Analysis of Accelerated Model Pavement Testing for the Characterization of Pavement Materials

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This study is focused on characterizing lightly stabilized materials under accelerated traffic loading and therefore to predict the behaviour of pavements involving such materials. The Accelerated Model Pavement Testing (AMPT) setup for this research consisted of a 1175 mm diameter tank with 150 mm of lightly stabilized base layer on top of a 600 mm of subgrade layer. For the first test a hydraulic ram was used to exert a sinusoidal stress pulse of 1500 kPa at 3 Hz onto the compacted pavement layers. For the second test the same hydraulic ram was used but the stress pulse was initially started at 750 kPa at 3 Hz and was increased by 250 kPa each week up to 1500 kPa. Two Linear Variable Differential Transformers (LVDT) were setup to measure the deformation at the surface, at half depth of the base layer, and at the bottom of the base layer. There were also two horizontal strain gauges placed at half depth of the base layer and also at the bottom of the base layer. Two AMPT tank experiments were completed and the results were compared and analysed. A preliminary analysis was conducted using FLAC3D finite difference method software. Using an elastic isotropic model the stiffness modulus of both layers of the pavement was found. The lightly stabilized base layer had a stiffness modulus of 3650 MPa and the subgrade had a stiffness modulus of 70 MPa at the 10,000th cycle. The loading and vertical deformation at the 10,000th cycle was matched quite accurately therefore verifying the elastic modelling of the pavement and the suitability of FLAC3D for pavement modelling. This research confirmed that lightly stabilized base materials can be characterized by resilient response, rutting and fatigue cracking. In both experiments the pavement did not fail even though 1500 kPa is greater than any tire pressure currently on road pavements. The study conducted highlighted the complexities with understanding and attempting to characterize the behaviour of lightly stabilized materials.

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N = allowable number of load repetitions to fatigue
\( \mu \varepsilon \) = the tensile strain (microstrain) produced by the load
E = Modulus of cemented materials, MPa
RF = Reliability factor for cemented materials fatigue typically 1.0 for 95% project reliability.
MR = Resilient modulus, MPa
\( \sigma_d \) = Deviator stress, MPa
\( \varepsilon_r \) = Resilient strain, mm/mm
\( W_0 \) = Energy initial dissipation rate, Js\(^{-1}\)
\( W_n \) = Energy dissipation rate at the nth cycle, Js\(^{-1}\)
\( n \) = nth cycle

I. Introduction

A. Overview
Throughout the world for the past few decades cementitious stabilized materials have been very popular and proven to be very suitable for roads and runway pavements. However there is only limited knowledge of their complex properties, fatigue cracking and behaviour under heavy traffic (Hu, Li, Meng, Shen and Wei 1997). Research on the characteristics of pavements will not only affect new construction of roads and runways but with every road and runway maintenance is needed to repair its failures. After substantial maintenance usually the pavement will be completed ripped up and relayed, thus research on the behaviour of pavements is essential to better characterize and understand the failure modes and will have an affect on every road currently in service and those constructed in the future.

Both unbound granular materials and stabilized materials have been extensively researched and utilized in pavements over the last 50 years though their complex behaviour is still being understood. Even less is known about lightly stabilized materials which have only recently started to be used in pavement construction due to the benefits of stabilizing but at a lower cost. Due to the nature of the pavement industry and the need for new roads and existing roads constantly needing maintenance any further characterization of pavement materials especially lightly stabilized base materials under heavy traffic will directly improve pavement design and will translate into more efficient and economical road construction.

Pavements are classified as rigid or flexible. In common practice rigid pavements are known as concrete pavements and all other types are known as flexible pavements. Flexible pavements are the focus of this study and in particular the base layer which supports the majority of the traffic loading.

Stabilization is the process of mixing an additive to the pavement base or subgrade in order to bind the particles of that material together for increased strength and water resistance. Lightly stabilized base materials will be analyzed using both analytical and numerical models in order to provide a characterization of their properties. The Accelerated Model Pavement Testing (AMPT) was chosen for this study because it records the deformation of a simulated traffic like load on a real life pavement section. The stress and strain of the lightly stabilized base material will be recorded for approximately 9,000,000 load cycles. The great advantage of this test is it uses an actual pavement section, under controlled laboratory conditions, and applies an accelerated dynamic loading which some pavements won’t be subjected to in their lifetime in a matter of weeks. This testing gives considerable amounts of significant data which can then be analyzed in order to characterize the properties of the material loaded.
B. Aim
The main aim of this research is to characterise lightly stabilized base materials through analysing experimental and numerical data of the accelerated model pavement testing.

C. Objectives
1. Carry out two accelerated model pavement tests and obtain reliable experimental test data.
2. Analyse accelerated model pavement behaviour.
3. Characterize lightly stabilized material properties.
4. Predict the performance of lightly stabilized pavements and validate against experimental data.

II. Literary Review

A. Flexible Pavements
Flexible pavements are able to flex and adjust with traffic loadings with no serious deformation or cracking. They have low flexural strength compared to rigid pavements which are able to transfer concentrated loads over a large area. Generally, flexible pavements consist of a surface course, base course, sub-base course and subgrade (Underwood 1995). The complete pavement profile is shown below in figure 1.

![Figure 1. Pavement profile (Underwood 1995).](image)

The top layers of the pavement must have greater strength hence better quality than the other layers due to the concentrated load. This load is transmitted on approximately a 45 degree angle to the subgrade as in seen in figure 2. The lower layers may be of lower quality due to the load being distributed (Craig 2004).

