USING A SHAKEDOWN APPROACH AS A SIMPLE MEANS OF PREDICTING RUTTING IN UNSEALED AND CHIP-SEALED PAVEMENTS

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ABSTRACT

The shakedown approach was originally developed to describe the behaviour of metal in pressure vessels subjected to repeated stress cycling. Although the aggregate which comprises the base course of pavements is a very different material, it is shown by reference to laboratory tests that it exhibits many of the same features of behaviour as predicted by the classical shakedown model. Several laboratory test results are presented and the shakedown interpretative framework is applied to them in order to show its application and limitations.

These results show that there are some differences between granular materials and the original model. These must be taken into account before the shakedown model can be satisfactorily used to replicate the behaviour of granular material under repeated loading.

With this proviso, it is shown that the modified shakedown model can be used to predict whether aggregate will undergo stabilising, slow continual or rapid plastic straining under the application of traffic loading. Thus the technique can be used to predict whether or not the aggregate will undergo acceptable or unacceptable shear movement and hence to predict whether the pavement will suffer from excessive rutting.

To predict the amount of rutting which will take place requires extrapolation of the Repeat Load Tri-axial (RLT) test at the worst stress condition encountered in the pavement. Further, the expected rutting response of the pavement is categorised into 3 Ranges (A, B or C). If a Range A response is predicting then rutting is minimal and is stabilising. A Range C response with indicate likely pre-mature shear failure. It is concluded that the technique has value as a predictive tool to ascertain whether or not rutting will be of concern to the road owner but that the magnitude of rutting which may arise cannot be predicted with such great accuracy at present.

INTRODUCTION

Most current pavement thickness design guides [HMSO 1994, TRL 1993, Austroads 1992] assume that rutting occurs only in the subgrade. The thickness of the unbound granular sub-base layers is determined from the subgrade condition (California Bearing Ratio and/or resilient modulus) and design traffic (including traffic during construction). The assumption that rutting occurs only within the sub-grade is assumed to be assured through the requirement of the unbound granular materials (UGMs) to comply with material specifications. These specifications for UGMs are recipe based and typically include criteria for aggregate strength, durability, cleanliness, grading and angularity, none of which is a direct measure of resistance to rutting caused by repeated loading. Permanent strain tests in the Repeat Load Tri-axial (RLT) apparatus commonly show a wide range of performances for granular materials even though all comply with the same specification [Thom and Brown 1989]. Accelerated pavement tests show the same results and also report that 30% to 70% of the surface rutting is attributed to the UGM’s layers [Little 1993 and Pidwerbesky 1996].

Furthermore, recycled aggregates and other materials considered suitable for use as unbound sub-base pavement layers can often fail the highway agency material specifications and thus restrict their use. There is potential of the permanent strain test in the RLT (or similar)
apparatus to assess the suitability of these alternative materials for use at various depths within the pavement (e.g. sub-base and lower sub-base). Thus, current pavement design methods and material specifications should consider the repeated load deformation performance of the UGM layers. Thus, as an approach to overcome the limitations of current practice, the shakedown concept was used as a design method to predict the rutting potential of unbound granular materials in pavements. This design method utilises results from permanent strain tests and stresses computed using a multi-layered linear elastic program CIRCLY (Wardle, 1980).

SHAKEDOWN CONCEPT

The performance of UGMs in permanent strain RLT tests is highly non-linear with respect to stress. There are a range of permanent strain responses to stress level and load cycles that cannot be described by a single equation. Several researchers [Wekmeister et al 2001, Sharp and Booker 1984] who related the magnitude of the accumulated permanent (plastic) strain to shear stress level concluded that the resulting permanent strains at low levels of additional stress ratio, $\Delta \sigma_1/\sigma_3$, eventually reach an equilibrium state after the process of post-compaction stabilisation (i.e. no further increase in permanent strain with increasing number of loads). At slightly higher levels of additional stress ratio, however, permanent deformation does not stabilise and appears to increase linearly. For even higher levels of additional stress ratio, however, permanent deformation increases rapidly and results in failure of the specimen. These range of behaviours can be described using the shakedown concept.

