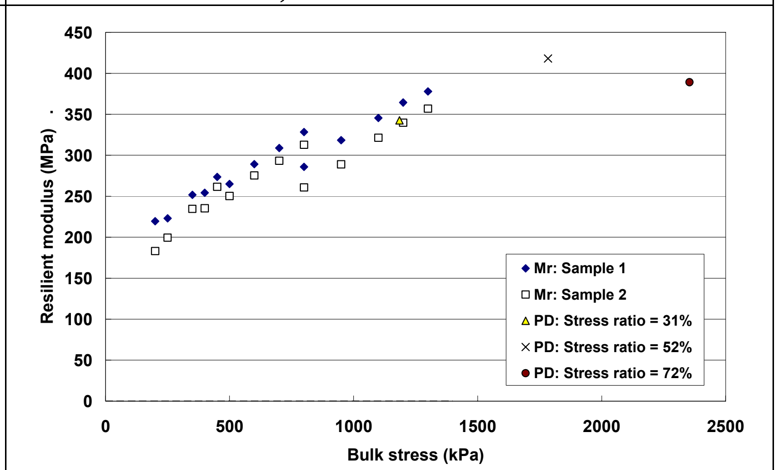
1% foamed bitumen, no cement



2% foamed bitumen, no cement



4% foamed bitumen, no cement



2% foamed bitumen, 1% cement

Figure 10. Comparison of Resilient Moduli for Foamed Eersterivier Materials

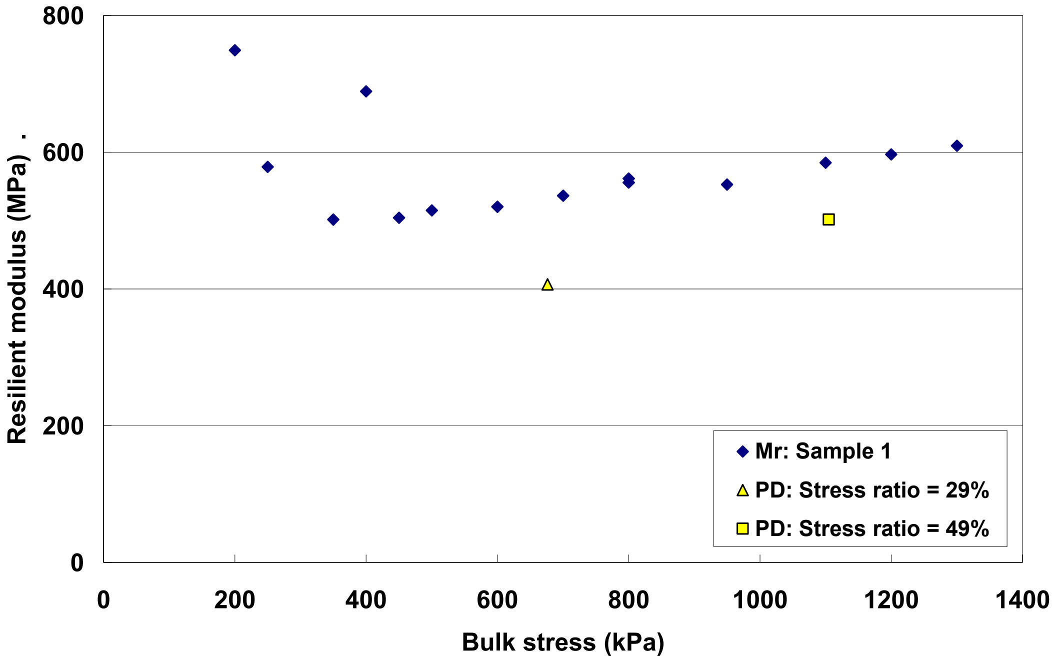


Figure 11. Comparisons of Resilient Moduli for Foamed Vanguard Drive Material with 1,5% BC and 2% Cement

