



Pavements Unbound

Edited by Andrew R. Dawson

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PAVEMENTS UNBOUND

PROCEEDINGS OF THE 6th INTERNATIONAL SYMPOSIUM ON PAVEMENTS
UNBOUND (UNBAR 6), 6–8 JULY 2004, NOTTINGHAM, ENGLAND

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Edited by

Andrew R.Dawson

*Nottingham Centre for Pavement Engineering,
University of Nottingham, England*



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Table of Contents

Introduction	ix

Laboratory testing and granular material behaviour

Development of a simplified test for unbound aggregates and weak hydraulically bound materials utilising the NAT <i>J.P.Edwards, N.H.Thom & P.R.Flemming</i>	2
Assessment of the effect of seasonal variations on the unbound materials of low volume roads by laboratory testing <i>P.Kolisoja, N.Vuorimies & T.Saarenketo</i>	13
Shear strength and permanent deformation of unbound aggregates used in Brazilian pavements <i>W.P.Núñez, R.Malysz, J.A.Ceratti & W.Y.Y.Gehling</i>	27
Modeling of material crushing in granular road bases <i>S.Lobo-querrero & L.E.Vallejo</i>	38
Fractal analysis of the abrasion and crushing of gravels <i>L.E.Vallejo, Z.Chik, S.Tucek & B.Caicedo</i>	50
Comparative analysis of compaction procedures of unbound traditional and non-conventional materials <i>M.Pasetto & N.aldo</i>	61
Cyclic plasticity based model for flexible pavements <i>C.Chazallon & F.Allou</i>	74
Fundamental study on permanent deformation analysis of granular base course material using elasto-plastic model <i>Y.Takeuchi, T.Nishizawa, K.Endo, M.Koyanagawa & T.Maki</i>	85
Shakedown analysis of unbound road pavements—an experimental point of view <i>P.S.Ravindra & J.C.Small</i>	97

Pavement performance, evaluation and management

Damage law exponents for thin surfaced granular pavements <i>G.Arnold, D.Alabaster & B.Steven</i>	108
Behaviour of granular materials: field results versus numerical simulations <i>J.M.C.Neves & A.Gomes Correia</i>	120

Test of the influence from mica and LWA on permanent deformations and calculation of the elastic and permanent response under HVS testing <i>P.Ekdahl, J.Hansson, A.Huvstig & H.Thorén</i>	132
Influence of spring thaw on pavement rutting <i>V.Janoo & S.Shoop</i>	143
Application of acceleration measurement method for estimating the stiffness of unbound aggregates in roadbed <i>M.Kamiura & S.Nakayaka</i>	156
Performance testing of unbound materials within the pavement foundation <i>B.Rahimzadeh, M.Jones, B.Hakim & N.Thom</i>	169
Neural network-based structural models for rapid analysis of flexible pavements with unbound aggregate layers <i>H.Ceylan, A.Guclu, E.Tutumluer, M.R.Thompson & F.Gomez-Ramirez</i>	177
Measurement of road performance and impact on transportation operations with the <i>Opti-Grade</i> system <i>S.Mercier, M.Brown & Y.Provencher</i>	189
Adaptation of a grading management system for unsealed road networks in New Zealand <i>R.A.Douglas, S.A.Mitchell & B.D.Pidwerbesky</i>	198

Design of thin and unsealed pavements

Deformation behaviour of granular pavements <i>G.Arnold, A.Dawson, D.Hughes & D.Robinson</i>	213
A simplified method of prediction of permanent deformations of unbound pavement layers <i>A.El abd, P.Hornych, D.Breysse, A.Denis & C.Chazallon</i>	225
Simplified model based on the shakedown theory for flexible pavements <i>T.Habiballah, C.Chazallon & P.Hornych</i>	240
Empirical shear strength models for unbound road-building materials <i>H.L.Theyse</i>	250
Design criteria of granular pavement layers <i>S.Werkmeister, F.Wellner, M.Oeser & B.Moeller</i>	263
Design of low-volume roads in Lithuania <i>D.Zilioniene, D.Cygis & A.A.Juzenas</i>	276
Mechanistic-empirical design models for pavement subgrades <i>H.L.Theyse</i>	289
A timber piled road over deep peat in North West Ireland <i>T.Ryan, C.McGill & P.Quigley</i>	301

Recycled and secondary materials

The performance of an experimental road constructed from quarry waste <i>L.R.de Rezende & J.C.de Carvalho</i>	312
A laboratory study of the early life performance of a slag bound base <i>N.H.Thom, O.Wood & N.Ghazireh</i>	324
The use of recycled aggregates in slag bound mixtures <i>N.Ghazireh & H.L.Robinson</i>	333
Load-deformation behavior of fly-ash and bottom-ash capping and fill layers based on FWD deflection measurements <i>M.S.Hoffman</i>	344

