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Backcalculation of Resilient Modulus of Lightly Stabilised Granular Base Materials from a Cyclic Load Testing Facility

Piratheepan, J. and Gnanendran, C. T.

Abstract

Resilient moduli of pavement layers are the basic input parameters for the design of pavements with multilayers in the current mechanistic-empirical pavement design guides. Field measurements are generally believed to provide accurate values to backcalculate pavement layer moduli, but the tests to measure the relevant parameters are difficult to perform, expensive and causes disturbance to the public. Therefore, backcalculation of pavement layer moduli from laboratory scale model testing has been a focus of recent pavement research. This paper presents a backcalculation analysis to evaluate pavement layer moduli using a 3D numerical model developed using the FLAC3D finite difference software. The pertinent measurements that are required for the backcalculation analysis were collected from a cyclic load testing facility under traffic type cyclic loading conditions with a typical pavement structure consisting of a granular road base lightly stabilised with cement-flyash over an expansive soft clay subgrade. This study indicates that the stabilised base material had cross-anisotropic resilient properties with an average vertical resilient modulus of 2875 MPa and an average horizontal resilient modulus of 1598 MPa. From this investigation, the resilient moduli of the stabilized granular base layer and subgrade clay were backcalculated reliably from the analysis using FLAC3D numerical model.

Key words: Backcalculation; numerical model; resilient moduli; cyclic loading; stabilised granular materials; pavement model testing
Introduction

Granular materials lightly stabilised with slow-setting binders are widely used as base and sub-base of flexible pavements for the last few decades. Slow setting binders such as slag-lime, flyash-lime and flyash-cement allow the required working time during pavement construction due to their slow hydration process and thus are less susceptible to shrinkage cracking (Chakrabarti and Kodikara 2005; Foley 2001a; White and Gnanendran 2005). For the design of pavements with lightly stabilized granular materials, these materials are usually characterized in terms of tensile strength and stiffness properties. For example, in the current mechanistic-empirical pavement design approach, cementitiously stabilized granular materials are characterized by their resilient modulus and the thickness design is based on the tensile-strain – fatigue-life relationship (e.g. AUSTROADS 2004; NCHRP 2004). In particular, the resilient modulus of the stabilized material layer of a pavement structure is a key input parameter for estimating the tensile strains induced in the material by the axle loading in a typical mechanistic pavement design (e.g. Huang 1993; Wardle 1997).

The resilient modulus of stabilized materials can be evaluated by various laboratory testing methods such as cyclic triaxial and indirect tensile testing. However, there are limitations in these approaches particularly because the characteristics are determined on smaller specimens (e.g. 100 to 200 mm Diameter sample whereas the largest aggregate particle could be 19 to 25 mm) and that the testing is done in isolation from possible interaction between different material layers. Field tests are generally believed to provide good indications of performance as they are free from inherent experimental limitations, but they are often costly and long term trials will take many years, and thus are often not suitable for the current improvement in the design process. A possible alternative is a laboratory scale test in which repetitive (or cyclic) loads that a prototype pavement experiences could be applied to the model pavement
structure and enable monitoring the pavement response in a short time (Al-Qadi et al. 1994; Chehab et al. 2007; Gnanendran et al. 2010; Llenin et al. 2006; Martin et al. 2003; Perkins et al. 1998; White and Gnanendran 2005). This could be more appropriate, primarily because of the fact that the same amount of damage can be induced in the model pavement under controlled environmental and other conditions much quicker than in actual pavement. However, some factors such as rate of loading, environments, aging of stabilised materials etc. would be difficult to achieve.

Evaluation of resilient moduli of pavement layers can be carried out in many ways. In situ resilient moduli of pavement layers can be determined using plate bearing test, multi-depth deflectometer or falling weight deflectometer (FWD)-generated data. Several investigations reported using the FWD to characterise pavement materials in terms of their structural properties (Hossain et al. 1997; Janoo 1994; Zhou et al. 1997). A similar approach was used in this investigation and the objective of this paper is to evaluate the resilient moduli of pavement material layers using pavement model testing and back calculation.