Dawson and Wellner [1999] have applied the shakedown concept to describe the observed behaviour of UGMs in the RLT permanent strain test. The results of the RLT permanent strain tests are reported as either shakedown range A, B or C. This allows the determination of stress conditions that cause the various shakedown ranges for use in determining the rutting potential of the pavement. The shakedown ranges are:

- **Range A** is the plastic shakedown range and for this to occur the response shows high strain rates per load cycle for a finite number of load applications during the initial compaction period. After the compaction period the permanent strain rate per load cycle decreases until the response becomes entirely resilient and no further permanent strain occurs. This range occurs at low stress levels and Werkmeister et al [2001] suggest that the cover to UGMs in pavements should be designed to ensure stress levels in the UGM will result in a Range A response to loading.

- **Range B** is the plastic creep shakedown range and initially behaviour is like Range A during the compaction period. After this time the permanent strain rate (permanent strain per load cycle) is either decreasing or constant. Also for the duration of the RLT test the permanent strain is acceptable and the response does not become entirely resilient. However, it is possible that if the RLT test number of load cycles were increased to perhaps 2 million load cycles the result could either be Range A or Range C (incremental collapse).

- **Range C** is the incremental collapse shakedown range where initially a compaction period may be observed and after this time the permanent strain rate increases with increasing load cycles.

Cumulative permanent strain versus permanent strain rate plots can be used to aid in determining the shakedown range [Werkmeister et al 2001]. An example plot is shown in Figure 1 [Figure 3, Werkmeister et al 2001]. Range A response is shown on this plot as a vertical line heading downwards towards very low strain rates and virtually no change in cumulative permanent strain. The horizontal lines show a Range C response where the strain rate is not changing or is increasing and cumulative permanent strain increases. A Range B response is between a Range A and Range C response, typically exhibiting a constant, but very small, plastic strain rate per cycle.
RLT PERMANENT STRAIN TESTS

A series of Repeated Load Tri-axial (RLT) permanent strain tests were conducted on two Northern Ireland unbound granular materials (UGMs). The two aggregates chosen for testing were located at the same quarry. They are both greywackes, where one is of lower quality and has a slight red colour. These aggregates are later referred to as NI Good and NI Poor. The greywacke, according to BS 812 Part 1: 1975 is part of the gritstone group of aggregates that embraces a large number of sandstone type deposits. Greywacke is one of the major sources of coarse aggregate in Northern Ireland. The aim was to determine the range of stress conditions that cause the various shakedown range responses either A, B or C. In the RLT permanent strain tests the cell pressure (confinement) was held constant while the vertical load was cycled.

Tests

To assist in setting the RLT permanent strain test stress conditions, tri-axial static shear-failure tests were undertaken at different cell pressures. The maximum yield stress results were plotted in $p$ (mean normal) – $q$ (deviatoric) stress space (Figure 2). Plots of this type for granular materials typically result in straight lines in $p – q$ stress space and may be referred to as Drucker-Prager yield criteria.

It was assumed that RLT permanent strain tests conducted at stress levels close to the yield line plotted in $p – q$ stress space would result in the highest deformations. Further, permanent strain results were needed over the full range of stress conditions that occur within a pavement. On this basis stress levels were chosen by keeping $p$ constant and varying $q$. Figures 3 and 4 detail in $p – q$ stress space the stress levels and paths used in the permanent strain tests, where $q$ (principle stress difference) is also the cyclic vertical deviator stress.

The initial testing reported in this paper were screening tests designed to determine the best test conditions for future research in determining the stress conditions that cause the various shakedown ranges. These screening tests aimed to cover the full range of stress conditions by using the same sample for tests at different stress levels. For the same sample $p$ (mean normal stress) was kept constant while $q$ (principle stress difference) was increased for each subsequent test of 50,000 load cycles. This method of testing allows the full spectra of stresses to be tested while only using 3 samples. As stress history is likely to affect results ideally a new sample is required for each new stress level. The affects of stress history will be investigated in the future.