Stabilisation

Laboratory and in situ evaluation of stabilisation of limestone aggregates using lime <i>P.Hornych, O.Hameury, M.Kergoët & D.Puiatti</i>	360
Rehabilitation of Unbound Pavements using foamed bitumen stabilisation <i>J.D.Jones & J.M.Ramanujam</i>	373
Strength and swelling properties of Oxford Clay stabilized with wastepaper sludge ash <i>J.M.Kinuthia, R.M.Nidzam, S.Wild & R.B.Robinson</i>	387
Unsealed GeoCrete-road with high bearing capacity <i>C.van Gurp & B.Kroesen</i>	398

Aggregate supply and specification

Aggregate supply and performance issues, Auckland, New Zealand <i>P.Black</i>	410
Unbound mixtures for pavement layers—BS EN 13285 <i>D.Rockliff & R.Dudgeon</i>	416
Material and performance specifications for wearing-course aggregates used in forest roads <i>G.Légère & S.Mercier</i>	427
An end product specification for road foundations <i>B.C.J.Chaddock & D.B.Merrill</i>	439
Author index	453
Subject index	456

Pavements Unbound—Dawson (ed.)

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Introduction

Since the UNBAR series of symposia were started by my former colleague, Dr Ron Jones, in 1981 there has been a great deal of change in the use of aggregates in roads. The drivers of this change are many and various, but the following, seem to me, of particular importance:

- Environmental concerns, reflected by increasingly voluminous legislation, increasing the pressure to incorporate unconventional materials into pavement construction.
- Reducing taxation regimes, with public spending moving away from infrastructure to education and health, thereby requiring more efficient use of money and resources.
- The completion of the principal road network in most developed countries and the consequent movement from construction to maintenance and improvement.
- An increasing recognition of the importance of road transport infrastructure in the economies of developing countries, yet where funding is very difficult.

In response to these issues, engineers have been driven back to a fundamental understanding of the way conventional crushed aggregates behave, how this compares to the behaviour of alternatives and how properties may be amended or best exploited to maximise their engineering capacity. In summary, one could say that the aim has been to get the necessary performance from different or poorer materials for less cost.

So there has been a lot of research work in the laboratory, along with practical work in-situ, to assess the properties of relevance and to investigate different formulations. Site trials have been made to demonstrate the feasibility of new construction methods or materials. In-situ testing has advanced considerably to allow quality control to assess properties directly related to the anticipated resources. Along with these “hard” developments “soft” engineering has also been moving forward—new specification approaches have been adopted to maximise the possibilities and to permit adequately performing, novel materials; new analytical approaches are being tried to permit better prediction of future performance.

It is for these reasons that this book sets out to report on recent advances and experiences. It aims to encompass granular bases and sub-bases together with alternatives to conventional granular materials in these applications including hydraulically bound and stabilised materials. Equally, their application in low volume and unsealed pavements and in the lower layers of bound pavements is addressed. This book includes 38 technical contributions from authors in every part of the world (once again, only

Antarctica is unrepresented!). The papers were presented at the “Pavements Unbound!” symposium (UNBAR 6) held at the University of Nottingham in England from 6th to 8th July 2004.

This book wouldn’t exist without the authors! So my sincere thanks go to all of them for their hard work in preparing and correcting their papers and (to most of them) for getting the papers to me on time. I owe a special debt to the referees who willingly assisted in assessing the papers and in suggesting many improvements, often to tight deadlines. Thanks are also due to the editorial team at Balkema, particularly Richard Gundel, for working around my inefficiencies and still producing an excellent publication. I hope that every reader will find this book a valuable resource on a subject that is becoming increasingly important.

Andrew Dawson

UNBAR 6 Convenor

Nottingham, April 2004

*Laboratory testing and
granular material behaviour*

Development of a simplified test for unbound aggregates and weak hydraulically bound materials utilising the NAT

J.P.Edwards & N.H.Thom

Scott Wilson Pavement Engineering

P.R.Flemming

Loughborough University

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ABSTRACT: The requirement for a performance based test for unbound and weak hydraulically bound materials (HBMs), which is also simple to use in comparison with research based apparatus, was identified some years ago as being key to characterising non standard and stabilised pavement foundation materials. Resilient modulus and permanent deformation resistance were identified as key material performance properties to be determined, both for input into new UK design procedures (where pavement thickness will depend on foundation class), and for assessing potential constructability prior to more expensive trials. Design features for such a test include the incorporation of an aggregate size up to 40 mm, the ability to cure HBM samples other than in the compaction mould or the test equipment itself, and utilisation of the standard Nottingham Asphalt Tester (NAT) loading frame, which is widely used for the testing of asphalt samples.

