Serviceability Design of Granular Pavement Materials

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**Keywords:** repeat load tri-axial, unbound granular material, permanent deformation, pavement design.

**ABSTRACT:** The Road authority specifications for unbound granular materials (UGMs) do not typically include a direct measure of resistance to rutting caused by repeated loading yet. These specifications can preclude the use of locally available aggregates or recycled materials that may provide adequate performance (resistance to rutting). As an alternative or compliment to current UGM assessment methods via specifications, a serviceability design method is proposed that utilises results from tests in the repeated load tri-axial (RLT) apparatus. Permanent strain behaviour at a range of stress conditions of four unbound granular materials (UGM) were assessed in the RLT apparatus. Behaviour was categorised into 3 possible ranges A, B or C, where A is a stable response and C is incremental collapse while B is intermediate. From this data, test stresses near or at the boundary of range A and B were determined to define a serviceability limit line stress boundary in p (mean normal stress) – q (principle stress difference) stress space. The serviceability limit line was applied as a yield criteria in a finite element model of a pavement to predict whether or not stable (Range A) behaviour occurs in the UGM for a range of asphalt cover thicknesses.

1 **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 usually 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.

The repeated load tri-axial (RLT), hollow cylinder (Chan 1990) and k-mould (Semmelink et al, 1997) apparatuses can in various degrees simulate pavement loading on soils and granular materials. Permanent strain tests 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 on the basis of performance under trafficking can often fail the highway agency material specifications. There is potential for 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 serviceability design concept was used to determine appropriate pavement design parameters from permanent strain tests of UGMs. These design parameters were then used in an ABAQUS Finite Element Model to define the linear Drucker-Prager yield criteria for the UGM which is used to predict whether or not stable behaviour occurs in the UGM layer.

2 PERMANENT STRAIN BEHAVIOUR

The performance of UGMs in permanent strain RLT tests is highly non-linear with respect to stress. There is a range of permanent strain responses to different stress levels and numbers of load cycles that cannot be described by a single equation. Several researchers (Werkmeister 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, \( \Delta \sigma /\sigma_0 \), 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 categorised as either Range A, B or C (Dawson and Wellner, 1999):

- **Range A** is the *plastic stable range* and for this condition 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 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, constant or slightly increasing. 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 range* where initially a compaction period may be observed and after this time the permanent strain rate increases or remains constant (but at a rather high magnitude of plastic strain per cycle) with increasing load cycles.

- **Ranges A, B and C** are illustrated, diagrammatically, in Figure 1.

3 RLT PERMANENT STRAIN TESTS

A series of Repeated Load Tri-axial (RLT) permanent strain tests were conducted on four unbound granular materials (UGMs) from Northern Ireland and Germany. The two UGMs from Northern Ireland 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 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. A sandy gravel and a granodiorite were tested from Germany and were taken from the test pit at Dresden University of Technology, Pavement Engineering Division.

The aim was to determine the range of stress conditions that cause the various behaviour 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.

![Diagram showing Plastic Strain Behaviours](image)

Figure 1. Indicative Plastic Strain Behaviours

3.1 **RLT TEST CONDITIONS**

3.1.1 *Northern Ireland UGMs*
To assist in setting the RLT permanent strain test stress conditions for the Northern Ireland UGMs, 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. 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 for each test on the same specimen were chosen by keeping the cell pressure constant and varying $q$. Figure 2 details in $p$ – $q$ stress space the stress levels and paths used in the permanent strain tests for the *NI Good* aggregate, where $q$ 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 behaviour 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 $p$ was kept constant while $q$ was increased for each subsequent test of 50,000 load cycles at a frequency of 5 Hz. This method of testing allows the full spectrum 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.

3.1.2 *German UGMs*
For the repeated load tri-axial loading tests on the UGMs from Germany the constant confining pressure was set at levels of 40 (Sandy Gravel only), 70, 140, 210 and 280 kPa. After the confining pressure had been reached, additional dynamic (frequency = 5 Hz) vertical stress (deviator stress)
was applied. The tri-axial tests were carried out with axial stress pulses reaching stress ratios of $\Delta \sigma_1/\sigma_3 = 0.5 - 11$.

![Figure 2: RLT permanent strain testing stresses for NI Good UGM.](image)

3.2 **Results**

To determine which stress level caused the various behaviour ranges (A, B or C) cumulative permanent strain versus permanent strain rate plots were produced (e.g. Figure 3). For calculation of cumulative permanent strain it was assumed that at the start of each new test stage the deformations were nil (or zero). Only one stress level was applied per sample for the German UGMs. Figure 4 shows the cumulative permanent strain plot versus permanent strain rate for the typical test result for the **NI Good UGM**.

At each test stress level the permanent strain response in terms of behaviour range (A, B or C) was estimated. For example, the data of Figure 4 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, 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. Behaviour Ranges for other RLT permanent strain tests including the German materials were determined in a similar manner. Results are shown in Figures 5, 6, 7 and 8. Stresses that cause Range A response in the UGM where stable behaviour results are ideal from a design perspective. The stress boundary between responses B and C are close to the yield line and stresses this high should be avoided in the UGM pavement layer to avert premature failure in this layer.
Figure 3. Typical RLT permanent strain test result for *NI Good* UGM.

