PERFORMANCE, STRENGTH AND DEFORMATIONAL CHARACTERISTICS OF FOAMED BITUMEN PAVEMENTS

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Abstract. The effects of foamed bitumen contents on the performance, strength and deformational behaviour of foamed bitumen pavements is complex and not fully understood. While some authors report increase in strength using one type of laboratory test, other authors report either only a small increase or even a decrease in strength using other types of test, thus discouraging foamed bitumen from being implemented as a cold-recycling technique for road pavement rehabilitation. This paper presents a laboratory study and a full-scale study carried out on a specific New Zealand granular material containing 1% cement and different foamed bitumen contents. The objective was to study the effects of foamed bitumen in the performance and behaviour of pavements. In the laboratory, Indirect Tensile Strength (ITS) tests, Monotonic Load Triaxial (MLT) and Repeat Load Triaxial (RLT) tests were conducted. The results showed that an increase of foamed bitumen content up to an 'optimum' content, increases the ITS but, at the same time, decreases both the permanent deformation resistance measured in RLT tests and the peak strength in MLT tests. The full-scale test on foamed bitumen pavements was conducted in the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF). Six pavement sections were tested. Three were constructed using foamed bitumen contents of 1.2%, 1.4% and 2.8% respectively, plus a common active filler content of 1.0% cement. Two more pavements were constructed adding cement only (1.0%), and foamed bitumen only (2.2%). In addition, one control section with the untreated unbound material was tested. Strains were collected using a 3D Emu soil strain system installed in each pavement section. Results showed that surface deflections decreased at sections with higher bitumen contents. After the application of 5,710,000 Equivalent Standard Axles (ESAs), the sections stabilised with cement only, bitumen only, and the control section all showed large amounts of rutting. Conversely, little rutting was observed in the three sections stabilised with foamed bitumen and 1.0% cement, showing that cement and FB together significantly improve pavement performance.

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INTRODUCTION

Pavement designers in New Zealand who are trying to use alternative materials such as foamed bitumen (FB) in new construction or rehabilitation projects are severely constrained by a lack of data on the performance of a range of stabilised materials. Although FB has been used for decades, only recently has the study on the performance of FB mixes gained focus (Jenkins, 1999). An important research effort on performance has been undertaken by some researchers at a laboratory level (Jenkins et al, 2007; Lee and Kim 2006) as well as full-scale testing of pavements (Brosseaud et al., 1997; Loizos and Papavasiliou, 2006; Long et al., 2002; Romanoschi and Metcal, 2004) but the investigations cited do not necessarily represent the particular conditions and materials used in New Zealand roads.

Between 2006 and 2008 a large research project was conducted in New Zealand to study the effects of FB stabilisation in the performance, strength and deformational characteristics of pavements. The investigation consisted of a laboratory study, in which different types of tests were conducted, and a full-scale test, in which accelerated load was applied to FB pavements.

The same aggregate, bitumen and cement were used in both laboratory and full-scale studies. In the laboratory, the granular material with a cement content of 1% was mixed with different FB contents, and later tested using Indirect Tensile Strength (ITS), Monotonic Load Triaxial (MLT) and Repeat Load Triaxial (RLT) tests. The full-scale experiment was carried out at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF), further described in this paper. Six pavements were constructed using different contents of bitumen and cement. Accelerated loading was applied to the pavement structures and the pavement responses, such as surface deformation (rutting), surface deflections and strains, were periodically recorded during the execution of the test. The strains were collected at different depths using an array of Emu strain gauges. Deflections were recorded using both a Falling Weight Deflectometer (FWD) and the CAPTIF beam deflectometer, which is a modified Benkelman beam. A total number of approximately 5,710,000 equivalent standard axles (ESAs) were applied to the pavement sections.

LABORATORY STUDY

MATERIALS TESTED AND EXPERIMENTAL PROCEDURES

Materials

The aggregates used in the laboratory study were a blend of coarse granular soils ‘H40’ and sand material (‘AP5’). The H40 is a crushed aggregate of 40 mm maximum size, with specific gravity of 2.69 t/m^3, maximum dry density of 2.22 t/m^3 and optimum moisture content (OMC) of 4.0%. The H40 aggregate particle size distribution was not directly suitable for FB stabilisation (Figure 1) because it was too coarse according to the recommended grading for FB mixes (Asphalt Academy 2002). Therefore, the H40 particle size distribution was adjusted using an AP5 crusher dust to bring the aggregate into the recommended grading zone. A final blend of 85/15 (H40/AP5) by mass was found suitable to satisfy the grading requirements for FB stabilisation. The maximum particle size of the AP5 is 5 mm, the maximum dry density is 1.82 t/m^3, and the OMC is 9.0%. No plastic fines were found in the H40/AP5 aggregate blend using the ASTM D4318-05 standard (ASTM 2005). The OMC of the H40/AP5 was 6.0%, determined following the New Zealand Vibrating Hammer Compaction test NZS 4402.4.1.3:1986 (NZS 1986).

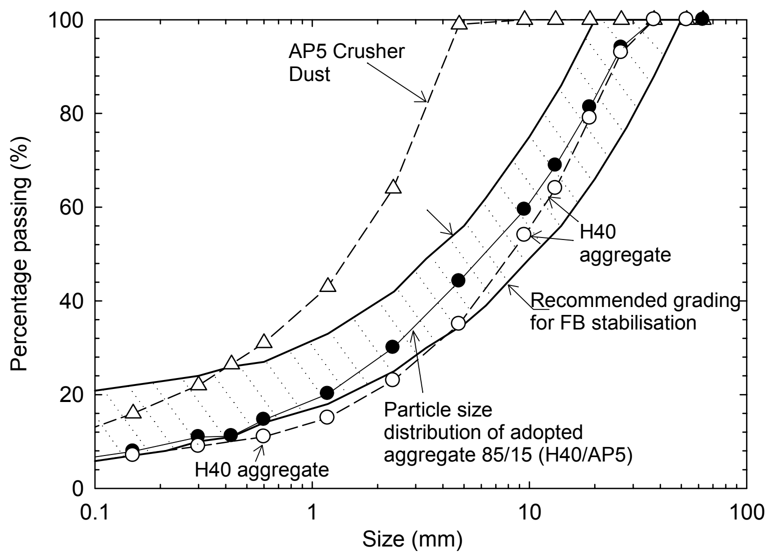

Figure 1: Particle size distribution of H40, AP5 and H40/AP5 mix adopted for foamed bitumen stabilisation

An 80/100 grade bitumen commonly used for FB stabilisation in New Zealand was used for this project. The bitumen had a penetration of 87, softening point of 48°C and viscosities of 5012 mPa.s and 485 mPa.s at 60°C and 135°C, respectively.