1 INTRODUCTION

In recent years, there has been a strong tendency throughout civil engineering to move away from traditional “recipe and method” specifications and towards those that are “performance related” (Fleming, et al 2000). The determination of fundamental engineering properties of materials is key to their inclusion within analytical or mechanistic pavement designs.

An overview of laboratory test method indicated a lack of recognised mechanical tests in the UK applicable to unbound and weakly bound pavement materials (Edwards, 2003). Specialist tests are available such as the Repeated Load Triaxial (RLT) test and Hollow Cylinder Apparatus (HCA), as are much simpler techniques such as the California Bearing Ratio (CBR).

A need was therefore identified for a relatively simple test which was capable of generating the required mechanical properties for input into analytical pavement design, most notably stiffness modulus, but also resistance to permanent deformation. The need for this test relates to conventional unbound materials (soils, capping, granular sub-

bases), but is perhaps more critical in the case of less well understood materials, in particular stabilised soils, hydraulically bound cappings or sub-bases, and cement bound materials. In some of these cases, there is a clear need to be able to obtain information on specimens at different stages of curing.

A new laboratory test for the characterisation of unbound and weak hydraulically bound mixtures under repeated loading was therefore developed at Scott Wilson Pavement Engineering Limited. The equipment is known as the “Springbox” and is loosely based around the principle of a variably confined test, similar to that adopted in the mechanically more complex South African K-Mould (Semmeling and De Beer, 1993). The Springbox has been designed to fill the gap between relatively complex research based laboratory tools and the more empirical CBR test, as a relatively simple and practical tool, but one which is capable of generating scientifically meaningful data.

The initial concept behind the Springbox was to utilise the NAT loading frame, instrumentation and software. The NAT was identified as a piece of equipment commonly available in UK materials testing laboratories, and widely used for the testing of asphalt samples.

Utilising the NAT loading frame and hardware meant the following constraints applied to the test design:

- The maximum load is 5 or 10 kN (dependent on type of NAT);
- The width of the apparatus is restricted to 250 mm;
- The length of the apparatus is restricted to 500 mm (assuming the temperature control cabinet is not removed).

This paper details the test equipment, sample preparation protocol (primarily compaction), trials and preliminary results for a range of materials tested, and then concludes by suggesting that the equipment described, whilst still a prototype, represents a real advance in material characterisation technology. Areas for further research prior to full implementation are highlighted.

2 DESCRIPTION OF THE SPRINGBOX

In order to generate meaningful data, it was decided that a degree of horizontal strain had to be permitted within a test specimen and the Springbox achieves this by the specimen taking the form of a cube, a pair of whose horizontal faces are spring-loaded; the other pair are fixed. The Springbox has been designed for use within a NAT loading frame. In order to accommodate as large a particle size as possible, the full 250 mm dimension (maximum width of the test apparatus within a standard NAT loading frame) is used, which restricts the dimension of the specimen to 170 mm. The form of test is therefore to apply pulsed vertical load to the full upper surface of the specimen, recording displacement both vertically and in the movable horizontal direction. Vertical load is controlled (three levels have been used) and, since spring stiffness is fixed, the load in the movable horizontal direction can be deduced from the measured horizontal strain. The spring-loaded horizontal faces have been designed to accommodate a range of spring sizes, with varying spring rates, allowing material specific selection.

The full test equipment comprises the following main elements:

- Removable sample liners;
- Sample liner compaction jacket;
- The Springbox test box;
- Loading platen;
- Adjustable spring plates;
- Linear variable differential transformer (LVDT) frame;
- Instrumentation;
- Interchangeable chrome alloy die cast springs;
- PC, NAT loading frame, software and hardware.

2.1 *The Springbox test box*

The Springbox test box has been designed to lock and constrain the removable liner along its fixed edges to prevent significant deflections during testing, to house the spring plates which not only give variable confinement to the specimen during testing but also to provide housing for horizontal measurement transducers.

The design of the Springbox has been optimised with regard to weight. The 6 mm plate thickness was considered a minimum to ensure acceptably low deformation of the box during a test.

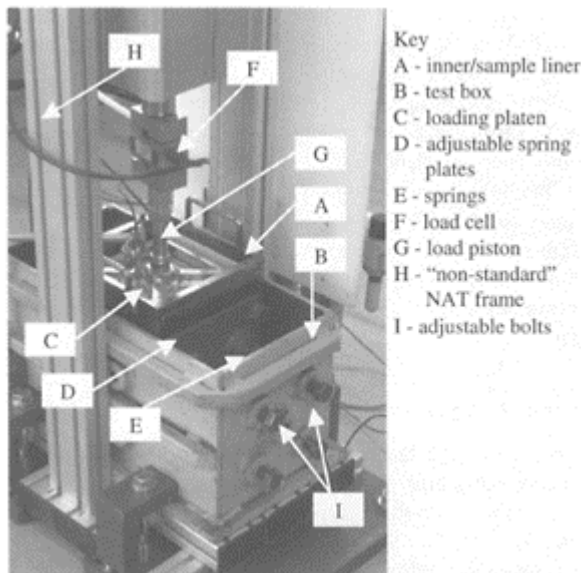


Figure 1. Photograph showing the Springbox apparatus.