The top layer is the surface course and its main purpose is to provide a layer resistant to traffic wear and to impede water entering the pavement. On low traffic roads this may not be required and the first layer of the pavement may be the base layer. However one or more layers of bituminous surfacing will be used. The base course or base layer is the main load carrying layer and may consist of one or more layers composed of fine crushed rock, natural gravel, broken stone, stabilised or improved material, asphalt or other material.

The sub-base usual purpose is to transfer loads from the base layer to the subgrade and provide a layer between the base layer and the subgrade layer to ensure no intrusion of the subgrade occurs. It is also used if the natural subgrade is weaker than required. Usually it will be made up of a lower quality of material compared to that of the base layer. The subgrade is the Foundation of natural earth or it may be compacted layers of fill laid as a result of earthworks. If a subgrade material is stabilized it becomes part of the sub-base layer.

B. Stabilized Materials
Stabilized base materials provide an economical and easily constructed pavement with improved characteristics. Stabilized materials are defined as materials which have stabilizers or additives added to them. Materials can be stabilized by granular materials, Portland cement or cementitious materials (cement, flyash and slag), lime, bitumen, tar and even geotextiles. Granular materials are usually added to improve sands and clays which are poorly graded (Underwood 1995).

The most common reason for using stabilizing materials is that it is more of an economical solution to improve the strength of a lower quality readily available material than to import a higher quality one. The increase in tensile strength is attributed to the development of interparticle bonds which also gives a higher elastic modulus. A higher quality pavement also can mean a lesser thickness of material is required. The material also becomes more water resistant which decreases the changes in volume and strength which would usually occur under variable soil conditions.

In this research a cement and flyash combination was used for stabilizing the base layer. However the base layer will be considered as a lightly stabilized granular material thus it will still be a flexible pavement though will not be unbound. It will have considerable benefits of stabilizing without the thermal shrinkage
cracking which occurs in rigid pavements (Paul and Gnanendran 2010).

C. Shakedown Theory

The pavement is modelled as a layered elastic/plastic structure. There is a critical load or shakedown load which is the threshold for which after the soil experiences this load it will settle down or shakedown to a steady state. This shakedown load was first recognised by Booker and Sharp (1984). And later confirmed by Sharp (1985) and Brett (1987) which both concluded that many pavements do shakedown rather than continuously deforming. Shakedown theory suggests that all soil is rate-independent elastic/plastic material. The theory enables us to determine long term resulting behaviour without having to model the time intensive repeating loadings to get to the settlement end state. As shown in the diagram below the soil under repeated load which is less than the shakedown load is completely elastic and after the soil strains the strain is recovered. However above this load each cycle after the initial cycle plastic strain occurs.

Collins and Boulbibane (2000) discuss three basic causes of this shakedown effect:

1. Residual stresses still exist in the soil from the previous cycles and thus the effects from the next cycle is due to a total of residual stresses and applied stress.
2. Changing of material properties (e.g. strain hardening or softening)
3. Changes to the geometry of the surface

The effects of the first and second causes are quite important where the third is considered negligible.

There are three possible long term responses that the shakedown theory accounts for as shown in figure 3, the first case is above the elastic limit and after numerous cycles of continuous plastic straining the pavement undergoes 'shakedown' and exhibits purely elastic behaviour. This is due to the combined residual stresses and changing of material properties. Thus the pavement has undergone plastic deformation and or cracking and then after a number of cycles the deterioration ceases which is known as elastic shakedown.

![Figure 3. Four responses of the Shakedown Theory (Collins and Boulbibane 2000).](image)

The next two cases occur at higher levels of load to the previous case where shakedown doesn’t occur instead a cyclic plasticity occurs where the permanent strain settles in a closed loop or the pavement strain increases indefinitely known as ratchetting. In the two latter cases the pavement will fail. The load bracket for which the soil shakedowns and above it the soil fails is known as the shakedown load and is the critical parameter for this theory in characterising unbound granular soils.

D. Characterization of Base Materials

In order to characterize base materials tests have to be conducted to understand their behavioural properties to be able to determine how the material would perform in the actual field environment. These properties need to be accurate in order to provide effective pavement design (Gnanendran and Piratheepan 2010). The AMPT testing and the use of FLAC3D finite difference method analysis software will be
used to characterize the lightly stabilized base material used in this study. The following are laboratory tests which are usually used to characterize lightly stabilized base materials.

1. **Triaxial Test**
   The dynamic confined triaxial compression test axially loads a cell made from a rubber membrane filled with soil confined by a desired fluid pressure (usually water is used). The confining pressure is used to simulate field conditions. The main reason to conduct this test is to find the resilient modulus and permanent deformation (Austroads 2006). It has been used to characterize stabilized materials (Paul 2011).

2. **California Bearing Ratio (CBR)**
   CBR is a simple test and is used all around the world. The test is a penetration test where a standard piston 1935 mm² is used to penetrate the soil at a standard rate of 1.3 mm per minute, the pressure at each 2.5 mm increment is recorded up to 12.7 mm and the results are correlated to the bearing value of a standard crushed rock which is known as the CBR value.

3. **Indirect Diametrical Tensile (IDT) Testing**
   IDT testing or the Brazilian Test allows measurement of the tensile strength of brittle materials by applying a compressive load along both sides of the cylinder sample to induce fracture in the diametrical plane. Recently IDT testing has been used to determine the strength and stiffness characteristics of lightly stabilized materials, however the results gained have proven to be inconsistent and unreliable (Foley and Group 2001b).