The characteristics of the foam were investigated using the foamed bitumen laboratory model WLB10, which has been used by many researchers and practitioners for the production of bitumen foam (e.g. Kim et al., 2008; Saleh, 2004b, among others). To produce the foam, the testing method proposed by the TG2 South African guidelines was used (Asphalt Academy 2002; Jenkins 1999). The results showed that 2.5% foaming water in combination with a bitumen temperature of 170°C yields a Foam Index (Jenkins 1999) of 128, which is considered a good foam quality (Asphalt Academy 2002), and is consistent with the properties of bitumen foams produced in New Zealand (Saleh 2004a).

Preparation, Mixing and Compaction of Materials

The H40 was separated into five fractions: 19 mm, 13.2 mm, 9.5 mm, 4.75 mm and under 4.75 mm, which were later remixed for the reconstitution of the H40 material, to minimise any variability of the aggregate in the mixing process. Aggregate sizes larger than 26.5 mm were not used in the preparation of samples to ensure that the maximum aggregate size did not exceed ¼ of the 150-mm diameter specimen. The aggregate was kept in the oven at 25°C for at least two hours before mixing with FB, because temperature has been reported to be an important factor affecting the mechanical properties of FB mixes (Gaudefroy et al. 2007; Lee and Kim 2006; Van De Ven et al. 2007).

The aggregates and the 1% cement were placed in the bowl of a mixer available for this study. The moisture content of the aggregate was raised to 85% of the OMC (5.1%) for all mixes, to avoid the introduction of another variable into the study (moisture content). The mixes were produced adding the FB produced in the WLB10 laboratory into the aggregate, producing approximately 10 kg batches of FB mixture.

Vibratory compaction was used for the preparation of ITS and triaxial specimens. This compaction method was selected because it has been successfully applied in other FB investigations (Jenkins et al. 2007; Jenkins 1999; Long and Theyse 2002a; Long and Ventura 2004) and is suitable for preparation of large triaxial specimens.

Test Methods

The ITS was determined in accordance with NZS 3112: Part 2:1986. The loading speed applied during the test corresponds to 50.8 mm/min. The specimens were prepared with 0%, 1%, 2%, 3% and 4% bitumen contents.

MLT compression tests were conducted in a triaxial cell mounted in a Humboldt Master Loader HM-3000. The strain rate was set at 2.0%/min, which is the rate commonly adopted for tests on FB mixes (Jenkins et al. 2007; Long et al. 2002; Long and Ventura 2004). The specimens were prepared at bitumen contents of 0%, 2% and 4% bitumen, and tested at different confining stresses.

The RLT tests were conducted in a triaxial cell, and the load was applied using a pneumatic loading ram, following a multi-stage stress sequence on the same specimen. The confining stress remained constant at 50 kPa for each stage, but the deviatoric stress was increased from 75 kPa up to 525 kPa, in seven increments of 75 kPa for each stage. The load frequency was 4 Hz, and 50,000 load cycles were applied in each of the seven stress stages. Twelve specimens were prepared and tested at 0%, 2% and 4% bitumen contents (four repeat tests under nominally the same conditions for each bitumen content).

RESULTS

Indirect Tensile Strength (ITS)

The results shown in Figure 2 indicate that ITS increases when the bitumen content increases up to about 3.0%, and then from 4.0%, the ITS value decreases, showing that a large percentage of bitumen may negatively influence the tensile strength of the mix for the materials studied. This is possibly because of the low fines content in the H40/AP5 aggregate that can be mixed with the bitumen, and therefore the excess bitumen acts as a lubricant and reduces the ITS of the material. An exponential curve was fitted to the results, showing an estimated optimum FB content of 2.8%.

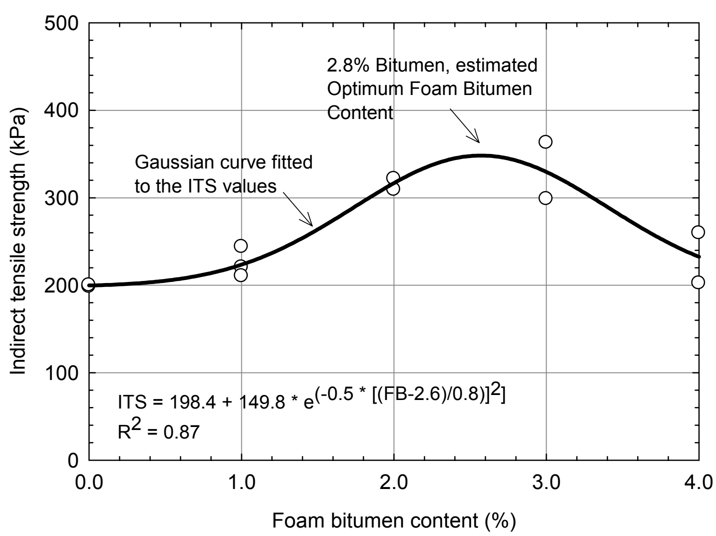

Figure 2: Indirect tensile strength test results at different FB contents

Monotonic Load Triaxial (MLT) Tests

The maximum or peak axial stress measured in MLT tests at different bitumen contents and confining stresses is summarised in Table 1. The results show a decrease in the peak axial stress of the mixes with an increasing FB content, and an increase in the peak axial stress with the confining stress, as expected for granular type materials such as FB mixes.