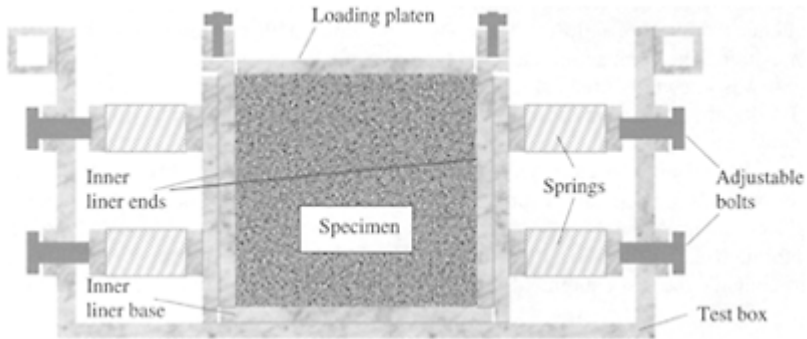


Figure 2. Representation of a longitudinal section through the Springbox apparatus.

The design requires stiffening ribs along each side of the box and at each end (Figures 1 and 2). These ribs give the added advantage that the box becomes easier to manhandle. Handles are also provided at each end to facilitate horizontal manoeuvring of the equipment. The total weight (not including the liner and specimen, which are inserted once the box is in position) is a little less than 20 kg.

2.2 Adjustable spring plates and die cast spring selection

The spring plates (shown in Figures 1 and 2), are adjustable up to the moveable inner liner sides. These plates run on low friction bearings along the base of the Springbox and house the die cast springs, which provide lateral confinement during testing.

The springs have been designed with regard to the amount of strain expected, which is desirable in a test. Since granular materials under simple stress conditions tend to reach peak stress at a strain of around 1–3%, it was considered sensible to allow movement of at least this level. With a specimen dimension of 170 mm, this equates to a movement of around 2 mm at each spring. The vertical load level to be applied to the specimen is variable, but is likely to be a maximum of 300 kPa. This is capable of generating an accumulated horizontal stress of up to 150 kPa under repeated load, equating to a little over 1 kN per spring (assuming four are used). Thus a spring stiffness of around 500 N/mm is appropriate for the test. The decision was taken to limit the number of springs to each side of the box to four. This was primarily due to the practicalities involved with using/spacing out a larger number of springs.

To ensure a consistent start-of-test condition with respect to the horizontally acting springs, the approach taken is to tighten them using a torque wrench only, until the first signs of resistance are encountered, the intention being for the start point to be zero horizontal stress (or as close as can be realistically obtained).

2.3 Inner sample liner and compaction jacket

The inner sample liners are constructed from stainless steel due to its relative cost, resistance to abrasion, resistance to corrosion, high stiffness and buildability issues associated with the liner's detachable sides. The specimen weight is typically around 10–12 kg and the total assemblage within the liner weighs 20 kg.

Additional support is given to the liner by the utilisation of a compaction jacket as can be seen in Figure 3. This compaction jacket was designed to be fully adjustable to individual liners (allowing for construction tolerances) and interfaces with the requirement of a mechanism, that would enable the two detachable walls of the liner to move freely during a test, but rigidly restrained while the liner was not in the test equipment.

The intended procedure is to assemble the inner liner, with jacking screws holding the end walls in position. The compaction jacket is then assembled around the inner liner and tightened. Although it was known that the compaction jacket, sample and liner have a combined weight in excess of 30 kg, the system was designed so that the liner never needed to be lifted while the compaction jacket was affixed.