4. **Unconfined Compressive Strength (UCS)**
   UCS testing is a common and simple testing method to determine the compressive strength of stabilised base materials, similar to the triaxial test except with only atmospheric pressure confining the sample and no membrane required (Craig 2004). When the sample is loaded quickly the pore pressures do not have time to dissipate, which is analogous to a fast rate of pavement construction (Paul and Gnanendran 2011). The test can be used to determine the gain in strength over time associated with cementitious binders and also can determine this in different temperatures to model for winter of summer conditions. Usually correlations with UCS and stiffness or other properties are used from UCS testing.

5. **Flexural Beam test**
   The flexural beam test is setup by applying equal load at two points, each point a third of the length of the beam from the beam end. The test can be used to find the tensile strength of the beam and the stiffness modulus or resilient modulus. This test is common due to its simple nature and applicability to field environment (Austroads 2006). Lightly stabilized materials rely on compaction and surrounding soil to distribute the tensile stresses in the pavement therefore a beam cannot give as accurate results as model pavement testing or in-situ field testing.
   The following are field tests which have been used to characterize lightly stabilized base materials in the past.

6. **Falling weight Deflectometer (FWD)**
   FWD is a common non-destructive test which is conducted to closely simulate traffic loading conditions on a pavement in-situ. A weight is dropped to create a pulse comparable to a single axle wheel load and sensors (usually geophones) are used to measure the deformation of the pavement (Gnanendran and Piratheepan 2010). FWD evaluates the behaviour of pavement in the field environment though has certain limitations due to the fact the deformation recorded by the sensors is collective from all layers and has to be calculated using a finite element/difference program to determine the stiffness properties and deformation of each layer (Paul and Gnanendran 2010).

7. **Accelerated Model Pavement Testing (AMPT)**
   The AMPT is a relative simple test to conduct but much preparation and equipment is needed. It gives a complete picture of the stress and strain reactions at multiple depths in the pavement layer system which are usually connected to a computerised control and data acquisition system. It also enables testing on an actual pavement system under controlled conditions in the laboratory to determine the stiffness modulus and permanent deformation characteristics of the pavement material. Instead of waiting 20 years for the pavement to have 9,000,000 cycles of wheel loads going over it this can be conducted in a matter of weeks in the laboratory. Gnanendran and Piratheepan (2011) conducted pavement model testing on recycled aggregates in order to determine stiffness modulus and permanent deformation (i.e. rutting) and concluded that AMPT could be used to simulate the traffic loading on pavements reasonably well.
E. Finite Element/Difference Method

Using computer software to calculate simulated pavement model is becoming more and more popular especially due to the costs of large scale pavement testing. The software should always be used in conjunction with testing so it can be validated. The best tests to compare and to be used in conjunction with the finite element/difference methods is without a doubt the real life pavement example such as the AMPT or other in-situ real life pavement tests.

The ICAR team developed models for the resilient and permanent deformation behavior of unbound aggregate bases (UAB) using the results from triaxial tests and field scale tests. They concluded that UABs should be modeled as nonlinear and cross-anistropic to account for stress sensitivity, the significant differences between vertical and horizontal moduli and Poisson’s ratio (Kim, Little and Tutumluer 2003). They also recommend not to use isotropic design approaches due to the risk of under designing flexible pavements or overestimating the number of design axle loads the pavement can withstand. (Paul and Gnanendran 2010) concluded that a reasonably good prediction for lightly stabilized base materials was found by the adoption of the isotropic elastic model using FLAC3D.

F. Failure Modes

There are three accepted modes of failure in flexible pavements, they are as follows:

1. Fatigue cracking

Fatigue cracking is due to repeated excessive flexure causing maximum tensile strain usually at the bottom of the surface layer or by reflective cracking from underlying cementitious bound pavements (Underwood 1995 and Huang 2004). There is still debate whether or not the cracks are propagated from the bottom layers to the top or from the top down. The literary study which Paul (2011) undertook, suggested that early studies claimed the crack propagation at maximum tensile strain at the bottom of the bitumen layer and more recent study have led to claims of crack propagation from the tyre-pavement interface causing maximum horizontal shear stresses. This research will attempt to investigate fatigue cracking. Fatigue cracking is the principal mode of failure for pavements involving stabilized granular materials.

2. Rutting

Rutting is plastic deformation attributed to repeated loading. It can occur in any of the layers of the pavement or a combination. Usually it occurs due to a lack of compaction or poorly mixed asphalt materials in the surface layer (Huang 2004). Rutting is the principal mode of failure for pavements involving unbound granular materials.

3. Thermal cracking

Thermal cracking is due to both low-temperature cracking and thermal fatigue cracking. Low-temperature cracking occurs in cold climates where the thermal stress is greater that the facture strength. Thermal fatigue cracking is similar to fatigue cracking and is caused by the tensile strain that is due to the daily temperature cycles (Huang 2004).

Pavement may fail due to one of the three modes, though in most cases it is usually a result of either two or all three of the modes. Because fatigue cracking is the principal mode of failure for stabilized materials and rutting is the principal mode of failure for unbound granular materials it is assume that with lightly stabilized materials a combination of fatigue cracking and rutting will occur. Pavement behavioral modeling is required to determine the life of pavements. The fatigue properties are usually characterized by stiffness modulus, tensile strength and tensile strain at the bottom of the pavement layer. Therefore the critical tensile strain level needs to be understood. In this research the long term failure modes of lightly stabilized materials will hope to be understood and characterized. There are many different definitions and fatigue models used to describe a failed pavement for cementitious stabilized materials. Austroads (2004) using the following equation to calculate the fatigue life of the pavement:

$$N = RF \left( \frac{113000}{E} + 191 \right)^{12} \quad \text{Eq. [1]}$$

This equation has some limitations and can only be used for moduli between 2000-10000 MPa (Paul, 2011). A common failure definition used by several researchers (Gnanendran and Piratheepan 2009, Paul, 2011) is the pavement has failed once its stiffness has reduced by 50 %. Other researchers (Read and Collop 1997) have picked arbitrary measurements such as a vertical deformation of 9 mm. The other common approach is using an energy dissipation method, such as the:

$$\text{Energy Ratio} = \frac{W_0}{W_n} \quad \text{Eq. [2]}$$
II. Accelerated Model Pavement Testing (AMPT) Setup