The peak stresses were used to model the failure envelope of the mixes in the conventional Mohr-Coulomb stress diagram (Figure 3). The Mohr circles representing the failure condition (peak axial stress state) at different confining pressures, were plotted for each of the mixes. The failure envelope was approximated using a straight line for a given FB content, which is characterised by the angle of internal friction (ϕ) and a cohesion intercept (c), as indicated in Figure 3. This linear approximation has been considered sufficient by other researchers for modelling of FB mixes (Jenkins et al. 2007). However, the estimation of cohesion by using the intercept of the failure line with the shear stress axis is not entirely accurate, because the projection of the failure envelope back to zero normal stress (σ_n=0) is uncertain, owing to the curvature of failure envelopes observed in most materials (Mitchell 1993) and, therefore, the cohesion is only an apparent value estimated from the available MLT results.

The estimated values for ϕ are shown in Figure 3. The most important effect of FB observed in Figure 3 is the reduction of approximately 50% (16°) in the angle of internal friction for every 2% of FB added to the mix. The effect of FB on cohesion is less significant, with little increase from 0% to 2% FB content, and decrease on cohesion from 2% to 4% FB content.

Table 1: Peak axial stress at different bitumen contents and confining pressures measured by MLT tests

Bitumen (%)	Cement (%)	Confining pressure (kPa)	Peak axial stress (kPa)
0.0	1.0	50	2305
0.0	1.0	50	2710
0.0	1.0	300	4003
2.0	1.0	50	1958
2.0	1.0	175	2413
2.0	1.0	175	2183
4.0	1.0	50	1298
4.0	1.0	300	1720
4.0	1.0	300	2025

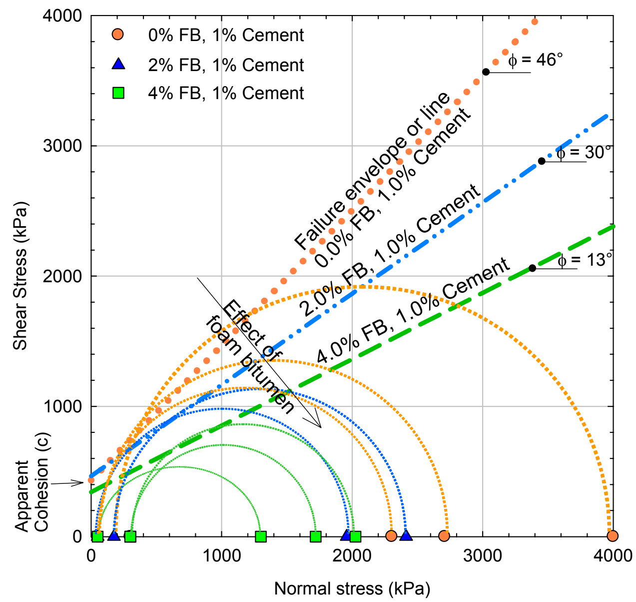

Figure 3: Mohr-Coulomb diagram: effect of foamed bitumen content on the angle of internal friction (ϕ) and apparent cohesion (c) of the FB mixes studied

Permanent Strain Triaxial (RLT) Tests

The results of the permanent deformation or permanent strain RLT tests are presented in Figure 4, showing the measured permanent strains for the seven loading stages, in which 50,000 load cycles were applied at each stage. The final permanent strain at each stage was added to the subsequent permanent deformation of the next or following stage. Note that for each bitumen content, four repeat tests under identical conditions were conducted to account for the inherent variability of FB specimens reconstituted in the laboratory.

Figure 4 shows a large variation in the permanent axial strain for a given initial condition. For example, the permanent axial strain at the end of the tests ranges between 4.8 x 10^3 and 9.2 x 10^3 μm/m for the four tests at 2% FB content. On average, however, the trend shows clearly that the measured permanent strain depends on the FB content and that the deformation increases with the FB content. In other words, when subjected to a repeat (cyclic) loading in compression, FB decreases the resistance to permanent strain of the material. These results are somewhat consistent with the MLT test results where a strength reduction was observed with an increase in the FB content, when specimens were subjected to monotonic loading in compression.

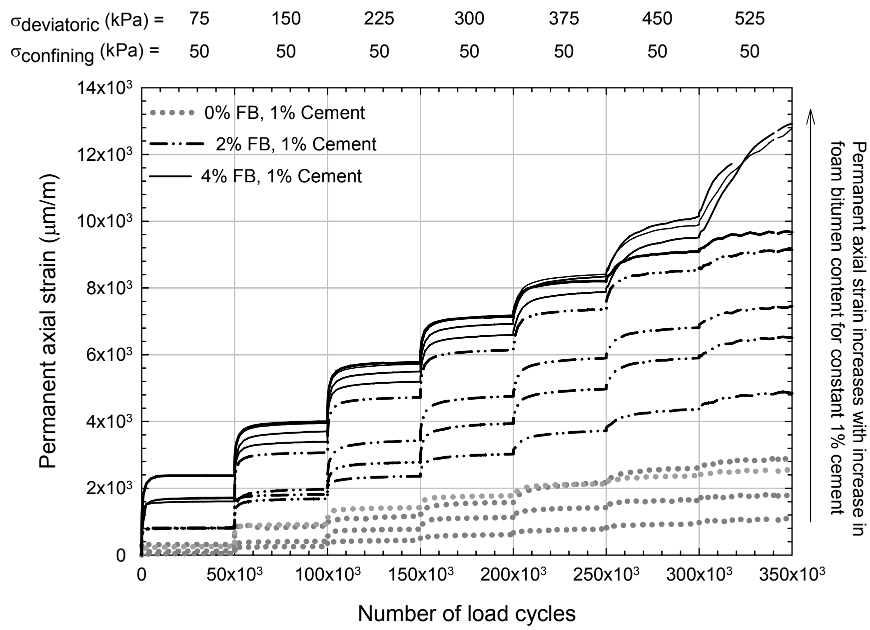

Figure 4: RLT permanent deformation tests for different FB contents

FULL-SCALE STUDY

DESIGN, CONSTRUCTION AND LABORATORY TESTING

Structural design of pavements

The structural design of pavements was carried out using the South African interim guidelines (Asphalt Academy, 2002) and the New Zealand supplement to the (Austroads, 2005) pavement design guidelines. Both utilise a mechanistic empirical approach for the design of pavements, and the design took into consideration the mechanical properties of the material having the optimum foamed-bitumen content determined in the laboratory study (2.8% bitumen, 1.0% cement).