A cyclic load testing facility was developed for evaluating the resilient modulus of different materials in a pavement structure. In particular, a typical pavement structure consisting of a lightly stabilised granular road base material over a typical soft clay (i.e. Queensland black clay) subgrade material was studied using a pavement model testing facility (PMTF) under cyclic loading conditions. In the PMTF, the model pavement structure was constructed in a 1.0 m x 1.1 m x 0.6 m steel tank and the wheel loading from the vehicle was simulated as a sinusoidal type cyclic axial loading applied over a stationary circular steel plate using an actuator assembly. The tensile strain at the bottom of the lightly stabilised material layer was monitored with dynamically rated special strain gauges installed during the laying and compaction of the stabilised material in the test tank. The vertical deformation at different
depth levels were monitored with purpose made settlement plates made of plastic. Details of the instrumentation adopted and typical results obtained from this study are discussed in the paper. In the backcalculation analysis, a 3D numerical model was developed to evaluate the resilient moduli using the FLAC3D (Itasca-Consulting-Group-Inc 2006) finite difference computer program. As the PMTF is a 3D problem, it cannot be analysed using 2D finite element programs. Moreover, the 3D pavement modelling allows accurate structural response analysis of pavement structures subjected to traffic loading. In the past several pavements have been modelled with 3D finite element simulations (e.g. Dong et al. 2001; Loizos and Scarpas 2005; Shoukry and William 1999; Uddin and Garza 2003; YOO et al. 2006). The backcalculated stiffness characteristics of the stabilised material and the subgrade clay are compared with those determined from other laboratory tests (e.g. cyclic load indirect diametral tensile (IDT) testing for lightly stabilised materials) and the results are discussed in this paper.

**Experimental Investigation Undertaken**

The model testing arrangement used in this research was to investigate the performance of a typical pavement structure consisting of a lightly stabilised granular base material placed over an expansive clay subgrade material under cyclic loading as illustrated in Figure 1. Queensland black clay as subgrade and granular base material stabilized with 1.5 % general blend (GB) cement-flyash binder under the optimum moisture content (OMC) conditions were chosen for this investigation. GB cement is a uniform blend of Portland cement and fly ash or slag that complies with the requirements specified in Australian Standard AS 3972 (2010). Properties of the materials used and the experimental details are presented in the following sections.
Table 1 summarises the testing program for the PMTF test investigation. The main objective of this PMTF test investigation is to determine the resilient modulus of a lightly stabilised granular base layer constructed over a soft clay subgrade. Therefore, the testing program was designed with two variables; the thickness of the base layer and the amplitude of the applied cyclic pressure. It was inferred from the first 5 test results that the moisture movement from subgrade to the base layer possibly changed the conditions of the materials. Therefore, in test PMT6, a layer of liquid rubber was applied at the interface of the base layer and the subgrade to prevent the moisture migration from one layer to the other while the configuration of the test was kept similar to test PMT4. For the pavement model testing, sinusoidal stress pulse of 3 Hz was applied to the loading plate through a servo control actuator assembly. The loading plate used in this investigation was a circular steel plate of 184 mm diameter and 30 mm thickness. Vertical deformation at the surface was monitored with (2 Nos of) LVDTs mounted on the loading plate and the settlement at the bottom of the modified material layer (i.e. at the top of the subgrade layer) and at mid depth were monitored using purpose-made settlement plate system by monitoring the tip of the rod with an LVDT (see Figure 1). The horizontal strain at mid depth and the bottom of the modified material layer were monitored with special strain gauges (Figure 1).