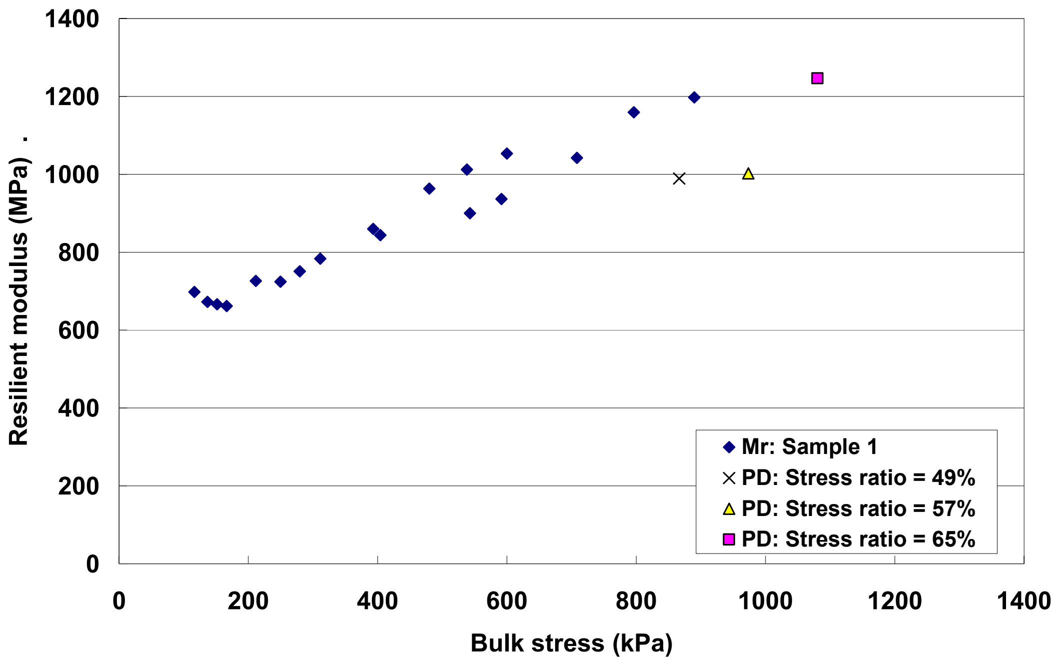


Figure 12. Comparison of Resilient Moduli Results for Foamed Mixed Granulate (MGtud) with 2%BC and no Cement

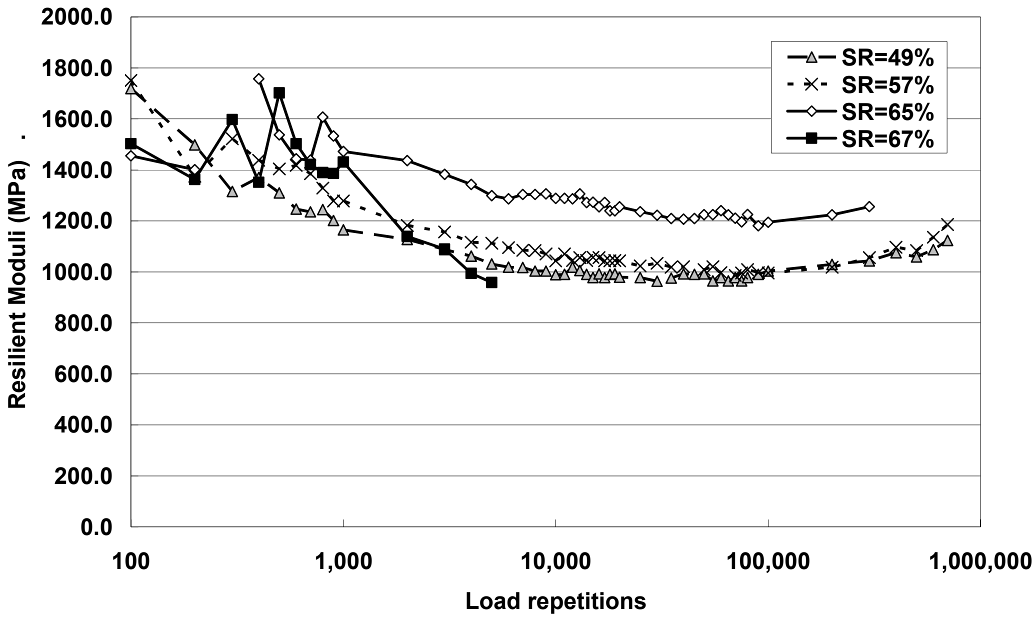


Figure 13. Stiffness reduction during permanent deformation dynamic triaxial tests, Foamed Mixed Granulate at TUD