The inner liner itself is constructed of 6 mm steel. The base and the two sidewalls are monolithic; the two ends are separate 6 mm sheets with a clearance of around 1 mm to the sidewalls. A strut of steel connects the two sidewalls at the top of the liner at each end. Two jacking screws are threaded through each strut onto a rib at the top of each end wall. These screws are tightened whilst the liner is outside the test equipment, generating friction between the end walls and the base of

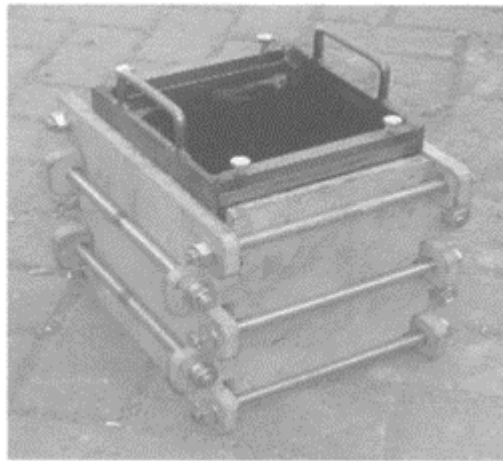


Figure 3. Sample inner liner and adjustable compaction jacket.

the liner sufficient for zero slip. An approximate calculation was carried out based on the likely “locked-in” horizontal stresses following compaction. The jacking screws have to

be capable of generating sufficient downward force on the liner ends, and therefore friction against the liner base, to withstand these locked-in stresses. Once the liner is in position in the test apparatus, these screws are released.

The choice of 6 mm steel plate was made after consideration of the forces involved. During the test, the fixed sides are supported against four adjustable bolts and the movable sides rest against the spring plates. An approximate computation suggests a maximum plate distortion under load of around 10 microns (for a 6 mm plate). This is considered acceptable in comparison to the anticipated displacements during a test.

To minimise friction between the specimen and the walls, base and loading platen combinations of lubricants and/or membranes were trialed with varying degrees of success during equipment development. After trials of several different options, 0.5 mm PTFE membrane was found to be the most suitable material to use between the steel of the liner and the specimen. This generated very low friction between the PTFE and the steel and avoided the possibility of embedment of stones into the PTFE, which occurred when a thicker membrane was used. The PTFE membrane was continued around the internal angles of the liner, in order to prevent particles from the specimen entering the joint between the fixed and free panels of the liner and potentially inhibiting movement.

2.4 Instrumentation, software and data acquisition system

The following measurements are taken during the test:

- Load magnitude (controlled);
- Vertical displacement (transient and permanent);
- Horizontal displacement (transient and permanent).

The first, load magnitude, can easily be achieved through the existing NAT load cell. The magnitude of strains which it is desirable to measure may be as little as 10 microstrain, equivalent to only 1.7 microns over 170 mm. For this, LVDTs are seen as the only realistic option. To give averaged data (and to remove any error due to plate tilt), two LVDTs are needed for vertical measurement and two more for horizontal measurement. The horizontal LVDTs measure to the centre of each movable liner wall/adjustable spring plate.

Cooper Research Technology undertook software developments. The intention was to use the existing NAT software as a basis for the new developmental trials. Test/specimen reference details factually recorded during the set up comprised: operator, file name and specimen dimensions. Test input data comprised:

- Test duration (to the nearest 100 pulses);
- A facility for undertaking conditioning pulses prior to the test, test load (between 0.1 and 10 kN, to the nearest 0.1 kN);
- Target load duration (between 100 and 1000 ms, to the nearest 20 ms);
- Duration of the whole load/unload cycle (between 100 and 3000 ms, to the nearest 100 ms).

The software was written to record permanent deformation data from the four LVDTs, at every tenth pulse. A base line reading was taken on exiting the LVDT set-up screen prior to starting the test.

LVDTs 1 and 2 extend during the test, while LVDTs 3 and 4 compress. At the development stage all the LVDT measurements were reported individually. The transient deflections of the sample were recorded by taking readings from the four LVDTs and the load cell every 5 ms over the length of a specified pulse, and also by recording the loading stage of the subsequent pulse. The only limitation on the number of these data sets that could be specified is the size of the resulting data file. The time between reading the first and last channels at each specified point is approximately 4 ms. The order in which the channels are read is: LVDT 1, LVDT 3, load cell, LVDT 4 and LVDT 2. This meant that the sum (or mean) of both the vertical and horizontal LVDTs is as coincidental with the load cell readings as possible.

The in-test monitoring screen updates every 10 pulses and displays the 4 individual LVDT readouts, the approximate shape of the loading pulse, and the mean of the two LVDTs measuring the accumulation of permanent deformation which is also graphically shown against a number of load applications (pulses).

The resolution of the LVDTs was reviewed and is still subject to re-assessment. At low stress levels the 5mm sweep LVDTs and 16-bit processor produce rather ill defined hysteresis loops (especially the LVDTs measuring horizontal deflections). This is a problem also encountered with RLT testing where the measuring device needs to be able to measure relatively large permanent strains, while also requiring the resolution to accurately record smaller transient strains. An initial assessment indicated that this was not a significant problem during the trials, as it was only noticeable in very stiff materials at the lower stress applications.