The data was collected in a computer and then transferred to another computer for processing and analysis. Two load measurements, six LVDT measurements and four strain gauge measurements were collected at a sampling frequency of 200 Hz during the cyclic load PMTF testing and they were saved in files in the computer throughout the test. Therefore, about 66 data sets were available per load-cycle as the load was applied at 3 Hz. In this investigation, a program code was developed using MATLAB to process the raw data from the cyclic load PMTF tests.
Materials and Sample Preparation

Two different materials were used to construct the model pavement structure in this investigation; one is Queensland black clay (or black soil) obtained from the state of Queensland, Australia as the subgrade and the other is a quarried granular material consisting predominantly angular particles also from Queensland which was lightly stabilised with cement and flyash as the base layer. The granular material was classified as GM-GC (Silty Gravel – Clayey Gravel) according to the Unified Soil Classification System (USCS) and well-graded granular material predominantly with gravel (A-1-a) according to AASHTO classification. The subgrade material used in this investigation was a clay material obtained from Norwin road, Bowenville, Queensland. The subgrade material was classified as CH - High Plasticity or Fat Clay according to USCS and Clay with moderate plasticity and which may be highly elastic as well as subject to a considerable volume change (A-7-5) according to AASHTO classification. Figure 2 shows the particle size distributions of the clay and granular base materials used in this investigation. The Atterberg limits presented in Table 2 were determined on the materials passing through Number 325 sieve (425 μm) according to AS 1289 (methods 3.9.1 (2002), 3.3.1 (1995) and 3.4.1(1995)).

The stabilised material was prepared by mixing 1.5% binder (75% GB cement and 25% flyash) to the quarried aggregate from Wagners Wellcamp Quarry in Queensland Australia, which gave an unconfined compressive strength (UCS) of 1.2 MPa and a resilient modulus of 580 MPa from cyclic load indirect diametrical tensile (IDT) test (Gnanendran and Piratheepan 2009). Details of the IDT test setup and its application to determine strength, resilient modulus and fatigue life of lightly cementitiously stabilised granular materials can be found elsewhere (Gnanendran and Piratheepan 2009; Piratheepan et al. 2010). It is noted that the load frequency of the cyclic load IDT tests was 3 Hz same as that of PMTF cyclic...
Properties of the subgrade material and the parent material of the stabilised material are summarised in Table 2.

Laboratory test was undertaken initially to determine the Standard Proctor moisture content-dry density (MC-DD) relationships for the materials using the ASTM-D698 (2007) method. These relationships were determined for a compaction effort of 600 kN-m/m³ (12400 ft-lb/ft³). Based on these MC-DD relationships, the moisture contents selected for preparing the model pavement structure in PMTF and other tests such as CBR and IDT tests in this investigation were the optimum moisture contents (OMCs) for the materials which were 31.95% (≈ 32%) for clay material and 9.25% for stabilised material.

Considering the capacity of the mixer and to maintain the consistency and uniformity of the mix, the dry material was divided into many batches (e.g. total weight of 490 kg clay soil required for 330 mm thick subgrade). Then, each batch (e.g. about 82 kg) was mixed separately in the mixer. Each batch of materials was mixed thoroughly in the mixer for 10 minutes before adding water to further ensure consistency. The required amount of water (corresponding to OMC) was then added slowly to the material and the material was mixed for another 10 more minutes. It is noted that the mixer used in this investigation for the mixing purposes of subgrade clay and stabilised granular material was a concrete mixer.

The material layers for the PMTF testing were compacted in the test box in several sub-layers to achieve the required maximum dry density at the respective OMC. The compaction of material layers was performed using a vibratory compactor/Jack hammer. The compaction and installation of instrumentation were performed carefully to obtain the model pavement structure.
For the curing of the stabilised base layers, the surface of the base layer was covered with wetted hessian mat. Saturated hessian is not a new technique and has been used for curing concrete and stabilised pavement materials for a long time. The hessian was wetted often to keep the relative humidity above 95%.