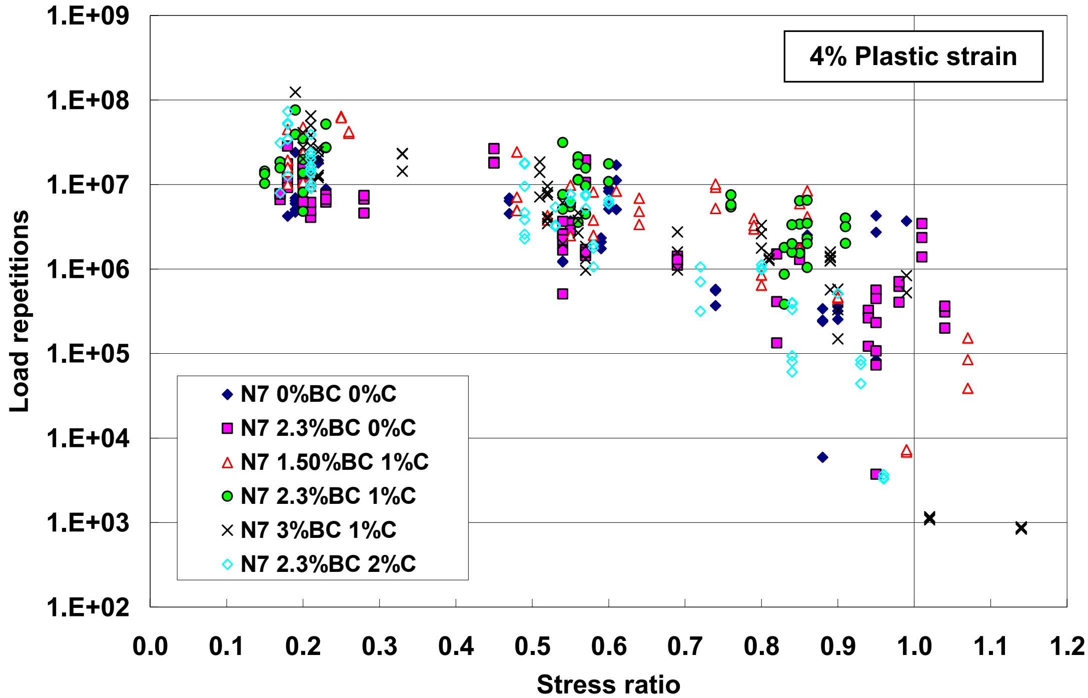


Figure 14. Stress Ratio influences from Permanent Deformation (PD) Dynamic Triaxial Tests on Foamed N7 Mixes at varying Binder Contents (BC) and Cement Contents (C), showing Load Repetitions to achieve 4% Plastic Strain (Long and Ventura 2003)