The pavement structure used for this research, is shown in figure 5, and was constructed with a lightly stabilized base layer of 150 mm and a subgrade layer of 600 mm compacted inside a 1175 mm diameter tank. The lightly stabilized base layer was made up of 1.5 % binder (75 % GB cement–25 % fly ash mix) and Queensland road (granular) material (QRM) Base Type 2.1 quarried aggregate from Wagners Wellcamp Quarry, Queensland. This was classified as a well-graded sandy gravel with some fines according to the Unified Oil Classification System. The subgrade chosen was Queensland black soil (clay). Hu et al. (1997) stated there is potential for non-uniform inadequate mixing of the stabilizer and selected soils so this was taken into consideration and extra care was taken to ensure complete uniform mixing.

The clay subgrade with optimal moisture content (OMC) was compacted to achieve approximately 100 % compaction. The base layer was then mixed with the binder at OMC and then spread over the top of the subgrade. The base layer was compacted to approximately 96% dry density. The compaction was achieved in numerous stages. A line of measurements were made on the side of the tank to ensure the required calculated mass of the material was compacted to the necessary height as shown in using the densities calculated through experimentation in table 1. The compaction was achieved using a vibratory compactor. The base layer was then left 7 days to cure under a saturated hessian cloth. The base layer was then sealed using a latex liquid rubber.

During construction a series of two horizontal strain gauges were placed at the bottom of the base layer and at half depth of the base layer to record the tensile stress. Both horizontal strain gauge pairs were spaced 50 mm apart and the centre of the pair was in line with the axial centre of the circular tank. A series of two Linear Variable Differential Transformers (LVDT) were also setup using a special settlement-plate type mechanical system (figure 6) at the bottom and half depth of the base layer to determine the vertical deformation throughout the base layer. The LVDTs were setup in a cross formation all spaced 90 mm from the loading plate (centre to centre). Two LVDTs were also setup on the actual loading plate to measure complete vertical deformation of the pavement. The complete set up is shown in figures 8 and 9. The loading system consisted of a hydraulic ram with a 110 mm diameter loading plate for the Test 1 and 184 mm diameter plate for Test 2. The cyclic loading was setup to have a minimum of greater than zero force so there would be no rocking, impact action or eccentric movement of the loading plate when the force would be reapplied each cycle. This setup was found to be successful by Gnanendran and Piratheepan (2010) and therefore was used in this study.

Figure 5. Circular Accelerated Model Pavement Testing Setup (originally, Gnanendran and Paul 2010, modified by author.)

Figure 6. LVDT set up.

Figure 7. Loading system and LVDT setup.

Figure 8. AMPT setup with data control and acquisition system.
III. Methodology

The traffic loading was simulated by a sinusoidal stress pulse of 1500 kPa at 3 Hz through the loading plate for Test 1 and was conducted in week long phases of 750 kPa, 1000 kPa, 1250 kPa and 1500 kPa for Test 2. This loading of 1500 kPa initially was chosen as it much greater than the standard tyre pressure accepted by Austroads (2004) for designing flexible pavements which is 750 kPa. Austroads (2004) states that the tyre pressure for heavy vehicles are within the range of 500-1000 kPa and the standard tyre pressure is 750 kPa. The increase in load was chosen to ensure the pavement fails.

The response of the LVDTs and strain gauges were recorded using a computer data control and acquisition system (figures 4 & 5). The test will run approximately 9,000,000 cycles and would have expected to have failed before then. The recorded stress, vertical deformation and horizontal strain will be compiled by a series of Matlab programs. The results will then be analysed initially as a whole pavement system. Appropriate stress–strain analytical models will be used to attempt to characterize the behaviour of the pavement and find the stiffness and shear modulus. Initially the system will be modelled as a single cycle and linear elastic, isotropic, 1-layer system. Progressively more complex models will be used to try to characterize the lightly stabilized pavement up to a dynamic, nonlinear, anisotropic, 2-layer systems. Whether the analytical models are deemed successful or not the pavement system will then be modelled by FLAC3D a finite difference analysis program using the stress and strain experimental data. The stiffness modulus for base layer will then be back calculated to fit the experimental data and the modulus will then be compared to the analytical models modulus.

Using the calculated modulus found an analysis will then be conducted using the FLAC3D program to understand the effects of different physical variables such as the size of the loading plate, boundary conditions and the frequency of cyclic loading on the lightly stabilized pavement model.

The pavement testing will initially be characterized by using the theory of elasticity for geomaterials. The theory of elasticity states that once the material has been unloaded the strain is recovered, it assumes the medium to be elastic, homogenous and isotropic (Goodier and Timoshenko 1934, Das 2008). An isotropic material is defined as a material which has equal strength properties in each direction.

It is known that soils are not actually elastic, however if the load is small compared to the strength of the material the deformation under each load is almost completely recovered, as seen in Figure 6, thus it can be considered to be elastic. The resilient modulus is defined as the elastic modulus for soils under the theory of elasticity. At the initial loads there is considerable permanent deformation though as the number of repetitions increase the plastic strain due to each load decreases (Huang 2004).