The pavement structure was tested after 7 days curing of the stabilised layer. After the curing, two layers of latex liquid rubber were applied on the surface of the stabilised layer. In addition to this, a sheet of rubber mat was placed over the surface of the pavement structure, after the latex rubber dried, to provide more protection against moisture loss. Then the model pavement test tank was positioned in place underneath the actuator mounted on the steel frame for the testing. The loading plate was then placed and levelled on the surface of the stabilised layer at the centre of the test box.

**Backcalculation of Resilient Moduli**

The resilient moduli of the two material layers in the PMTF tests were calculated from the vertical deformation measurements taken at various depth levels through a process called back calculation or inversion. Generally, the back calculation technique is used to determine the pavement layer moduli according to their deflection basins, as measured using any Accelerated Loading Facility (ALF), Heavy Vehicle Simulator (HVS) or Falling Weight Deflectometer (FWD) tests (e.g. de-Beer et al. 1989; Reddy et al. 2004; Zha and Xiao 2003). A number of back calculation methods are available for determining the moduli of pavement layers from these measured deflections, such as regression equations and multi-layered elastic analysis using a computer program. In this analysis, the FLAC3D computer program was used for back calculation of the moduli of the two material layers used in the PMTF tests.
Throughout this analysis, the input layer moduli were repeatedly changed until the stress and deformation curves matched those measured in the experiments.

**Development of Numerical Model**

In this back calculation analysis, a 3D numerical model was developed to analyse the PMTF tests using the FLAC3D finite difference computer program. As the geometry of the PMTF is quarter symmetry, quarter symmetry advantage was taken into consideration in this numerical modelling. Radial cylinder-shaped primitives from the centre of the loading plate were used to generate the numerical mesh. The domain used for the numerical simulation was the quarter tank sketched in Figure 3, which was generated with 18,000 zones. A system of coordinate axes was selected in which the x and y axes were in the plane of the model’s base and the z axis was pointing upward along the edge of the mesh. The loading plate was represented by a disk segment with a radius of 92 mm. The dimensions of the domain were 550 mm length in the y direction, 500 mm width in the x direction and heights, h, in the z direction being 480 mm and 580 mm. Displacements of the boundaries were restricted in the x direction at \( x = 0 \) mm, in the y direction at \( y = 0 \) mm, and in all the x, y and z directions at \( z = 0 \) mm, \( x = 500 \) mm and \( y = 550 \) mm.

The developed model consisted of two layers; one a subgrade and the other a stabilised base. Generally, in the backcalculation of pavement layer moduli, all the layers are modelled as elastic (e.g. de-Beer et al. 1989; Reddy et al. 2004). However, in this investigation, the subgrade material was modelled as an elasto-plastic material with Mohr-Coulomb failure criterion and the lightly stabilised granular base material layer was modelled as an isotropic elastic material. To allow slip at the interface between the two materials, an interface element layer was used in the finite difference analysis, as shown in Figure 3. The stabilised granular
material and the expansive soft clay were considered poor bonding materials. Hence, an interface element was used in the numerical analysis.

The friction angle of the interface element was taken to be the same as that of the subgrade material. The shear ($k_s$) and normal ($k_n$) stiffness values of the interface element were determined using the following equation given in the FLAC3D manual (2006):

$$k_s = k_n = 10 \times \left( \frac{(K + \frac{4}{3}G)}{\Delta Z_{\text{min}}} \right)$$  \hspace{1cm} (1)

where $K$ and $G$ are the bulk and shear moduli of subgrade material respectively; and $\Delta Z_{\text{min}}$ is the minimum zone size adjacent to the interface.

The loading plate used in this study was a thick steel plate and, therefore, it was considered to be rigid, and hence the same deflections at all grid points underneath it. As can be seen in Figure 4, the stress distribution on the loading plate will be non-uniform so the best input loading for the numerical analysis was a uniform displacement to the grid points covered by the loading plate.