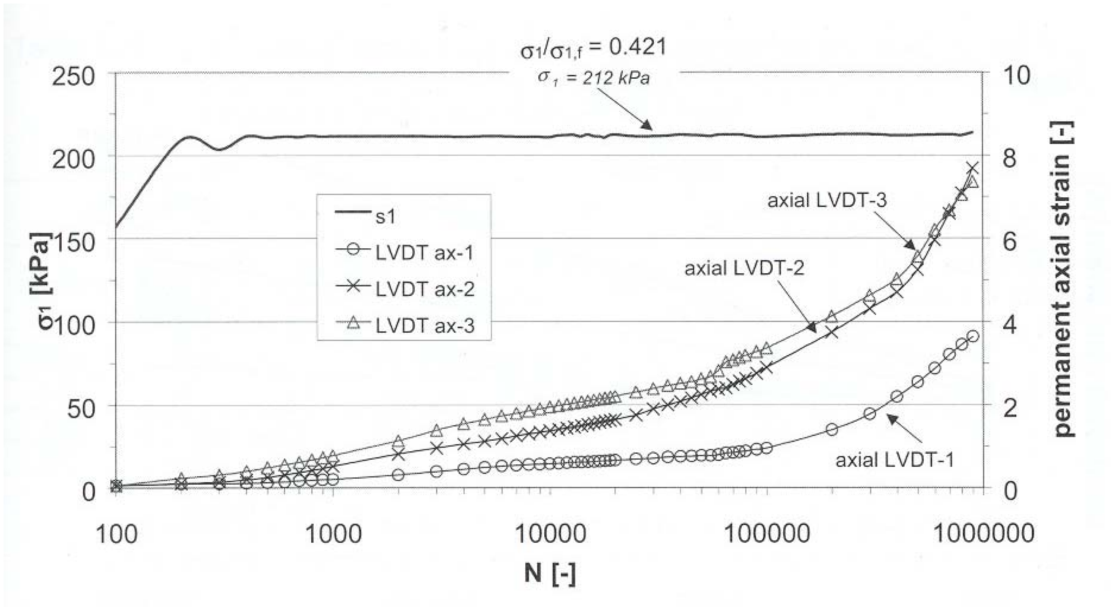


Figure 15. Permanent Deformation PD Triaxial Tests on unbound MGtud (van Niekerk, 2002)

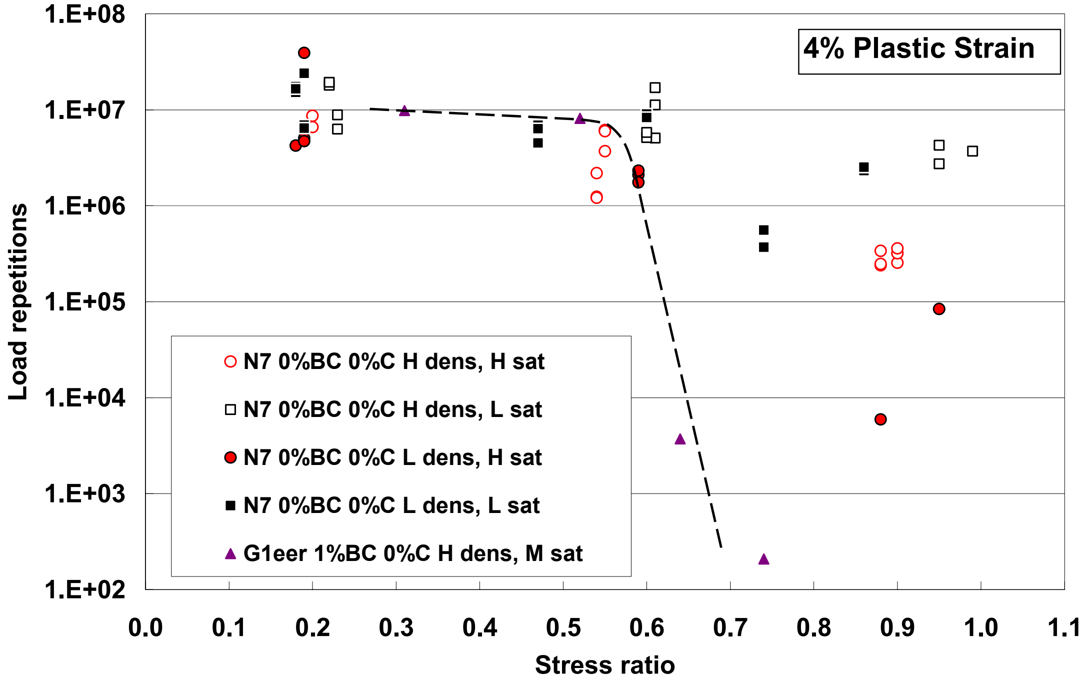


Figure 16. Stress Ratio influences on PD of Granular Materials for short duration tests (50 000 repetitions) on N7 and longer tests (>100 000 repetitions) on G1eer