The resilient modulus is defined as:

\[ M_R = \frac{\sigma_e}{\varepsilon_r} \quad \text{Eq. [3]} \]

Thus the main property used to characterize the behaviour of base layers is the resilient modulus, in this research the most accurate model will be used to find and characterize the behaviour of base layers as well as looking at the long term behaviour of fatigue cracking or rutting. The first AMPT test was started on the 7th of April 2011 and will conclude on the 16th of May 2011. The second test will run from the 7th of June 2011 until the 16th of July 2011.
IV. Results and Analysis

Figure 10. The Vertical Deformation Min and Max Amplitude of the Pavement Structure versus Cycles.

There were two AMPT tests conducted over the period of the study. The first and second AMPT test was conducted as per the setup previously reported. However, the loading plate of the second test had a diameter of 184 mm opposed to 110 mm for the first test. The loading for the second test also differed having incremental stages starting at 750 kPa and after each week it was increased by 250 kPa until 1500 kPa was reached and then the test remained on 1500 kPa for a following two weeks. Test 2 was conducted to provide further analysis on the behaviour of a lightly stabilized pavement undergoing different periods of increased cyclic loadings.

During the Test 1 several interferences occurred. The test stopped five times for an average period of four hours each time. This was due to the hydraulic ram and loading actuator system malfunctioning. The results were edited to maintain continuity and it appeared to not affect the results greatly. This conclusion verifies that the AMPT can be used for pavements even though it is a constant cyclic loading. The hydraulic ram also leaked a small amount of hydraulic fluid throughout the test but it did not appear to affect the results. During Test 2 there was only one notable interference where a power failure occurred during the last loading phase and the test was restarted approximately six hours later. This period of unloading did not significantly affect the pavement stiffness behaviour, resilient response or fatigue response.

For the second test the complete base layer and approximately 200 mm of the subgrade was excavated and replaced. The depth of 200mm of subgrade was excavated to ensure the hydraulic fluid had been removed from the pavement structure. The hydraulic ram was also taken apart completely and fixed by one of the Lab Technicians and a new loading actuator was purchased and installed.

It appears over the whole pavement structure several stages of consolidation and shakedown effects occurred which is mainly a result of one of the layers undergoing consolidation and not the pavement structure as a whole. The base layer as a whole undergoes a sort of consolidation in the vertical deformation and at a different number of cycles in the horizontal deformation which highlights the complex structure of a lightly stabilized base layer and a pavement structure in general.

The vertical deformation of the lightly stabilized base layer increased until the 2,000,000th cycle where it then underwent a consolidation and the base layer remained at a constant (lower) amplitude for the remainder of the test. There was almost no similarity in deformation pattern between the base layer and the subgrade. The subgrade deformation increased up until the 12,000th cycle and then underwent a change in the structure and then rapidly decreased in net deformation amplitude until the completion of the test.

The horizontal deformation generally at the middle of the base decreased up until the 2,000,000th cycle and then continued in a stable consistent amplitude for the remainder of the test. The horizontal deformation at the bottom of the base though continued increasing to around the 6,000,000th cycle and then underwent a consolidation phenomenon and then recovered deformation for the remainder of the test.
A. Test 1 Resilient Response: Pavement Stiffness Behaviour

Figure 11. The Vertical Deformation of the Whole Pavement Structure versus Cycles and Log Cycles.

The vertical deformation amplitude, defined as the net deformation, of the whole pavement structure occurring at each cycle, generally decreased overall with the increase in the number of cycles. Initially the vertical deformation amplitude was approximately 0.3 mm and then the vertical deformation amplitude of the pavement structure increased in amplitude up to 0.35 mm until the 12,000th cycle where it then underwent a consolidation or shakedown phenomenon. The pavement structure vertical deformation amplitude rapidly decreased until it became stable around the 2,000,000th cycle at an amplitude of 0.15 mm and remained stable for the next 7,000,000 cycles.

The whole pavement after the 12,000th cycle underwent a shakedown response and as cycles continued the pavement’s stiffness increased until a steady state was achieved after the 2,000,000th cycle. For the rest of the cycles the vertical deformation amplitude remained stable at its lowest value during the test and hence the stiffness modulus increased up to the 2,000,000th cycle and then also remained stable for the rest of the test. Thus the whole pavement tended towards a consolidated state with increased stiffness as the cyclic loading progressed.

Figure 12. The Vertical Deformation Amplitude of the Base Layer versus Cycles and Log Cycles.

The lightly stabilized base layer vertical deformation amplitude only contributed to approximately a third of the deformation of the whole pavement structure. The base layer followed a similar deformation trend as the whole pavement structure though only slightly increased in amplitude from 0.1 mm to 0.105 mm and then underwent a consolidation or shakedown phenomenon around the 12,000th cycle and then rapidly decreased until the 2,000,000th cycle where it was stable for the next 7,000,000 cycles.

The lightly stabilized base layer was similar to the whole pavement as after the 12,000th cycle the base layer underwent a shakedown response and the stiffness increased rapidly until the 2,000,000th cycle where it
remained at a stable consistent stiffness. Thus the base layer as the test progressed consolidated to a consistent stiffness behaviour.

![Figure 13](image1.png)

**Figure 13. The Vertical Deformation Amplitude of the Subgrade versus Cycles and Log Cycles.**

The vertical deformation amplitude of the subgrade layer mirrors the whole pavement structure. It is clear that the dynamic load cycle is transferred through the lightly stabilized base layer with only a small vertical deformation. The load is then transferred to the subgrade, even though it has been further spread out through the structure, the subgrade still deformed twice the amount of the base. This is due to the subgrade’s much lower strength and stiffness properties. The subgrade doesn’t find a stable state even though the net amplitude decreases and thus the stiffness increases, the subgrade continues to deform and consistent amplitude does not occur.