However, it was necessary to ensure that the average applied stress over the loading area was the same as the applied stress on the loading plate. Therefore, the stresses on top of the base layer over the loading area were collected in a history file in order to calculate the average stress during the numerical iterations. Figure 5 shows a typical example of vertical stresses under the loading plate at 9.2 mm interval from the centre of loading plate along the radius.

The arithmetic and weighted average of the stress over the loading plate from numerical analysis was equal to the applied pressure of 1248 kPa in PMT3. The material inputs were changed in a systematic fashion until the vertical deformation of the loading plate, the
vertical deformation at the interface, the applied pressure on the loading plate and the horizontal strain at the bottom of the stabilised layer matched the corresponding values measured during the experiment. The loading input to the numerical model was applied as a sinusoidal velocity for the duration of one cycle.

**Analysis, Results and Discussion**

The base material was modelled as elastic while the subgrade material was the Mohr-Coulomb model. The Poisson’s ratio of the stabilised material used in this analysis was assumed to be 0.2 (AUSTROADS 2004). In this numerical simulation, the bulk and shear moduli were changed for the stabilised layer until the vertical deformations measured on the loading plate and at the interface, and the average applied pressure, matched the experimental results. Then, the vertical deformation measured at the half-depth of the stabilised layer was compared with the experimental value. Figure 6 shows the plots for the applied pressure and deformations at different depth levels in experiment PMT6 and the corresponding parameters monitored during the numerical simulation. Similarly, Figure 7 and Figure 8 show the horizontal strains at the bottom and half-depth of the stabilised layer with time and stress with deformation of the loaded surface respectively.

**Error! Reference source not found.** It is noted that the experimental and numerical curves agree very well, especially the maximum deformations of the cycles are the same. A desirable feature of this analysis is that verification or cross-checking of the backcalculated modulus of the stabilised base layer can be compared with the deformation measurements at mid-depth of the stabilised layer obtained from experiments and the numerical simulation. For calibration of the numerical model, as the vertical deformation at mid-depth of the stabilised layer was not considered, an independent parameter was introduced which could then be used for
verification/validation of the modulus of the stabilised base layer. Moreover, the stresses at the bottom of the stabilised layer underneath the centre of the loaded area were collected from the numerical simulation to determine the resilient modulus from the experimental strain measurements. This horizontal resilient modulus was compared with the backcalculated vertical resilient modulus.

The horizontal tensile resilient modulus, $E_h$, was determined using the elastic theory solution:

$$E_h = \frac{1}{\varepsilon_c} (\sigma_z - \nu(\sigma_y + \sigma_z))$$

(2)

where $\varepsilon_c$ is the measured critical horizontal tensile strain during the experiment; and $\sigma_x$, $\sigma_y$, and $\sigma_z$ are the stresses at the bottom of the stabilised base layer obtained from the numerical simulation.

The backcalculated material properties of the stabilised base material and the subgrade under different test conditions are presented in Table 3. The backcalculated resilient moduli for the stabilised base layer, assuming it to be isotropic elastic, were relatively consistent for all the tests that were carried out with an average value of 2117 MPa. On the other hand, the resilient modulus for the subgrade material was very inconsistent. Comparatively, tests PMT1 and PMT6 showed very low resilient modulus for the subgrade material. The subgrade material in PMT1 was prepared freshly by compacting in multiple layers at OMC and then the stabilised base layer was constructed over it. However, there was no liquid rubber layer applied on the surface of the subgrade to prevent moisture movement between the two layers. The test was carried out after 7 days curing of the stabilised layer and, during this time, there might have been moisture movement from the subgrade to the stabilised layer. Thus, the subgrade became stiffer due to this moisture loss and indicated a relatively higher resilient modulus of 9 MPa compared to PMT6.
At the same time, the subgrade material in PMT6 was similarly prepared and compacted at OMC but a layer of liquid rubber was applied on top of the subgrade layer to prevent any moisture movement. Therefore, the subgrade layer in PMT6 had the lowest resilient modulus of 0.675 MPa. No liquid rubber being applied at the interface to prevent moisture movement in the other tests must have caused the higher resilient modulus for the subgrade. The largest resilient modulus found for the subgrade was 40.5 MPa in PMT5 (Table 3) in which the subgrade thickness was 430 mm while the thickness was 330 mm for all the other tests. It is noted that the base layer also showed higher resilient in PMT5 than in other tests, apparently due to increased hydration and consequent higher stiffness/strength gain. It appears that the stiffness of both the subgrade and base increased with higher subgrade thickness suggesting that it is preferable to carry out model tests with highest possible subgrade thickness. However, this involves processing larger volume of subgrade material and consequent higher labour cost and time. To ensure the consistency of test results through preventing moisture movement between subgrade and stabilised granular base, a thin layer of liquid rubber was applied over the subgrade as soon as it is prepared in PMT6.