Details of the structural design are not presented in this paper, but a 200-mm basecourse combined with a relatively weak (60-80 MPa) subgrade provided a pavement structural capacity close to 1,000,000 ESAs of 80 kN using both the design methods.

CAPTIF description and layout design

CAPTIF is located in Christchurch, New Zealand. It consists of a 58-m long circular track contained within a 1.5-m deep and 4.0-m wide concrete tank in which the moisture content of the pavement materials can be controlled and the boundary conditions are known (Figure 5a). A center platform carries the machinery and electronics needed to drive the system. Mounted on this platform is a sliding frame that can move horizontally by 1 m. This radial movement enables the wheel paths to be varied laterally and can be used to have the two ‘vehicles’ operating in independent wheel paths. At the ends of this frame, two radial arms connect to the Simulated Loading and Vehicle Emulator (SLAVE) units (Pidwerbesky, 1995).

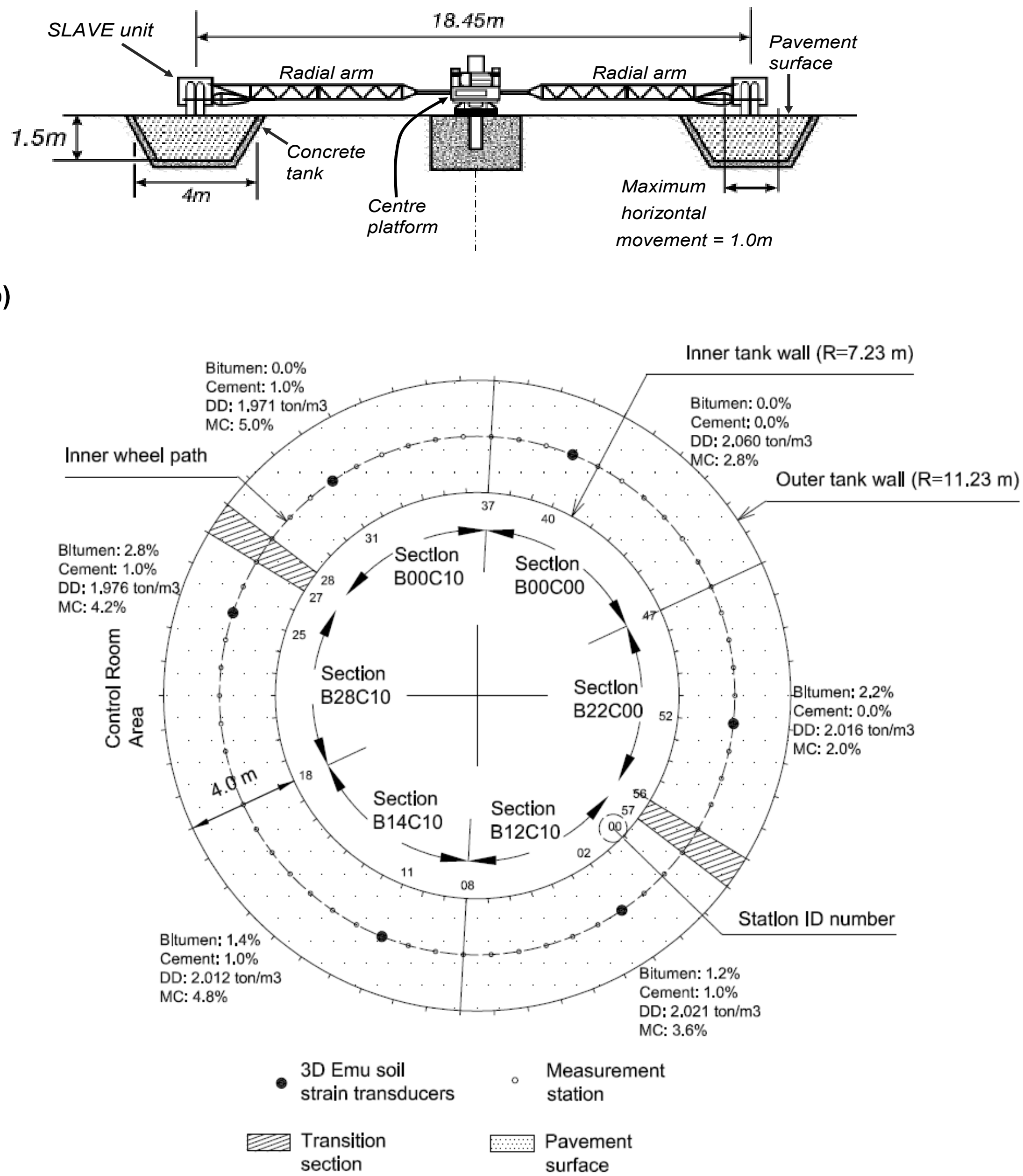

Figure 5: CAPTIF (a) cross section; (b) Top view of the foamed bitumen test (DD = Dry Density, MC = Moisture Content).

CAPTIF enables up to six pavement sections of about 10 m length each to be tested. Four sections were stabilised using 1% cement at different bitumen contents. One section was retained as a control with the H40 material only, and another section had FB only, to separate the effects of the FB with the cement. A top view of the pavement test track is presented in Figure 5b, in which the six pavement sections are depicted. The sections were named B12C10, B14C10, B28C10, B00C10, B00C00 and B22C00, where the first two digits (after B) indicate the bitumen content, and the last two (after C) indicate the cement content. For instance, section B14C10 was built adding 1.4% of FB and 1.0% cement. Section B28C10 had the 'optimum' FB content determined from ITS test.

Materials and construction

Subgrade
The top 525 mm of the subgrade was clay, extended in lifts of 225, 150 and 150 mm, and compacted using a roller available at CAPTIF (Figure 6a).