![Figure 14](image2.png)

**Figure 14. The Horizontal Deformation Amplitude at the Bottom of the Base Layer versus Cycles and Log Cycles.**

The horizontal deformation amplitude after the 1000th cycle rapidly increased up until the 2,000,000th cycle and then remained at a semi-stable state for the remainder of the test. Using the common definition that a pavement could be considered to be failed once the stiffness modulus has reduced by 50%, however including all the data this may not be appropriate for lightly stabilized pavement. The horizontal deformation is directly proportional to the strain and can be easily converted to strain using a scalar factor provided by the manufacturer. Thus in this case if the whole horizontal deformation is taken into account and hence because it is directly proportional to the horizontal strain which is inversely proportional to the stiffness modulus then the lightly stabilized base layer could be perceived as almost failed. However half of this is due to the increase up to 20,000 cycles thus initially the pavement usually undergoes a significant deformation before it consolidates and then continues deforming at a reduced rate, this may be disregarded to give a true picture of whether or not the pavement has or will fail. In this case if you do count the deformation from all the cycles the lightly stabilized base layer has actually failed as the strain has almost increased to twice that of the initial value. Even though the base layer has actually recovered part of its deformation and appears to have reached a stable state. If the initial
20,000 cycles are discounted then the initial deformation would be 0.015 mm and due to the increase only to 0.019 mm the base layer could be reported as not failed.

B. FLAC3D Backcalculation

The circular tank was simplified and modelled as a quarter grid as shown in figure 15. At the 10,000th cycle the vertical deformation of the base and subgrade were matched on FLAC3D-Dynamic by using the backcalculation method. The stiffness modulus found for the lightly stabilized base layer was 3650 MPa and for the subgrade the stiffness modulus was found to be 70 MPa, the pavement model was matched using a linear elastic isotropic model and it seemed quite accurate as shown in figures 16 and 17. The Poisson’s ratio for each layer was inputted into FLAC3D which was assumed to be 0.2 and 0.3 for base and subgrade respectively according to Austroads (2004) and Gnanendran and Piratheepan (2009). The analysis conducted using FLAC3D was a limited analysis this was due to the long period of time learning how to operate the software and how to write the code for the pavement model.
Table 1. Material properties used in FLAC3D to model Test 1.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Stiffness (MPa)</th>
<th>Modulus Bulk (MPa)</th>
<th>Modulus Shear (MPa)</th>
<th>Density (kg/m³)*</th>
<th>Poisson’s Ratio#</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base</td>
<td>3650</td>
<td>2028</td>
<td>1521</td>
<td>2474</td>
<td>0.2</td>
</tr>
<tr>
<td>Subgrade</td>
<td>70</td>
<td>58</td>
<td>27</td>
<td>1775</td>
<td>0.3</td>
</tr>
</tbody>
</table>

(*Required density found by experiment conducted by Gnanendran and Paul (2010). #Acceptable values according to Austroads (2004).)

Once the code was written each time the program ran it took an estimated five hours to replicate one cycle which equates to a third of a second, this turned out to be quite laborious and due to the trial and error nature of backcalculation only preliminary values were found. These however now can be used to study the effects of loading plate area, depths of subgrade, different strength subgrades, etc. The possibilities of research using FLAC3D, once the results are verified by an experiment, are endless.

C. Test 1 Permanent Deformation: Rutting Failure

![Graphs showing permanent deformation](image)

Figure 18. The Permanent Vertical Deformation of the Pavement versus Log Cycle.

The most common form of failure for flexible pavements is permanent deformation or rutting failure. Overall the first AMPT has not failed and has only suffered slight permanent deformation. The whole pavement underwent 0.7 mm permanent deformation during the whole test. The subgrade and the base layer both contributed to the permanent deformation however the subgrade contributed to 0.4 mm opposed to the base
layer’s 0.3 mm. Though both layers had comparable permanent deformation, the base layer has underwent an elastic shakedown phenomenon at the 2,000,000th cycle and remained stable and elastic for the next 7,000,000 cycles. Whereas the subgrade after the 200,000th cycle appears to have started the ‘ratchetting phenomenon’ where rutting increases rapidly and the stiffness modulus reduces until failure. This is common with most flexible pavements where rutting failure is contributed to the subgrade layer due to its low stiffness modulus opposed to the base layer and as confirmed by the values given in the backcalculation method.

D. Test 1 Horizontal Deformation: Fatigue Response

Figure 19. The Horizontal Deformation Amplitude at the Middle of the Base Layer versus Cycles and Log cycles.

Figure 20. The Horizontal Deformation Amplitude at the Bottom of the Base Layer versus Cycles and Log Cycles.

The comparison of the middle and bottom base layer horizontal deformation highlights that the horizontal deformation at the bottom of the base is at least ten times greater than the horizontal deformation at the middle of the base layer. This is to be expected due to the flexible plate nature of the stress distribution of the base layer, with compression forces at the top of the layer and tensile stresses at the bottom of the layer. The net horizontal deformation of each cycle at the middle of the base immediately increased until the 100,000th cycle and then decreased until the 6,000,000th cycle where it then slightly increased and then stabilized. The bottom of the base layer had stable 0.01 mm net horizontal deformation and then slightly increased from 1000 cycles up until 2,000,000 cycles and then stabilized and slightly decreased.
The horizontal deformation or the fatigue response is usually not relevant in unbound granular materials but in stabilized base materials fatigue cracking usually contributes significantly to a failed pavement. Fatigue cracking results from repeated loads which is generally accepted forms cracks at the base of the layer and reflects these cracks up to the top surface of the pavement. However lightly stabilized base materials are still yet to be characterized and it hasn’t been confirmed whether or not fatigue cracking or rutting failure is the main fail mode. In this experiment there was no visual cracking on the surface of the base layer and thus the lightly stabilized base layer didn’t fail. As recorded in the graphs above the base layer has actually reduced in stiffness and then slightly recovered and remained stable for the last 7,000,000 cycles.