The estimated cohesion values of the subgrade presented in Table 3 show similar variations among the different tests as did the back calculated resilient moduli. The subgrade material in PMT1 and PMT6 tests apparently had very low cohesions of 1 kPa while the PMT2, PMT4 and PMT5 tests had the maximum cohesion of 10 kPa.

The resilient modulus ($E_h$), calculated using the horizontal tensile strain measured at the bottom of the stabilised layer, is also presented in Table 3. The results show that the modulus values were very consistent for all the tests with an average value of 1196 MPa. In addition, the results indicate that the values of the resilient moduli determined by the measured horizontal strains were less than those of the backcalculated resilient moduli when assuming
the stabilised material to be isotropic elastic. Therefore, it is inferred from this observation that the stabilised base material had cross-anisotropic resilient properties instead of isotropic elastic.

Therefore, another numerical analysis was performed using the finite difference method to backcalculate the material properties taking the stabilised base layer into account as being cross-anisotropic elastic and the subgrade layer as an elasto-plastic material with Mohr-Coulomb failure criterion. The backcalculated material properties are presented in Table 4. In this analysis, the Poisson’s ratios in both the vertical and horizontal directions were assumed to be the same, i.e., 0.2, and the cross-shear modulus was determined using the following equation, assuming the \( xy \) plane to be the plane of isotropy (Lekhnitskii 1981):

\[
G_{xy} = \frac{E_x E_y}{E_x (1 + 2\nu_{xy}) + E_y} \tag{3}
\]

where \( G_{xy} \) is the cross-shear modulus, \( \nu_{xy} \) is the Poisson's ratio characterising lateral contraction in the plane of isotropy when tension is applied in this plane and \( E_x \) and \( E_y \) are the elastic moduli in the \( x \) and \( y \) directions respectively. However, as the elastic modulus in the horizontal direction (\( xy \) plane) was the same in this analysis, the cross-shear modulus was calculated by the following simplified equation:

\[
G = \frac{E}{2(1+\nu)} \tag{4}
\]

The performance of the proposed back-calculation method was evaluated by comparing the measured and estimated parameters. Table 5 shows the measured, the estimated and the root-mean-square (RMS) error of the applied pressure and deflections at different depth levels. As
shown in Table 5, the backcalculated parameters were accurately estimated with a maximum RMS error of less than 3%. Therefore, the backcalculation analysis used in this investigation is accurate and reliable.