The Springbox software allows input of up to three consecutive load levels for a user-specified number of pulses. As with standard pneumatic NATs, the nature of the NAT hardware means that the target load is generally not instantaneously obtained. Dependant on the target test, the NAT equipment typically took up to a maximum of 5 pulses before the specified load was obtained. Given the simplified nature of the test, this was not considered overly significant.

3 SAMPLE PREPARATION TRIALS AND PROCEDURES

The range of unbound and lightly bound materials for which testing is ideally required is large, which gives rise to issues regarding the preparation of a range of realistic specimens. The main areas that were considered are: grading and maximum aggregate size, compaction, mixing (aggregate/soil binder affinity), curing and soaking. The materials chosen for testing included: Oxford Clay, Lime stabilised Oxford Clay, Type 1 unbound sub-base, Slag Bound Material (SBM) and Cement Bound Material (CBM).

3.1 Grading and maximum aggregate size

Various ratios of test specimen size to maximum aggregate size were identified during the literature review. There is clearly no unanimous view on the permissible limit of specimen size to particle size ratios; different researchers suggest numbers from 4 to 10 (Edwards, 2003). However, it seems clear that the chief complicating factor is the full particle size distribution in that the importance of the largest particle is greatly reduced

when surrounded by a mass of significantly smaller particles. A maximum aggregate size of 40 mm (in broadly graded aggregates) was selected during the equipment trials.

3.2 Compaction

Extensive work on the compaction procedure for Springbox sample preparation resulted in the recommendation of a method similar to that used for CBR, but with due account being taken of the difference in specimen area. The BS EN 13286–4 (2003) methodology for compacting samples with a vibrating hammer was used as a starting point for the sample compaction procedure. The vibrating hammer has the added advantage of being portable (when used with a small generator) and applicable to a wide range of materials. Ideally, alternative methodologies more suitable to compaction of materials prone to degradation under the relatively high compaction stress generated with a vibrating hammer should be considered. However, this is as an area for further possible research, using apparatus such as the vibrating table or gyratory compactor.

Building the sample up in four layers, using a full surface compaction foot and applying the compactive force for between 90 and 100 seconds, produced suitable samples without undue sample degradation. In the case of material that benefits from a kneading action of compaction (such as cohesive soils), a smaller scale compaction foot capable of shearing the soil during the sample preparation was utilised.

3.3 Curing

Samples of lime stabilised Oxford Clay, SBM and CBM were all cured. The length of the curing periods meant that only two curing methodologies were trialed during the test development phase, namely:

- Air curing with room temperatures recorded;
- Sealed curing at recorded room temperatures.

Curing periods were varied depending on the expected rate of strength gain. For example cured samples of SBM were tested at 40 and 80 days age, while the cured CBM was tested at 7 days age only.

4 EXPERIMENTAL RESULTS AND ANALYSIS

Bearing in mind the likely use of the test and the non-linearity of the materials that would normally be tested, it was decided that a procedure had to be developed which applied a range of different stress levels, and so measured stiffness and permanent deformation resistance applicable to different levels in a pavement. The procedure adopted for these tests was as follows:

- Apply 1000 load applications at a low load level (1.5 kN, equivalent to 52 kPa);
- Apply the same number at an intermediate stress level (3 kN, 104 kPa);
- Repeat at a high stress level (5 kN, 173 kPa).

Results are presented for a range of materials tested which include:

- Cohesive soil (Oxford Clay);
- Lime stabilised Oxford Clay (5% quicklime);
- Type 1 sub-base (carboniferous limestone from Longcliffe quarry);
- SBM (crushed limestone+15% blast furnace slag+10% steel slag);
- CBM material (capping ex M6 Toll+4% OPC by dry weight).

Figure 4 shows a range of permanent deformation measurements for the suite of materials tested in the Springbox. It is not considered that the permanent strain information will be input directly into design, since computation of permanent deformation in a pavement is not currently carried out.

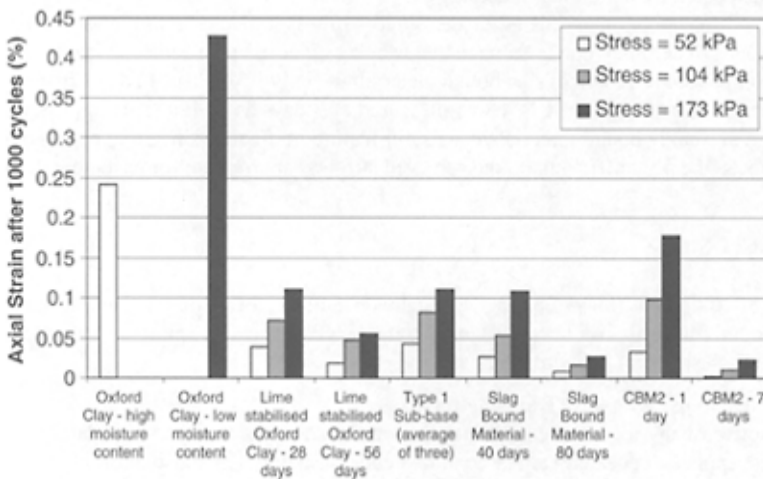


Figure 4. Permanent axial strain data.