Results

For each material three multi-stage RLT permanent strain tests were conducted. The results were analysed to determine cumulative deformation as shown for one test in Figure 5. As discussed to determine which stress level caused the various shakedown ranges (A, B or C) cumulative permanent strain versus permanent strain rate plots were produced. For calculation of cumulative permanent strain it was assumed that at the start of each new test stage the deformations were nil (or zero). Figure 6 shows the cumulative permanent strain plot versus permanent strain rate for the test results shown in Figure 5.

At each test stress level the permanent strain response in terms of shakedown range (A, B or C) was estimated. For example, the data of Figure 6 indicates that the 2b, 2c and 2d results are of type B. The 2a result hasn’t had enough cycles of loading to fully stabilise as illustrated in Figure 1, but is rapidly doing so. Thus it is of Type A response. The 2f result is Type C while 2e appears to lie somewhere on a B/C transition. Thus the stress levels associated with the boundaries between A and B and between B and C response and between B and C response can be defined. The stress levels at the boundary between the shakedown ranges were plotted in $p – q$ stress space. For comparison the yield line was also plotted and the results are shown in Figures 5 and 6. From the results it can be seen that the boundary between shakedown ranges A and B is significantly below the yield line and nearly parallel. Stresses that cause a shakedown range A response in the UGM where stable behaviour results are ideal from a design.
perspective. The stress boundary between shakedown responses B and C are close to the yield line and stresses this high should be avoided in the UGM pavement layer to avert pre-mature failure in this layer.

PREDICTING RUTTING POTENTIAL

The design process proposed is to categorise the expected performance of the pavement in terms of either a Range A, B or C type response. A Range A response is preferred as this indicates that rutting in the unbound granular material (UGM) is minimal and is stabilising (i.e. rate of rutting is decreasing with increasing load cycles). Range B response is satisfactory provided further analysis is undertaken to check the estimated rut depth for the design traffic is less than that acceptable. However, a Range C response predicts early failure and therefore the pavement should be re-designed. For example, the pavement design can be altered by either increasing the asphalt cover, demoting the UGM layer to a lower sub-base or capping layer, or replacing the UGM with a better quality UGM. For this research project this design process has been applied to the pavement design of a field trial constructed in November 2001 in Ballyclare, Northern Ireland, UK. The field trial uses the two different UGMs tested (NI Good and NI Poor) where the asphalt cover thickness varies from 40 to 100mm. Several design checks were undertaken for each UGM and asphalt cover thicknesses of 0, 40, 60, 80 and 100mm. Figure 9 shows the field trial cross-section and material parameters ($E$, $\nu$) used for design are detailed in Table 1.

Table 1. Field trial pavement design material parameters.

<table>
<thead>
<tr>
<th>Material Parameters</th>
<th>NI Good</th>
<th>NI Poor</th>
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<tbody>
<tr>
<td>Asphalt:</td>
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<td></td>
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<tr>
<td>Thicknesses, d (mm)</td>
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<tr>
<td>Thickness (mm)</td>
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Design

A linear elastic analysis using CIRCLY was undertaken for a range of asphalt cover thicknesses for the pavement shown in Figure 9. Principle stresses directly under the centreline of the wheel load were computed and transferred into Microsoft Excel. From these stresses the average principle stress ($p$) and principle stress difference ($q$) was calculated covering the full range of pavement depth. For each asphalt cover thickness these stresses were plotted in $p$-$q$ stress space alongside the Range A, B and C boundaries determined for the NI Good and NI Poor aggregates. Figure 10 shows the initial results and it predicts the undesirable Range C behaviour to occur asphalt cover less than 80 mm for the NI Poor aggregate.

Closer examination of the results show for an asphalt thickness of 100 mm the NI Good aggregate will exhibit Range B behaviour while the NI Poor aggregate will perform in the stable Range A category. This result is against better judgement and is due to the limitations of a linear elastic analysis where tensile stresses are computed in the unbound material. Further, residual stresses due to compaction and active earth pressure was not considered. Further analysis was therefore undertaken assuming a residual stress of 30kPa. This residual stress has the effect of increasing $p$ and reducing $q$ with the effect of increasing the strength of the UGM. Principle stresses were computed under the wheel load centre for a range of asphalt cover thicknesses to illustrate the effect including a horizontal residual stress of 30 kPa using the design parameters in Table 1 and a tyre pressure $P$ of 550kPa. Figure 11 illustrates in $p$-$q$ stress space the effect of including residual stresses in relation to the Range A, B and C boundaries for the NI Good and NI Poor UGMs.
To predict rutting a conservative approach is to determine the most damaging $p-q$ stress values in the pavement from the CIRCLY analysis. This point in the $p-q$ stress plot will be the closest to the Range B and C boundary. The RLT permanent strain result at stress conditions the same or close to the most damaging stresses determined is used to predict rutting. The RLT permanent strain result is then extrapolated to obtain a permanent strain value for the design traffic. If the RLT result is categorised as Range B then a linear extrapolation is used otherwise for Range A an exponential function is used. The rut depth can then be predicted by multiplying the permanent strain value by the aggregate depth.