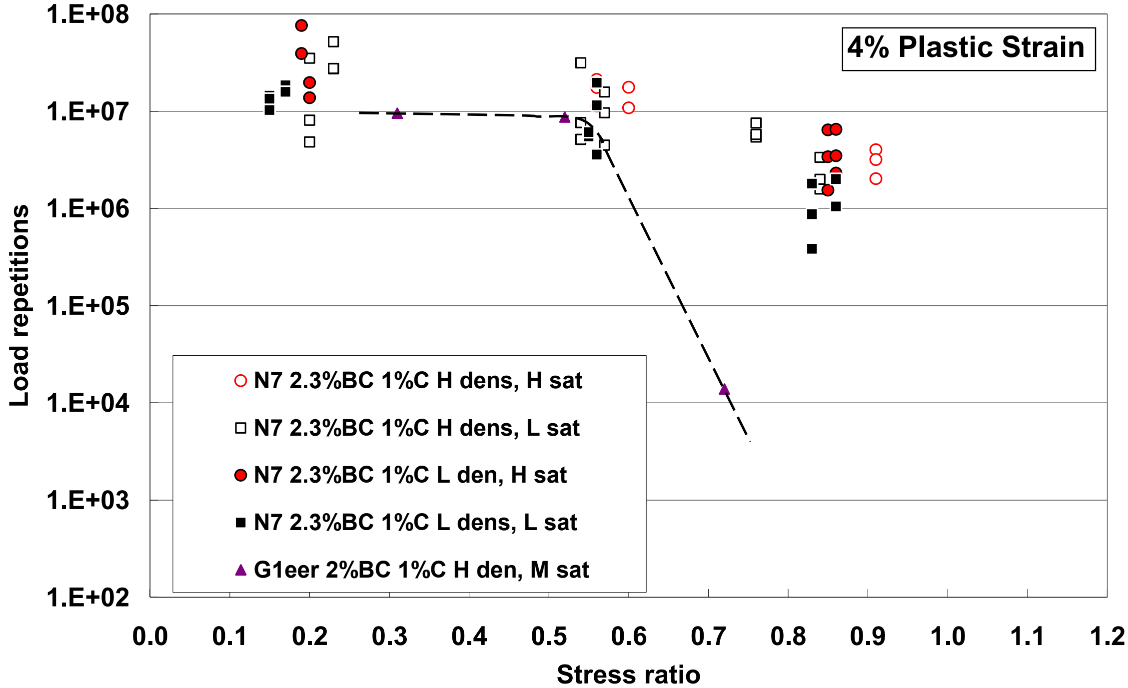
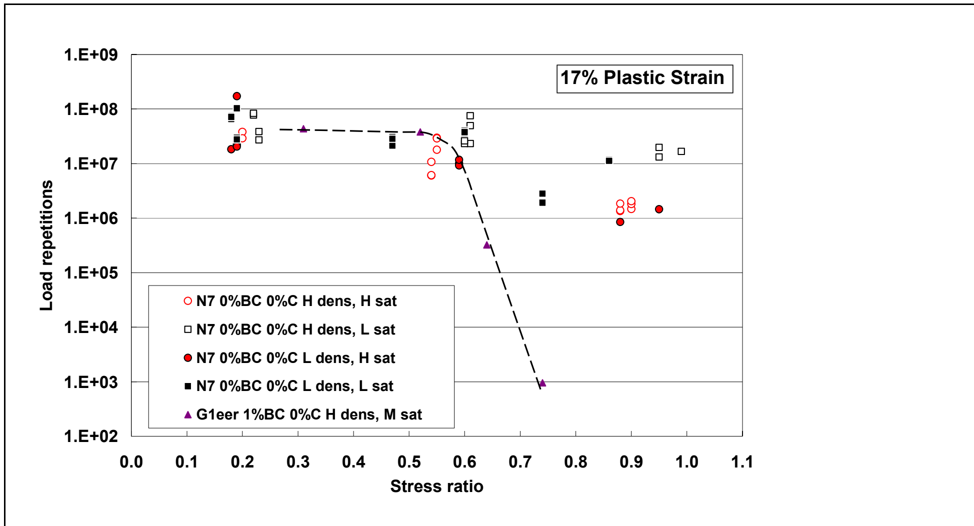
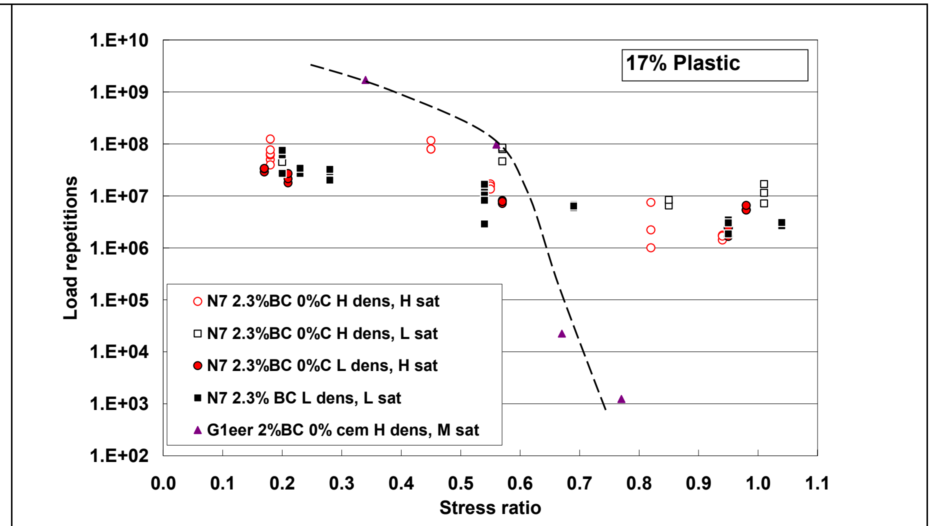
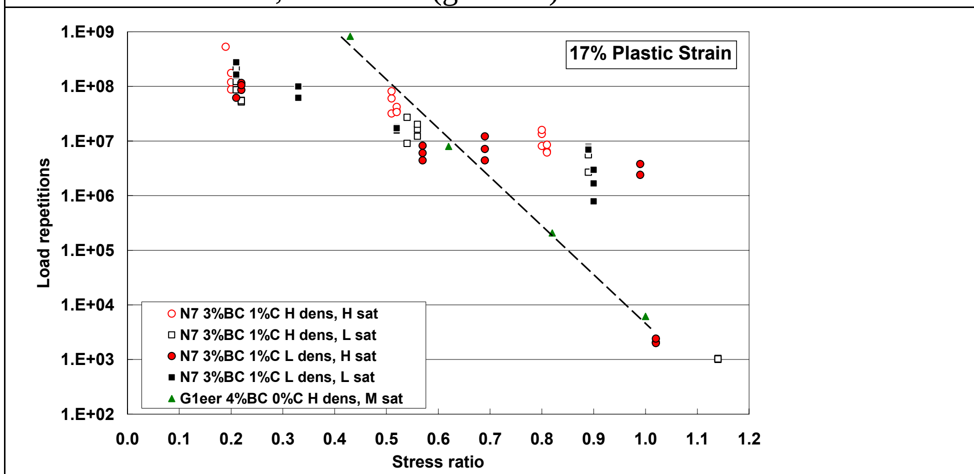