Figure 6: (a) Compaction of subgrade lifts using CAPTIF roller (b) FWD testing on subgrade

Once preparation of the three layers was completed, FWD tests were conducted to evaluate subgrade homogeneity (Figure 6b). Once the construction of the subgrade was finished, FWD deflections of the subgrade surface were measured by applying a pressure of 470 kPa (total load of 33 kN) to verify the layer homogeneity. The peak deflections showed an average of 1.510 mm with a standard deviation of 0.177 mm (Coefficient of variation, CV, 11.7%), which is considered fairly homogeneous for this type of subgrade material. A simple back-calculation analysis yields a subgrade stiffness of 60 MPa approximately, which is in agreement with the target stiffness from the structural design of pavements.

Stabilisation process
Two hundred tonnes of H40 crushed aggregates were transported to Christchurch. The material was delivered in two loads during the days before construction. About 30 tonnes of AP5 crusher dust material was imported from a local quarry.

The CAPTIF building is relatively small for large road construction machinery and therefore it was not feasible to directly stabilise the materials in place. Hence, the aggregate was blended with bitumen and/or cement outside the CAPTIF building.

For the stabilisation process, 440 mm deep trenches were excavated outside the CAPTIF building. A 340 mm layer of H40 crushed aggregate material was laid in two lifts and compacted to 95% of maximum dry density (2110 kg/m^3) at its optimum moisture content (4.0%). Later, a 70-mm thick layer of AP5 crusher dust was spread and compacted to 95% maximum dry density (1820 kg/m^3) at optimum moisture content (9.0%). The thickness ratio 70/340 yields approximately the target mass ratio of 15/85 obtained in the laboratory mix design. Once the untreated material was ready the trenches were stabilised with the recycling machine. The stabilised material was transported into the CAPTIF building (located about 50 m from the trenches) by loaders. During this process, material samples were taken for laboratory testing described later in this paper. A paver was used to place the basecourse material in two layers of 100 mm each and a steel roller was used for compaction (Figure 7a). As the CAPTIF steel roller is lighter than a roller used in normal field construction, the basecourse was compacted in two lifts to account for the lower compaction energy applied (Figure 7b). Nevertheless, the same compaction effort was applied to all stabilised pavements. The time between trench stabilisation and final compaction of the stabilised basecourse layer for one pavement section was between 2 to 3 hours.

Figure 7 (a) Paver used to spread the basecourse in CAPTIF (b) Compaction of basecourse

The construction of the control unbound granular section (B00C00) was slightly different. Instead of using a mix of aggregate and crusher dust, only the unmodified H40 aggregate material was used and laid in a single layer of 200 mm. The particle size distribution in this section was not exactly the same as the other sections, but the incorporation of the AP5 crusher dust was part of the stabilisation process.

Before laying the final surface layer, the sections were cured at ambient temperature for 30 days.

Surface layers
The surfacing was constructed 30 days after construction of the basecourse layers. All sections were sealed with a single coat chipseal. After a week to allow the chipseal to set up, all sections were surfaced with a skim coat of AC10 hot mix covering the top of the chipseal. The approximate thickness of this surface was 20 mm. Before the sealing, density and moisture measurements were taken using a nuclear gauge directly over the basecourse. Dry densities and moisture contents are included in Figure 5b.

During the execution of the experiment and after the application of 200,000 load cycles, the original thin surface started to show wear. A thin hot mix asphalt (HMA) layer of 30 mm was laid over the original surface, leading to a total surface thickness of approximately 50 mm for the rest of the project.

Instrumentation
The pavement instrumentation at CAPTIF includes 3D Emu (Dawson, 1994) soil strain transducers to measure the vertical, transverse and longitudinal strains in the pavement. The soil strain measuring system determines strains with good resolution (±50 μm/m). Details of the system can be found in Steven's thesis (Steven, 2005). The strain coils were installed during the formation of the subgrade and the basecourse layers, to minimise the disturbance to the materials.

The Emu strain coils were located coaxially at a spacing of 75 mm, directly under the inner wheel. The reported depth of the vertical strains corresponds to the midpoint between two coils, while the reported depth of the longitudinal and transverse strains corresponds to the coil depth. The 3D Emu stacks were located at stations 2, 11, 25, 31, 40 and 52 (Figure 5b). A transverse profile of the final pavement structure, the placement of the wheels, and the location of the coils are presented in Figure 8.

Other measurement systems used at CAPTIF during testing are an FWD, the CAPTIF deflectometer (which is a modified Benkelman beam), and a transverse profilometer.

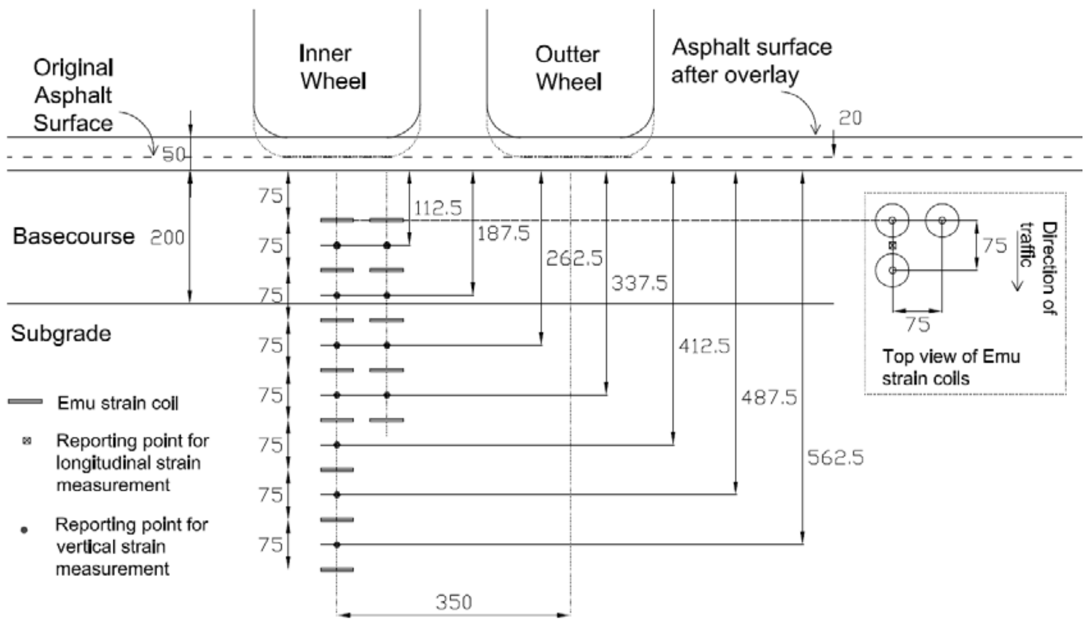

Figure 8: Instrumentation and cross-section profile for CAPTIF foamed bitumen experiment