Results

The results of the pavement analysis of the field trial were a prediction as to the range of rutting behaviour (Range A, B or C) that would occur in the UGM for different asphalt cover thicknesses and aggregate type (NI Good and NI Poor). Material parameters used and layer thicknesses are detailed in Table 1 and Figure 9. Predictions from the CIRCLY analysis (assuming a horizontal residual stress of 30 kPa) are shown in Figure 11.

The results show that stable Range A behaviour occurs for asphalt thicknesses greater than 40 mm for the NI Good and 80 mm for the NI Poor aggregates. Range C (pre-mature failure) does not occur for the NI Good aggregate while Range C is predicted to occur for asphalt thicknesses less than 40 mm for the NI Poor aggregate.

CONCLUSIONS

RLT permanent strain test results for two Northern Ireland unbound granular materials UGM (NI Good and NI Poor) showed a range of responses. These responses were categorised into three possible shakedown ranges A, B and C. Shakedown range A is where the rate of cumulative permanent deformation decreases with increasing load cycles until the response is purely elastic. Range C is incremental collapse or failure and range B is between A and C responses. The RLT test stresses at the boundaries between shakedown ranges A, B and C were plotted in $p$ (mean normal) – $q$ (deviatoric) stress space. Best fit lines were then derived to define the stress boundaries. These stress boundaries defining the limits of Range A, B and C rutting behaviour were then plotted along side stresses within the pavement computed from CIRCLY. In summary it was shown that:

- A practical method for determining design criteria from RLT permanent tests on UGM materials is possible and that this can be applied to stresses computed using a linear elastic program like CIRCLY for prediction of likely rutting response;
- The predictions of asphalt cover needed to ensure stable (Range A) behaviour in the UGM for the field trial appeared reasonable using the CIRCLY stresses and behaviour ranges from RLT tests;
- A residual stress of 30 kPa due to the effects of compaction and active earth pressure need to be applied to the CIRCLY stresses determined within the granular material to ensure reasonable results.

REFERENCES


Little, Peter H., (1993) The design of unsurfaced roads using geosynthetics, Dept. of Civil Engineering, University of Nottingham.


Wardle, L, 1980, *Program CIRCLY, a computer program for the analysis of multiple complex circular loads on layered anisotropic media*.

Figure 1. Cumulative permanent strain versus strain rate plot showing shakedown ranges [Werkmeister et al 2001].

Figure 2. Yield stresses from tri-axial static shear-failure tests plotted in p – q stress space.
Figure 3. RLT permanent strain testing stresses for NI Good UGM.

Figure 4. RLT permanent strain testing stresses for NI Poor UGM.
Figure 5. Typical RLT permanent strain test result for NI Good UGM.

Figure 6. Cumulative permanent strain versus permanent strain rate for NI Good UGM test 2 (Figure 5).
Figure 7. Shakedown range stress boundaries for NI Good UGM.

Figure 8. Shakedown range stress boundaries for NI Poor UGM.
Figure 9. Pavement cross-section of proposed field trial.

Figure 10. Centreline stresses in UGM caused by a contact tyre stress of 550kPa for a range of asphalt cover thicknesses.
Stresses in UGM for range of asphalt cover (AC) thicknesses (horizontal residual stress = 30 kPa)

Figure 11. Centreline stresses in UGM caused by a contact tyre stress of 550 kPa for a range of asphalt cover thicknesses, where horizontal residual stresses are assumed = 30 kPa