The results presented in Table 4 indicate that the backcalculated vertical resilient moduli of the anisotropic elastic stabilised base layer ranged from 1890 to 4140 MPa for the six tests that were conducted with an average modulus of 2875 MPa, and were higher than those calculated for the isotropic elastic base material. The horizontal resilient moduli ranged from 1337 to 1806 MPa with an average value of 1598.5 MPa. The cohesions and moduli of the subgrade material followed the similar trends as before when the stabilised base material was assumed to be isotropic elastic, but were slightly lower than those of the previous analysis. The results presented in Table 4 further indicate that the modular ratios (the ratio of the horizontal resilient modulus \(E_h\) to the vertical resilient modulus \(E_v\)) varied from 0.3758 to 0.7074. This value is the measure of the anisotropy and, as it increases towards 1, the anisotropy decreases. In this experimental program, the modular ratios of base layers in all tests are very close (i.e. between 0.5473 and 0.7074) except PMT5. The lowest modular ratio of 0.3758 was found in PMT5 in which the base layer thickness was 150 mm and the subgrade thickness 430 mm. This could be due to excessive moisture movement into base layer from subgrade as a result of increased volume of subgrade. Moreover, the modular ratio values determined in this investigation were very close to those calculated by Ashtiani et al. (2007) for four different Texas limestone aggregates lightly stabilised with 1-2 % Portland cement.

Despite the variations in subgrade clay characteristics due to moisture variations, the stiffness values of the stabilised material are very consistent for all the six tests; especially the horizontal resilient modulus. In the current mechanistic-empirical pavement design approach,
cementitiously stabilized granular materials are characterized by their flexural modulus (e.g. AUSTROADS 2004; NCHRP 2004). In these design guides, correlations were developed between the flexural modulus of cementitiously stabilised materials and the UCS. According to AUSTROADS (2004), the correlation is:

\[ E_{\text{flex}} = k \times UCS \]  

(5)

\( k \) is a constant of 1,000 to 1,250, typically used for GP cement. According to equation (5) the flexural stiffness should be in the range of 1200 MPa to 1500 MPa for a UCS of 1.2 MPa. From this investigation the average flexural or horizontal resilient modulus was 1598.5 MPa, which is slightly outside the limit. This small deviation is expected because there is a confining effect/pressure in the PMTF while there is no confining pressure in a flexural beam test.

The backcalculated average horizontal and vertical resilient moduli of the stabilised base layer from the PMFT are 2.75 and 4.96 times bigger than the stiffness from the cyclic load IDT test respectively. However, more test results are needed with varying binder contents to develop correlations between PMFT and IDT resilient moduli.

Changes in environmental conditions such as moisture content or temperature have not been taken into account for this particular numerical analysis. The material layers were prepared at their OMCs but the moisture contents of the materials during testing or their variation with time were not measured. Therefore, the changes in environmental conditions were not modelled in this analysis. Thus, the application of the developed model is limited to changes in environmental conditions.
Conclusions

A cyclic load PMTF was developed recently for determining stiffness characteristics of different materials used in a pavement structure. In particular, a typical pavement structure consisting of a lightly stabilised granular road base material over an expansive soft clay subgrade material was studied using the PMTF test facility under traffic type cyclic loading conditions.

In the PMTF test investigation, the model pavement structure was constructed in a 1.0 m x 1.1 m x 0.6 m steel tank and the wheel loading from the vehicle was simulated using a circular steel plate subjected to sinusoidal type axial loading at a frequency of 3 Hz through an actuator assembly. The tensile strain at the bottom of the stabilised material layer was monitored with dynamically rated special strain gauges and the vertical deformation at different depth levels was monitored with purpose made settlement rods made of plastic.

Based on the experimental results, the resilient modulus of the pavement materials were backcalculated through a numerical simulation analysis using a finite difference computer program. The analysis indicates that the stabilised base material had cross-anisotropic resilient properties. The results from the numerical analysis further indicate that the backcalculated vertical resilient moduli of the anisotropic elastic stabilised base layer ranged from 1890 to 4140 MPa for the six tests that were conducted with an average modulus of 2875 MPa. The horizontal resilient moduli ranged from 1337 to 1806 MPa with an average value of 1598 MPa. Also, the calculated flexural modulus from UCS of the stabilised granular material according to AUSTROADS (2004) is very close to the backcalculated horizontal resilient modulus from PMTF in this investigation.
Based on the results and error analysis discussed in this paper, it was concluded that the resilient moduli of the stabilized granular base layer and subgrade clay can be backcalculated accurately and reliably from the analysis using FLAC3D finite difference numerical modelling method. However, this numerical model has limitations for changes in environmental conditions as they were not incorporated in the model.