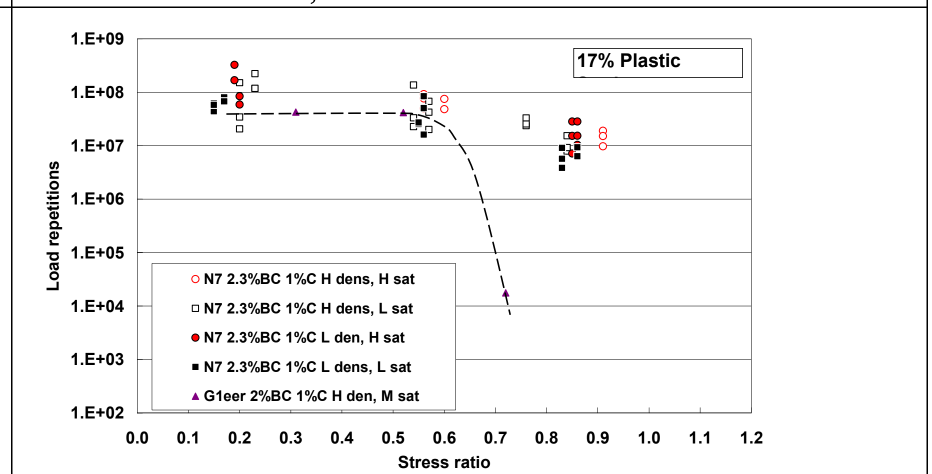
Figure 17. Stress Ratio influences on PD of Foamed Mixes for short duration tests (50 000 repetitions) on N7 material and longer tests (>100 000 repetitions) on G1eer material