E. Test 2 Resilient Response: Pavement Stiffness Behaviour

![Figure 18](image1.png)  
*Figure 18. Vertical deformation amplitude of the whole pavement structure vs log cycles for the 750 kPa, 1000 kPa, 1250 kPa, 1500 kPa test phases.*

![Figure 19](image2.png)  
*Figure 19. Vertical deformation amplitude of the subgrade vs log cycles for the 750 kPa, 1000 kPa, 1250 kPa, 1500 kPa test phases.*

The second test conducted was initially started with a 750 kPa cyclic loading after a week period the load was increased by 250 kPa until after three weeks it was at 1500 kPa where it remained until the completion of the test. This test provides an insight into multiple loadings on a single pavement and thus the pavement will undergo multiple consolidation phases.

The figures 18 and 19 above show the vertical deformation amplitude of the whole pavement structure and the subgrade, though the base is under investigation the subgrade greatly affects the stiffness behaviour of the pavement and contributes the most to the resilient response. In each loading phase after the 12,000th cycle the pavement underwent a consolidation or shakedown effect and the pavement’s resilient modulus became stiffer. Thus in all cases the pavement became more resilient after a number of loading cycles due to consolidation as well as the hydration of the cement.

The subgrade and hence in the pavement vertical deformation amplitudes are similar for both the 750 kPa and 1000 kPa, however there is approximately 0.05 mm increase between the 1000 kPa and 1250 kPa phases and then the same increase between the 1250 kPa and 1500 kPa. It is clear comparing figures 18 and 19 that the subgrade contributes over 80% of the pavement’s vertical deformation amplitude. All the vertical deformation amplitudes of the pavement and subgrade follow a somewhat similar pattern for each of the loading phases.

The pavement for each one of the loading phases can be considered to be not failed due to the stiffness modulus not reducing to half of its original value but instead in each phase the pavement’s stiffness modulus actually increased.
The resilient response of the pavement is also reflected by the horizontal deformation amplitude at the bottom of the base layer. All of the loading phases after approximately 40,000 cycles undergo a consolidation and increase in stiffness until the completion of that phase. The resilient response for the 750 kPa, 1000 kPa and 1250 kPa phases follow a similar pattern and are separated by an increase in amplitude of 0.001 as the load increases, however the 1500 kPa actually replicates the 1250 kPa pattern.

The pavement can be considered not to have failed but actually gained stiffness in the horizontal resilient modulus.

F. Test 2 Permanent Deformation: Rutting Failure

The rutting of the four loading phases are shown above in figures 22 and 23. The permanent deformation is calculated from zero for the start of each phase. The largest permanent deformation occurred during the initial 750 kPa phase after this period and after a certain amount of consolidation had been achieved the following phases didn’t experience as much rutting. The following phases deformed in a similar pattern and in similar overall values.

After the 750 kPa loading phase a consolidation or shakedown response has occurred which is the reason why the permanent deformation of the 1000 kPa phase was much less than that in the 750 kPa phase even though
the loading was increased. It appears that a stabilized state would have been achieved after the 1,400,000th cycle though because the loading was then increased it cannot be confirmed. The 1000 kPa and 1500 kPa phases started off at a rapid increase in permanent deformation however they seem to be stabilizing around the 1,000,000th cycle and the deformation isn’t occurring as rapidly. Overall the pavement hasn’t reached a complete stable state in any of the loading phases.

G. Test 2 Horizontal Deformation: Fatigue Response

The horizontal deformation also known as the fatigue response is the main contributing factor to surface cracking appearing at the top surface of the pavement. No cracking was visual at the surface and as it is recorded in figure 23 the resilient response amplitude actually decreased and thus the base layer actually stabilized against horizontal deformation.

H. Comparison of Test 1 and Test 2

The main differences between Test 1 and Test 2 are as follows: The loading plate in Test 1 had a diameter of 110 mm opposed to the loading plate in Test 2 which had a diameter of 184 mm. Test 2 had several loading phases starting at 750 kPa and increasing by 250 kPa at the end of each week up to 1500 kPa opposed to Test 1 which had a cyclic loading of 1500 kPa for the entirety of the test. All other variables were controlled between the two tests.

1. Resilient Response Comparison

The stiffness behaviour of the pavement in the first test was considerably less than the amplitude for the 1500 kPa phase of Test 2. The stiffness behaviour improved after the 12,000th cycle in Test 1 and this did occur for the 750 kPa phase in Test 2 though for the following phases it didn’t occur until after the 200,000th cycle. It would appear that any cyclic load 750 kPa and greater would cause the pavement to undergo a consolidation or shakedown affect after 12,000 cycles, however in the latter phases of Test 2 this did not occur due to the fact that the pavement had already undergone this initial consolidation. The amplitude of the 1500 kPa phase in Test 2 was at least 0.1 mm greater than in Test 1 which means that even though the pavement had undergone three previous phases of ‘consolidation’ the amplitude though constant was considerably greater than in the previous case.