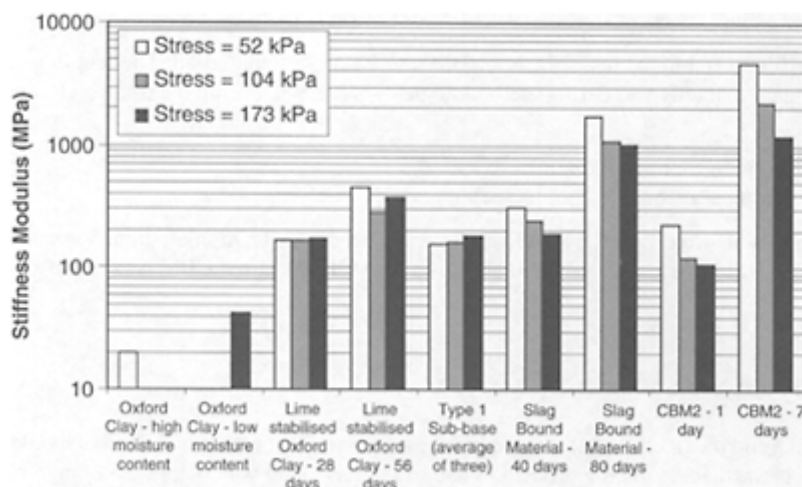


Figure 5. Stiffness modulus data (taken from 1000th load cycle).

However, it is very important to ensure that excessive deformation will not occur due to trafficking during construction. It is clear that the test is able to discriminate between materials regarding their deformation susceptibility, and the data can be used to assess whether a particular material would meet the performance requirements identified or it needed to be untrafficked (or cured prior to trafficking).

From Figure 5, it is immediately clear that the stiffnesses measured are in the correct range for such materials for use in pavement design. For example, the stiffness of Type 1 sub-base is generally taken to be 150 MPa for design purposes; the test results range from 157 to 185 MPa. Stabilised soil is normally expected to achieve a slightly better stiffness; the lime stabilised clay results range from 165 to 173 MPa at 28 days, rising to 285 to 448 MPa at 56 days. The following points should be noted:

- The trend for unbound materials is for the measured stiffness to increase slightly at higher levels of stress. This is a function of the increase in confinement generated as the test proceeds and is expected for a granular material.
- The trend for bound materials is for measured stiffness to decrease at higher levels of stress. This is almost certainly due to damage that is being induced under repeated loading. The measured compressive strength of slag bound material was only 1.5 MPa at 40 days, rising to 11 MPa at 80 days. The strength of CBM was measured at 7 days as 6 MPa. This tendency to induce damage in weakly bound materials is a useful feature of the test, since it is possible to include the effects of early trafficking on the achieved stiffness appropriate for pavement design.

5 CONCLUSIONS

A piece of equipment (known as the ‘Springbox’) suitable for testing a range of unbound and lightly bound materials has been designed, constructed, refined and initially validated. In summary, the key aspects of the equipment are:

- It is suited to use with a NAT loading frame.
- NAT software has been written to control the test and acquire the necessary data.
- The test applies a repeated vertical load to a cube of material and also allows horizontal strain in the specimen in one direction, with sides restrained by springs. In the other horizontal direction the sides are fully restrained.
- Both stiffness modulus and a measure of resistance to permanent strain are obtained.
- Techniques have been designed for compaction and curing/conditioning, using a stainless steel liner to directly enclose the specimen, which is then inserted into the Springbox when ready for testing.
- Data obtained on a wide range of unbound and lightly bound materials gives confidence that results from the Springbox test are suitable for use in material characterisation for pavement design.

ACKNOWLEDGEMENTS

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REFERENCES

- BS EN 13286–4, 2003, “*Unbound and hydraulically bound mixtures. Test methods for laboratory reference density and water content, Vibrating hammer*”, BSI.
- Edwards, J.P., 2003, “*Characterisation of Unbound and Bound Standard/Alternative Materials within Pavement Foundations*”, EngD Literature Review, Loughborough University.
- Fleming, P.R., Rogers, C.D.F., Thom, N.H., Armitage, R.J. and Frost, M.W., 2000, “*Performance Based Specification for Road Foundation Materials*” *Institute of Quarrying Millennium Conference*, Bristol.
- Semmelink, C.J and De Beer, M., 1993, “*Development of a dynamic DRRT k-mould system*”, *Proceedings of the Annual Transportation Convention*, University of Pretoria.