Figure 4. Cumulative permanent strain versus permanent strain rate for *NI Good* UGM test 2 (Figure 2).
Figure 5. Shakedown range stress boundaries for *NI Good* UGM.

Figure 6. Shakedown range stress boundaries for *NI Poor* UGM.
Permanent Strain RLT Test Stresses and Associated Behaviour Ranges A, B and C - *German Granodiorite*

![Diagram showing stress boundaries for German Granodiorite](image)

Figure 7. Shakedown range stress boundaries for *German Granodiorite* UGM.

Permanent Strain RLT Test Stresses and Associated Behaviour Ranges A, B and C - *German Sandy Gravel*

![Diagram showing stress boundaries for German Sandy Gravel](image)

Figure 8. Shakedown range stress boundaries for *German Sandy Gravel* UGM.
4 PAVEMENT DESIGN OF UGM LAYER

The design process proposed is simply a check as to whether or not stabilised behaviour (Range A) will occur in the UGM pavement layer. Should Range A behaviour not occur then the pavement is re-designed and a further check as to whether or not Range A behaviour occurs is undertaken. 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 for asphalt cover thicknesses of 0, 40, 60, 80 and 100mm as to whether or not Range A behaviour occurs were undertaken for each Northern Ireland UGM and for comparison the two German UGMs using the ABAQUS computational technique. Figure 8 shows the field trial cross-section and the elastic material parameters ($E$, $v$) used for determining stresses within the pavement used for the design check.

4.1 Finite Element Modelling

The finite element model (FEM) developed for the pavement (Figure 9) in ABAQUS is not complicated, at this stage, as it simply computes principle stresses from linear elastic theory spatially within the pavement. These stresses are plotted in Microsoft Excel in $p$-$q$ stress space along with the Range A stress boundary limit line. Should the stresses at any point meet or exceed the Range A limit line then the design is unacceptable.

Linear elastic analysis in the FEM includes an estimated residual stress (due to compaction, active earth pressure and as a consequence of yield) in the UGM pavement layer of 30kPa. This residual stress has the effect of increasing $p$ and reducing $q$ with the effect of increasing the strength of the UGM. If residual stresses were not included the result is unrealistic. Asphalt cover thicknesses (> 160 mm) would be required whereas the NI Good UGM has been used successfully in Northern Ireland for asphalt cover thicknesses of 50 mm. Principal stresses were computed under the wheel load centre for a range of asphalt cover thicknesses to illustrate the effect of including a horizontal residual stress of 30 kPa using the design parameters in Figure 9 and a tyre pressure $P$ of 550kPa. Figure 10 illustrates in $p$-$q$ stress space the effect of including residual stresses in relation to the Range A limit lines for the NI Good, NI Poor, Granodiorite and Sandy Gravel UGMs.

Assuming the stiffnesses of all the aggregates to be the same ($E=200$ MPa, $v=0.35$), the results from Figure 10 show the German Granodiorite to be the superior material where for a thin asphalt cover of less than 40mm it is predicted that stable behaviour (Range A) will result. At least 60 mm of AC cover is required over the NI Good and German Sandy Gravel UGMs. About 70mm of AC is required over the NI Poor UGM.

Further refinements will be to design the pavements including material and moisture-specific stiffness data and to use an analytical method to deduce the horizontal stresses (e.g. the Upper Bound approach proposed by Collins & Boulbibane (1998) and Boulbibane & Collins (1998) and the Lower Bound approach proposed by Yu & Hossain (1998)).

5 CONCLUSIONS

RLT permanent strain test results for two Northern Ireland unbound granular materials UGM (NI Good and NI Poor) and two German UGMs (Granodiorite and Sandy Gravel) showed a range of responses. These responses were categorised into three possible behaviour ranges A, B and C. 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 boundary between range A and B were plotted in $p$ (mean normal) – $q$ (deviatoric) stress space. A best fit line was then derived to define
the stress boundary or *Range A limit line* between stable (acceptable) behaviour and unstable behaviour. This *Range A limit line* was then used to predict whether or not stable behaviour (Range A) occurs for the field trial pavement for a range of asphalt cover thicknesses. In summary it was shown that:

![Figure 8. Pavement cross-section of proposed field trial.](image)

![Figure 10. Range A limit lines and centreline stresses in UGM caused by a contact tyre stress of 550kPa for a range of asphalt cover thicknesses, where horizontal residual stresses are assumed = 30 kPa.](image)
• A practical method for determining design criteria from RLT permanent tests on UGM materials is possible and that this can be applied as a limit line in p-q stress to check whether stresses computed within the pavement are acceptable (i.e. fall below this limit line);

• The predictions of asphalt cover needed to ensure stable behaviour in the UGM for the field trial appeared reasonable using the finite element model to predict stresses and the limit line criteria. The horizontal stress assumptions are, however, critical;

• Each UGM material is unique and requires different thicknesses of asphalt cover to achieve optimum results. Further, the NI Poor UGM is predicted to perform satisfactorily if a asphalt cover of at least 70mm is used although it use in Northern Ireland is currently disallowed due to non-compliance with current specifications;

• Initial results suggest that it is possible to use multi-stage testing in the RLT apparatus to determine the permanent strain results at a range of stress conditions rather than using a new specimen for each new stress condition tested. However, this conclusion is still to be validated by further tests on separate samples in the RLT apparatus.

6 REFERENCES


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