EXPERIMENTAL PROCEDURE

Loading sequence and speed

The load was a dual truck tyre with a separation of 350 mm between the centers of the tyres inflated to 700 kPa. The original project intended that a constant loading of 40 kN would be applied for each SLAVE unit. However, since little rutting was measured during the early stages of the project, the load was increased to 50 kN at 150,000 load cycles. At 502,000 load cycles the load was increased again to 60 kN to induce a failure in the pavement sections.

The speed of the vehicles was kept constant at 40 km/h during most of the project. The load was applied on one wheel path with a lateral wandering of 100 mm. During the strain measurements the lateral movements of the wheels were restrained to ensure that the wheels rolled over the strain instrumentation, as shown in Figure 8.

Data collection

The strain measurements were taken from the beginning to the end of the test at several stages, at SLAVE speeds of 10 and 40 km/h. Vertical surface deformations were taken at each station at the same intervals. The rutting that was measured after the construction of the HMA overlay was added to the initial rutting measurements to calculate the total rutting.

The FWD testing was carried out before the trafficking (0 load cycles, Figure 9a) and after the application of 1,000,000 load cycles (Figure 9). The initial testing was conducted at a standard 40 kN load at different transverse and longitudinal locations of each pavement section.

Figure 9 (a) FWD testing before trafficking (b) FWD testing after the application of 1 million load cycles

CAPTIF modified beam tests were conducted during the initial phase of the project (0 to 35,000 load cycles) and during the last stage (502,000 to 1,326,000 load cycles), applying SLAVE loads of 40 kN and 60 kN respectively. The speed of the wheels during the beam test was about 6 km/h.

PAVEMENT TEST RESULTS

Rutting

The averaged rutting measurements for each pavement section are presented in Figure 10a. In this figure, the number of load cycles has been converted to ESAs of 80 kN in the second x-axis, assuming a fourth power law. The curves presented show the typical behaviour of pavements with a bedding-in phase during the initial vehicle loading, followed by a plateau phase. When the load was increased to 50 kN another increase in the rutting rate was observed. From 300,000 load cycles (after constructing the HMA overlay) the rutting increased approximately linearly up to 1,000,000 load cycles for all sections.

After 1,000,000 load cycles, sections B00C10, B00C00 and B22C00 started to show large amounts of heaving and rutting, while sections B12C10, B14C10 and B28C10 performed well, and little difference among them was observed.

Deflection tests

CAPTIF beam and FWD deflections are presented in Figure 10b. The results correspond to deflections measured before the trafficking of the sections (0 load cycles), and for which a load of 40 kN was applied in both deflection tests. The trends observed at this stage were relatively constant throughout the CAPTIF test. The beam and FWD provided similar results, illustrated by parallel curves fitted to the results. In general, FWD deflections were lower than beam deflections, and this could be caused by the shorter load pulse applied by the FWD.

The unbound section (B00C00) shows considerably higher deflections in comparison with the other sections. Both cement and bitumen have an important effect in reducing the deflections of the pavements studied, as shown by the lowest deflection recorded in the section at the highest bitumen content and 1% cement (B28C10).

Strain measurements

Only two sets of vertical strain measurements are presented in this paper, and they were taken after the application of 502,000 and 1,326,000 load cycles. The wheel load applied in this part of the test was 60 kN and the vehicle speed for the results presented here is 10 km/h (see the loads in Figure 10a). The trends observed at other stages of the test remained similar, indicating that the modulus of the stabilised pavements remained relatively constant during the experiment. This contradicts other full-scale experiments on FB pavements (Long et al, 2002; Asphalt Academy, 2002) where an important reduction in the modulus has been reported. However, in those experiments the cement content (2.0%) was higher than the FB content (1.8%) and therefore the behaviour of the pavements is not representative of the materials studied in the CAPTIF test.

To better examine the effect of FB the measured strains were plotted against the bitumen contents. In Figure 11 are presented the compressive vertical strains measured in the basecourse (depth=112.5 mm), the tensile longitudinal strains measured close to the bottom of the basecourse layer (depth=150 mm), and the vertical compressive strain close to the top of the subgrade (depth=262.5 mm). The locations of these strains within the pavement structure have been recognised as critical points by the current design methods for FB pavements (Asphalt Academy, 2002; Austroads 2005).

The vertical strains (Figure 11a) measured at sections with 1% cement were considerably lower than those of the other two sections (B00C00 and B22C00). Measurements also indicate that FB has a small effect in the compressive vertical strains measured in sections at 1% cement. Figure 6b illustrates that cement has a large effect in the reduction of the longitudinal strains. Also, these strains are reduced by 50% when FB is added. The compressive vertical strains measured at the top of the subgrade (Figure 11c) of the control section (B00C00) were considerably higher than the other sections. In the other sections the general trend was that these strains were lower at higher bitumen contents.