The horizontal deformation amplitude at the bottom of the base in Test 1 was much greater than either of the phases in Test 2. In regards to the horizontal stiffness behaviour of the lightly stabilized base layer in Test 2 the horizontal deformation amplitude in the 1500 kPa phase was half the value in Test 1. The only reason for this is that preloading the pavement with the 750 kPa phase and the following phases actually improved the stiffness behaviour of the pavement.
2. Rutting Failure Comparison

The vertical permanent deformation for 1,000,000 cycles in Test 1 for the whole pavement was 0.5 mm and the subgrade was 0.2 mm. For the whole of Test 1 the total deformation was 0.7 mm for the pavement and 0.4 mm for the subgrade. For Test 2 the vertical permanent deformation for 1,000,000 cycles for the initial 750 kPa cyclic loading was 0.3 mm and almost half of that for each one of the loading phases thereafter. Thus for each loading phase the deformation was much less than that for Test 1. However in saying that in total the pavement in Test 2 underwent a vertical deformation of 0.75 mm opposed to Test 1 total of 0.7 mm. The substantial rutting which occurred in the initial 1,000,000 cycles of Test 1 is a result of the greater cyclic loading the pavement was subjected to initially.

3. Fatigue Response Comparison

In both tests cracks were not observed and thus the pavements did not fail. The main differences was that Test 2 didn’t stabilize where as Test 1 did, this is due to the change in loadings as well as the fact Test 2 only had approximately 1,000,000 cycles per phase where as Test 1 stabilized after 2,000,000 cycles.

V. Recommendations

A. Accelerated Model Pavement Testing

The first and foremost recommendation from this report is that Accelerated Model Pavement Testing should be undertaken and further studied in order to characterize lightly stabilized base materials. The large amounts of information which can be recorded from a six week test can provide much greater analysis and potential behaviour characterization than any of the other laboratory or insitu tests that are currently accepted.

B. Set Up

The set up overall is recommended. The symmetry of the circular tank also made modelling the tank in FLAC3D simplified which meant each iteration was markedly quicker. The pair of LVDTs at the top, middle and bottom surface of the base worked well and provided accurate and verified results. The pair of horizontal strain gauges placed at the middle and bottom of the base layer worked well though in the last week of Test 2 one of the strain gauges broke and the measurements after this period were null and void, however the measurements from the other strain gauge could be used exclusively for the remainder of the test. Thus it is recommended that at least a pair of each measuring devices at each location needs to be maintained in the AMPT set up so the measurements can be verified, averaged and if need be have a redundancy.

C. Lightly Stabilized Base Materials

Lightly stabilized base materials with 1.5 % binder (75 % GB Cement and 25 % Flyash) are recommended for pavements. This is due to the conclusions of this report and the fact that even after 9,000,000 cycles, of a heavier load than the pavement would be ever exposed to, the pavement still did not fail.

D. FLAC3D and Abaqus

The finite difference method analysis conducted on FLAC3D was quite accurate and thus verified the major application the finite difference software can be to designing pavements once the program has been verified by experimental results. Once a verified program has been written and thus the pavement properties have been found the pavement can then be modelled for countless situations such as varying: loads, loading plate diameters, loading time, depths of pavement layers, boundary conditions, different subgrades, etc. The possibilities with a verified program are limitless. However the amount of iterations and time required to conduct each simulated cycle was a major drawback. An initial simulation was conducted on Abaqus though unsuccessful, the amount of time to simulate one cycle was only several minutes opposed to several hours with the FLAC3D program, thus if an accurate model can be verified in Abaqus a greater amount of characterization may be able to be achieved.

E. Further Research

It is recommended that further research on lightly stabilized base materials occurs in order to characterize their behaviour. The properties of lightly stabilized base materials are only just starting to be analysed and if thorough research is continued the economic benefits that could be achieved through pavement design, construction and maintenance could be significant.
VI. Conclusions

The following are the main conclusions which were reached after conducting two AMPT tests:

a. The pavement structure consisting of 150 mm lightly stabilized base material with 1.5 % binder (75 % GB cement and 25 % Flyash) and 600 mm subgrade after 9,000,000 cycles at 1500 kPa did not fail in regards to resilient response, rutting or fatigue cracking. This load is much greater than any heavy vehicle expected on road pavements according to Austroads (2004). Thus lightly stabilized base materials are suitable for road pavements. The stiffness modulus found for the lightly stabilized base layer was 3650 MPa and for the subgrade the stiffness modulus was found to be 70 MPa, the pavement model was matched quite accurately using a linear elastic isotropic model in FLAC3D.

b. Lightly stabilized base layers can be characterized by resilient response, rutting failure and fatigue cracking.

c. The AMPT tests conducted obtained reliable results which was verified by the comparison of Test 1 and Test 2.

d. In both AMPT tests the vertical and horizontal stiffness modulus of the pavement improved as the load cycles increased. After 1,000,000 cycles the base layer began to stabilize in each test. This was contributed to both the shakedown phenomena or consolidation as well as the hydration of the cement as the test progressed.

e. The shakedown theory was confirmed during both tests especially Test 1 as both the lightly stabilized base layer and subgrade layer underwent shakedown phenomena.

f. The cyclic loading stopped several times during Test 1 and even though there was no significant discrepancies once the cyclic loading continued. Thus lightly stabilized pavements handle non-uniform loading cases well.

The accelerated model pavement testing has been verified. It is a very useful test to run in order to monitor real life pavement behaviour in an accelerated timeframe. From AMPT testing there is large amount of information that can be used to potential characterize the behaviour of the pavement. The AMPT were both successful and the lightly stabilized pavement did not fail in resilient response, rutting or fatigue cracking. FLAC3D was used to characterise the stiffness properties of the pavement structure and could be used to model this lightly stabilized base layer in countless situations.

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