Acknowledgements

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References


Table 1 Summary of testing program for PMTF test

<table>
<thead>
<tr>
<th>Test code</th>
<th>Stabilised base thickness / (mm)</th>
<th>Subgrade thickness / (mm)</th>
<th>Maximum cyclic pressure / (kPa)</th>
<th>Rubber membrane at the interface</th>
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<tbody>
<tr>
<td>PMT1</td>
<td>250</td>
<td>330</td>
<td>1000</td>
<td>No</td>
</tr>
<tr>
<td>PMT2</td>
<td>250</td>
<td>330</td>
<td>1000</td>
<td>No</td>
</tr>
<tr>
<td>PMT3</td>
<td>250</td>
<td>330</td>
<td>1250</td>
<td>No</td>
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<tr>
<td>PMT4</td>
<td>150</td>
<td>330</td>
<td>1500</td>
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</tr>
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<td>PMT5</td>
<td>150</td>
<td>430</td>
<td>1500</td>
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<tr>
<td>PMT6</td>
<td>150</td>
<td>330</td>
<td>1500</td>
<td>Yes</td>
</tr>
</tbody>
</table>
Table 2 Basic geotechnical properties of the materials

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Subgrade</th>
<th>Granular (parent) material</th>
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</thead>
<tbody>
<tr>
<td>MDD (kg/m³)</td>
<td>1342</td>
<td>2270</td>
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<tr>
<td>OMC (%)</td>
<td>32</td>
<td>9.25</td>
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<td>Lab soaked CBR</td>
<td>1.5</td>
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<td>Liquid limit (%)</td>
<td>93.4</td>
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<tr>
<td>Plastic limit (%)</td>
<td>37.0</td>
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<tr>
<td>Plasticity index (%)</td>
<td>56.4</td>
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<td>Linear shrinkage (%)</td>
<td>24.4</td>
<td>3.8</td>
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Table 3 Backcalculated material properties for isotropic elastic base

<table>
<thead>
<tr>
<th>Test</th>
<th>Backcalculated modulus, E (MPa)</th>
<th>Cohesion of the subgrade material (kPa)</th>
<th>Modulus of base layer by horizontal strain measurement, $E_h$ (MPa)</th>
<th>$E/E_h$</th>
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<tr>
<td>Base layer</td>
<td>Subgrade</td>
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<td></td>
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<td>PMT1</td>
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<td>1</td>
<td>1394</td>
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<td>PMT2</td>
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Table 4 Backcalculated material properties for anisotropic elastic base material

<table>
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<tr>
<th>Test</th>
<th>Backcalculated Modulus (MPa)</th>
<th>Cohesion of the subgrade (kPa)</th>
<th>$E_h/E_v$</th>
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<tbody>
<tr>
<td></td>
<td>Base layer</td>
<td>Subgrade</td>
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</tr>
<tr>
<td></td>
<td>$E_v$</td>
<td>$E_h$</td>
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Table 5 Root-mean-square (RMS) percentage error for measured and estimated parameters

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<tr>
<th>Test</th>
<th>Pressure (kPa)</th>
<th>Vertical deformation of the loading plate (mm)</th>
<th>Vertical deformation at the interface (mm)</th>
<th>Vertical deformation at half depth of base layer (mm)</th>
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<td>Estimated</td>
<td>Error (%)</td>
<td>Measured</td>
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<td>0.00</td>
</tr>
</tbody>
</table>

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6 = pavement model test tank  
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Figure 8

Average Vertical Stress under the Loading Plate (kPa)

Deformation of the Loading Plate (mm)