No foamed bitumen, no cement (granular)	2.3% foamed bitumen, no cement





3 to 4% foamed bitumen, 0% to 1% cement	2.3% foamed bitumen, 1% cement

Figure 18. Comparison of Stress Ratios for 17% Plastic Strain in Foamed Materials N7 and G1eer

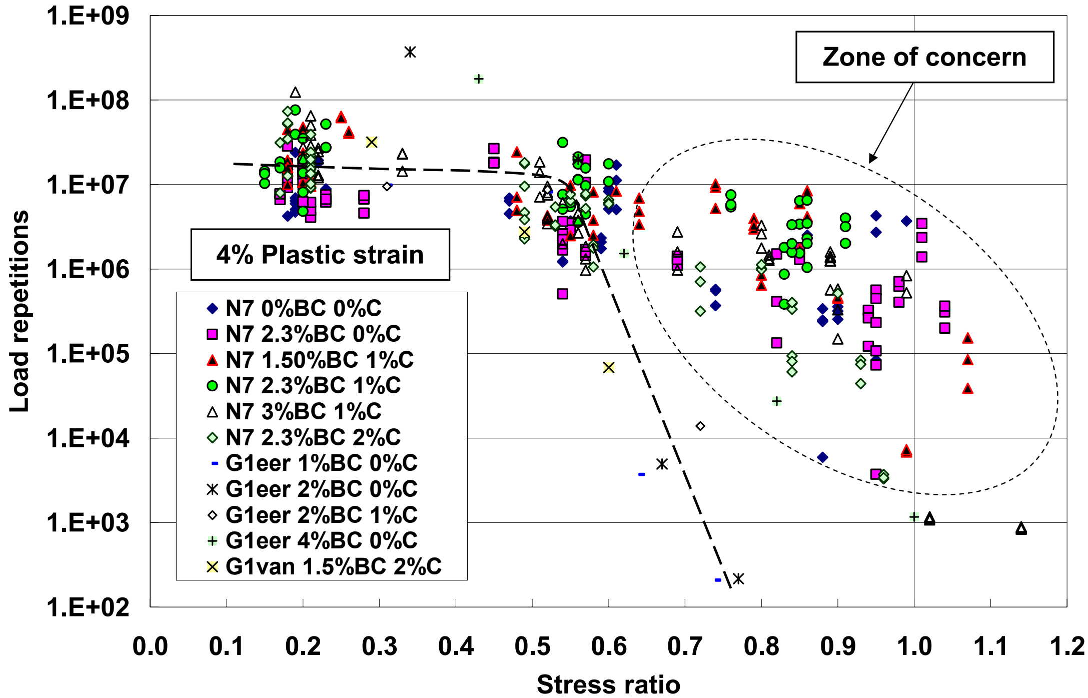


Figure 19. Collective Stress Ratio influences on PD results of 4% for Foamed Mixes using Graded Crushed Rock