(a)

(b)

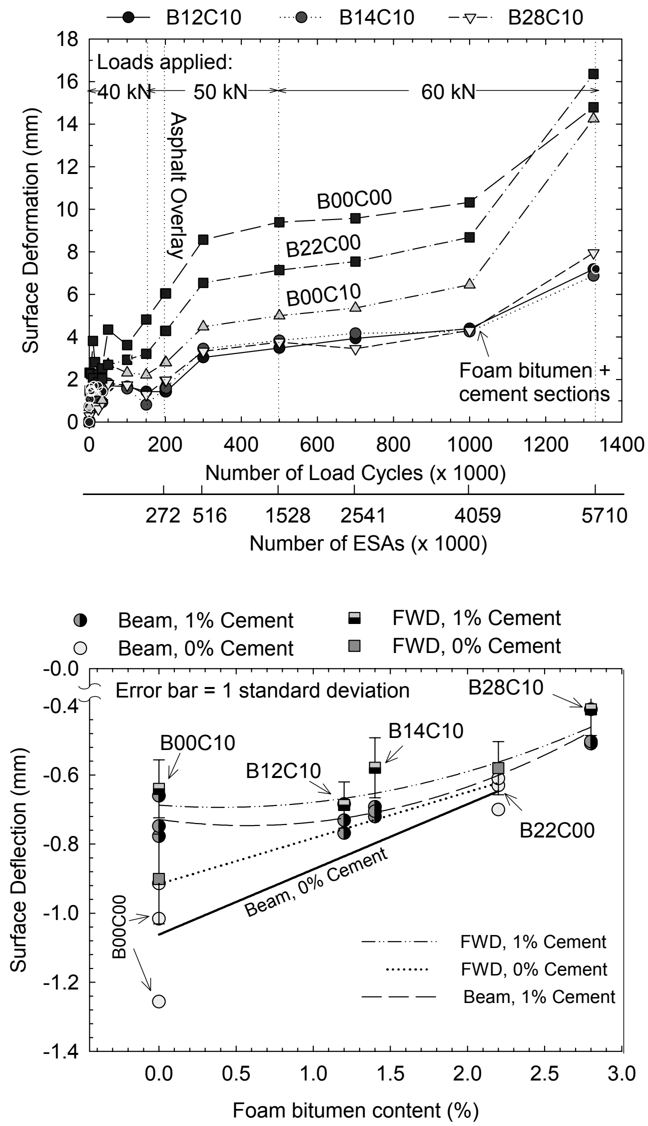

Figure 10: Pavement responses: (a) Rutting; (b) CAPTIF beam and FWD deflections at 40 kN before trafficking (0 load cycles).

(a)

(b)

(c)

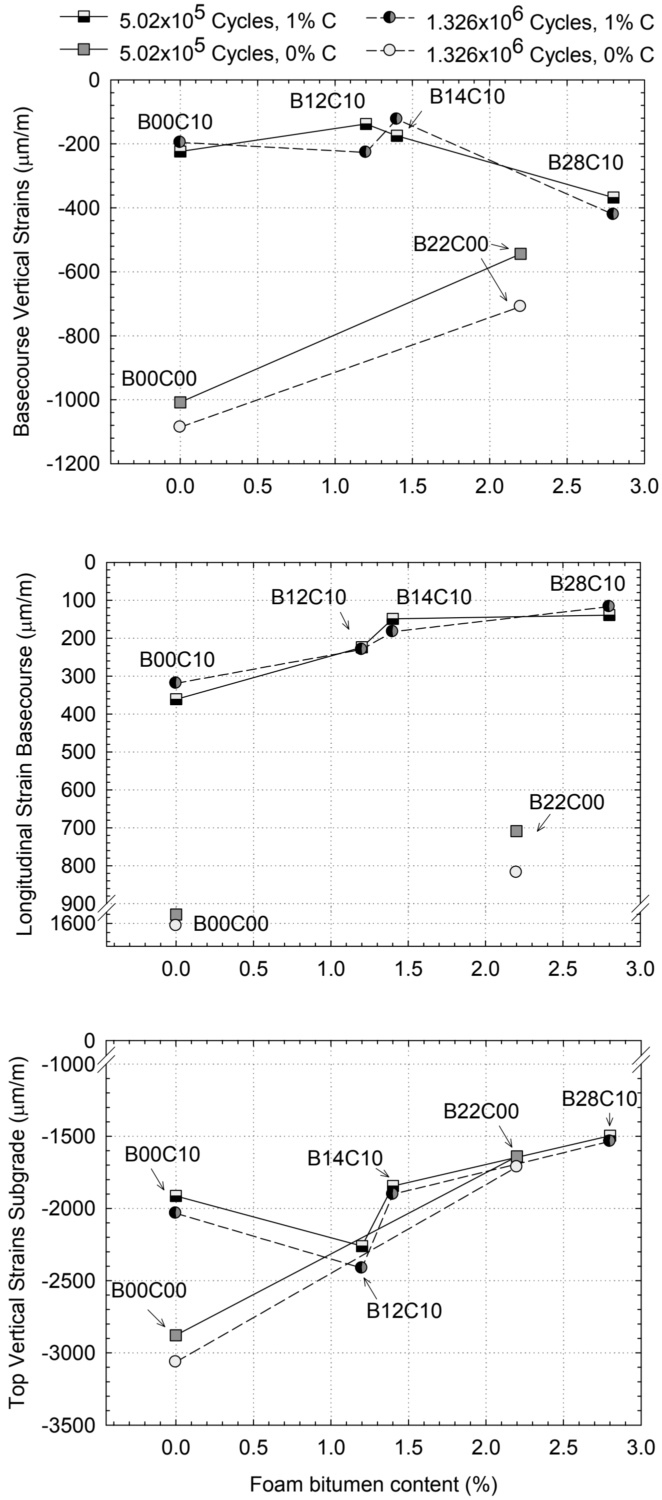

Figure 11: Strain measurements versus bitumen content at 502,000 and 1,326,000 load cycles: (a) vertical compressive strains in the basecourse; (b) longitudinal tensile strains in the basecourse; (c) vertical compressive strains at the top of the subgrade.

DISCUSSION OF RESULTS

The pavement test results presented indicate that stabilisation using FB and cement improved the performance of the pavements. The rutting of sections B12C10, B14C10 and B28C10 was consistently lower than in the other three sections.

Because the basecourse was placed over a fairly weak subgrade, the tensile behaviour (related to indirect tensile tests) had a dominating effect on the actual pavement behavior. A simple multi-layered linear elastic model of the pavements studied showed that only the upper 25% of the basecourse is loaded under a stress condition comparable to the RLT permanent deformation stress conditions (vertical and horizontal compressive stresses). The middle of the basecourse (where the basecourse strain coils were located, Figure 3) was affected by a combination of low compressive and tensile horizontal stresses, while the bottom of the basecourse layer is under compressive vertical stresses and horizontal tensile stresses.