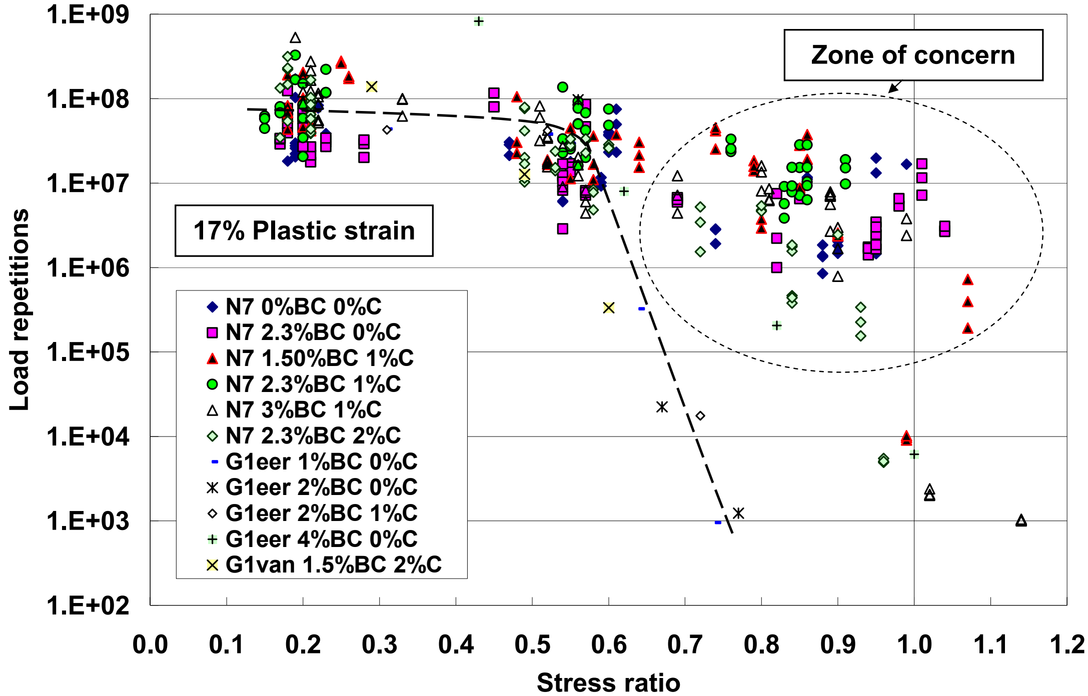


Figure 20. Collective Stress Ratio influences on PD results of 17% for Foamed Mixes using Graded Crushed Rock

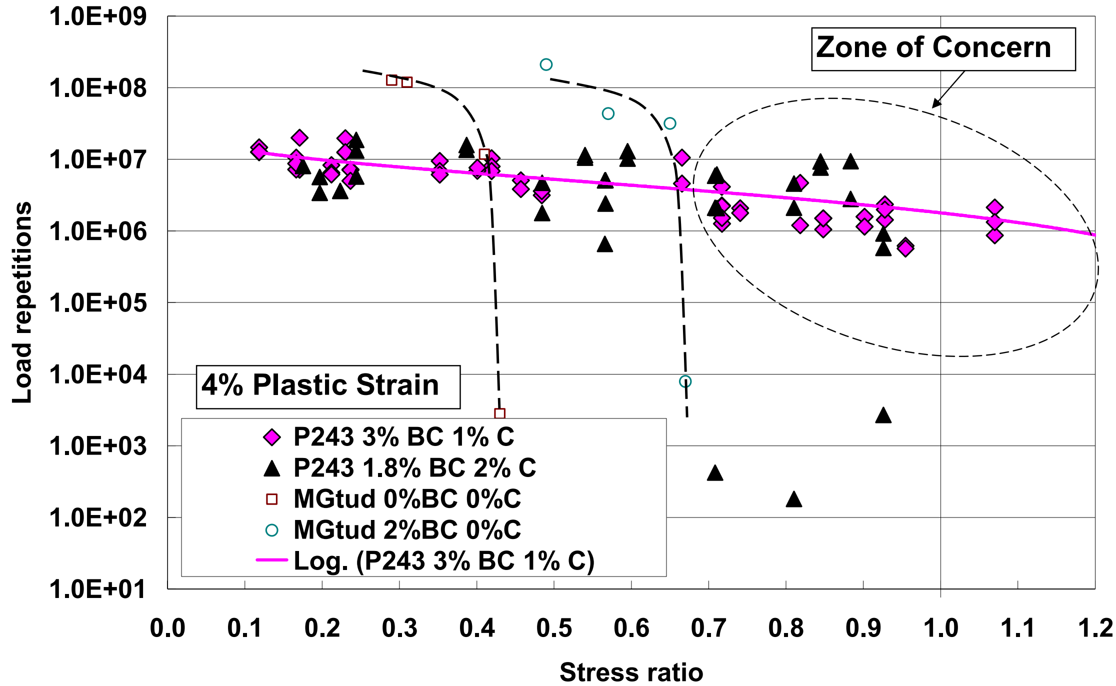


Figure 21. Collective Stress Ratio influences on PD results of 4% for Foamed Mixes using Gravel Materials, namely Ferricrete (P243) and Mixed Granulate (MGtud)