The vertical strains measured at the top of the subgrade (Figure 6c) follow a similar trend to those of the deflections (Figure 5b) indicating the large effect of the subgrade on the elastic response of the pavements. The lowest deflection was measured in section B28C10, caused by the reduction of the horizontal strains at higher bitumen content.

The laboratory tests showed contradicting trends for materials with 1% cement. The ITS results for 1% cement indicate that at higher FB contents the strength of the materials increases. This is the normal behaviour of FB mixes for which strength increases to an optimum bitumen content, after which the ITS value drops (Ruckel et al, 1983; Kim and Lee, 2006). Similar trends are observed from indirect tensile resilient modulus tests (Nataatmadja, 2001; Saleh, 2004b). Conversely, permanent deformation RLT specimens at 1% cement show an opposite trend to those of the ITS results, in that the higher the bitumen content, the higher the final permanent deformation. Similar results have been reported by other authors (Long et al, 2002; Long and Ventura, 2004) who, along with others (Frobel and Hallet, 2008; Browne, 2008), observed that Unconfined Compressive Strength (UCS) test results showed the FB to have a minor effect on the compressive strength of materials with cement. This trend was confirmed with the results obtained from MLT tests. The RLT and MLT results thus contradict the full-scale tests, in which performance was improved with FB content. This suggests that the stress conditions in a real pavement are much more complex and different from those applied in a particular laboratory test.

The pavement in the field is dynamically loaded by moving wheel loads, which at any time impose variable magnitudes of vertical, horizontal and shear stresses including rotation of the principal stresses depending on the particular location of the considered element relative to the point of load (Figure 12). This complex stress state may induce compressive lateral stresses in the upper part, while generating tensile lateral stresses in the lower part of the layer, depending on the axle configuration, load and pavement stiffness. This distribution of stresses together with the deformation and strength characteristics of FB mixes tested in the laboratory study under different stress conditions influence the performance of FB pavements. Therefore, laboratory test results should be interpreted correctly and this research suggests that further analysis is required to understand the benefits of FB stabilisation.

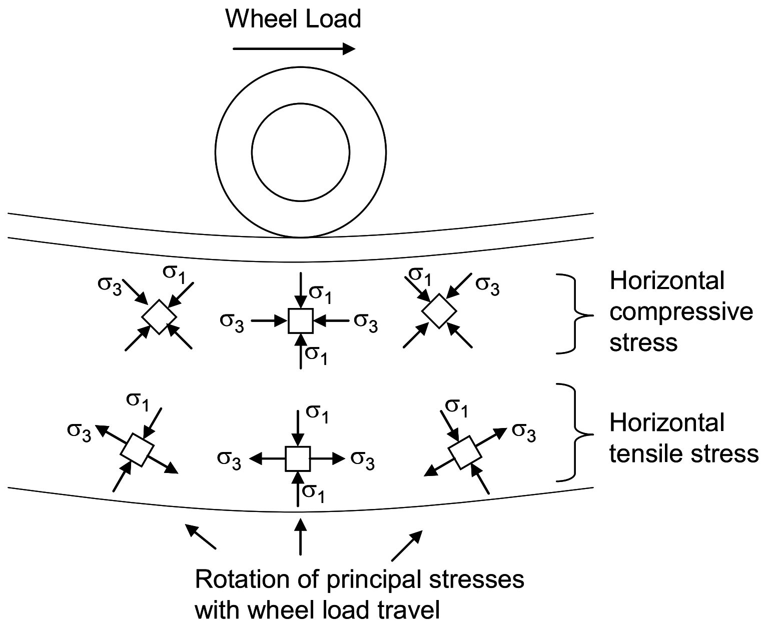

Figure 12: Stress state of a pavement under moving wheel load

CONCLUSIONS

The effects of foamed bitumen (FB) content on the performance, strength and deformational characteristics of pavements were presented in this paper.

Results from the laboratory study lead to the following findings:

- The addition of FB increases the indirect tensile strength (ITS) of foamed bitumen mixes up to an optimum bitumen content of approximately 2.8%, at which ITS reaches the maximum value.

- The results obtained from MLT tests shows that the addition of FB reduces the peak axial stress attained in triaxial compression.

- The shear strength properties of the mixes (i.e. angle of internal friction and apparent cohesion) were estimated using MLT test data, indicating a reduction in the angle of internal friction (ϕ) with FB content. The effects of FB content on the apparent cohesion were found to be relatively small for the investigated mixes.

- In the repeat load triaxial tests (RLT), the addition of FB increased the cumulative permanent strain of the mixes, creating a relatively weaker material as compared to that without FB. In this regard, both MLT and repeat load triaxial tests were consistent as they showed an increase in the permanent deformation of the material with increasing FB content.

Results from the full-scale study lead to the following findings:

- The rutting measured in sections B12C10, B14C10 and B28C10, after the application of 1,326,000 load cycles (5,710,000 ESAs) was the lowest, showing that the addition of foamed bitumen significantly improved the performance of the materials with 1% cement that were studied in this research. The sections B00C10 and B22C00 and the control untreated section (B00C00) showed large amounts of rutting and heaving by the end of the test. Little difference was observed within the sections stabilised with foamed bitumen and 1% cement.

- Pavement section B28C10 was designed to carry 1,000,000 ESAs of 80 kN with two design methods. However, after the application of 5,710,000 ESAs little rutting was observed, indicating that current design methods for foamed bitumen pavements are over-conservative.

- The deflections of section B28C10 were lower than those of the other sections, while the untreated section (B00C00) showed the largest values.

- The general results suggest that the stress conditions in road pavements in the field are more complex and involve both compressive and tensile stresses depending on the particular location in the basecourse layer. These stress conditions together with the strength and deformational characteristics presented herein need to be considered in order to fully understand the contribution of FB to the improved performance of pavements.

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