Assessment of the effect of seasonal variations on the unbound materials of low volume roads by laboratory testing

P.Kolisoja & N.Vuorimies

Tampere University of Technology, Tampere, Finland
T.Saarenketo

Roadscanners Ltd, Rovaniemi, Finland

Pavements Unbound—Dawson (ed.)

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ABSTRACT: The paper presents the key results of a series of large scale repeated load triaxial tests performed using a test procedure that intended to simulate the effect of seasonal variations. The analysis is made with a special view to the effects of stress level, test specimen condition and type of aggregate on the permanent deformation behavior. Based on the analysis a tentative framework for assessing the need for springtime weight restrictions on a low volume road with a known structure and traffic volume is suggested.

1 INTRODUCTION

The effect of seasonal variations plays an especially important role on the performance and mechanical behavior of low volume roads in the North European countries. The main reason for most of the bearing capacity problems in these areas is related to the effects of seasonal frost on which the road infrastructure is yearly exposed to. The particularly critical situation takes place during the thawing period in early spring when the ice in the road structures begins to melt from the top while the lower parts of the pavement and the underlying structure are still frozen. In the worst case this can lead to full saturation of the upper unbound pavement layers and consequent development of excess pore water pressure under traffic loading, a respective reduction in the effective stresses and ultimately to rapid accumulation of permanent deformation. Close to similar conditions may also appear in areas of milder climate if the road structures are inadequately drained while they are exposed to heavy rains.

Until recent times the amount of laboratory studies concentrating on the mechanical behavior of unbound road pavement materials exposed to the effect of seasonal variations has been fairly limited, especially as far as the permanent deformation behavior of

unbound materials is concerned. The main reason for this is most likely the extremely laborious and time consuming nature of all mechanical testing procedures including e.g. the effect of freeze-thaw cycles. This paper attempts to make a small contribution to the existing amount of knowledge in this area by making use of the results from a series of repeated load triaxial tests including simulation of the effect of seasonal variations performed at the Tampere University of Technology, Finland. The primary aim of these test series was to investigate the resilient deformation behavior of different types of unbound base and sub-base course materials. However, as most of the test specimens were also experiencing a reasonable amount of permanent deformation especially during the resilient deformation testing that was performed after a freeze-thaw cycle, it was also decided to analyze the test results with a view to the permanent deformation behavior. In spite of the relatively low number of load repetitions involved, the test results were considered potentially useful especially concerning low volume roads where the number of heavy vehicles during the critical thawing period can also be assumed to be correspondingly low.

2 THEORETICAL FRAMEWORK OF THE ANALYSIS

The idea of connecting the accumulation rate of permanent deformations taking place under repeated loading to the strength of the material under monotonic loading has been considered e.g. by Brown and Selig (1991). According to them, the rate of permanent deformation development remains low provided that the cyclic peak value of stress ratio between the deviator stress q and hydrostatic stress p remains below 70% of the respective value at static failure (Figure 1). Later on, the idea of the existence of a sort of critical stress level (also called the “shake-down limit”) has been developed further e.g. by Lekarp (1999) and Werkmeister (2003).

The idea of the existence of a critical stress ratio visualized in Figure 1 makes it easy to understand, at least qualitatively, the accelerating effects of both lowering the material density and increasing of the water content on the accumulation rate of permanent deformation. Both of these phenomena result in lowering of the static failure load of the material in question and thus, in higher relative intensity of the repeated load and correspondingly faster development of permanent deformation. Furthermore, the very detrimental effect of excess pore water pressure development in a nearly or totally saturated unbound granular material can also be understood very easily in this framework. Because the deviator stress component of the stress path corresponding to the applied external load—the axial load pulse in a normal constant confining pressure triaxial test arrangement or the wheel load in the case of an actual road structure—is not affected by the excess pore water pressure development while the mean effective stress is, the consequence is that the stress path turns counter clockwise towards the failure line (Figure 2). In terms of the cyclic peak stress ratio q/p realised in relation to the static failure condition of the material at the same hydrostatic stress level this means of course a drastically more unfavourable condition.

Referring to the above discussion it should be possible to present schematically the accumulation rate of permanent deformation as a function of the cyclic peak stress ratio q/p in the form of Figure 3. As long as the stress ratio is not exceeding the critical limit,

say 70% of the value at static failure, the permanent deformation remains low. Meantime, as the peak cyclic stress ratio approaches failure condition under monotonic loading, the permanent deformation rate should approach infinity i.e. the material must be assumed to fail under one load cycle provided that the effects related to the loading rate are insignificant.

To enable utilisation of the idea presented in Figure 3 one should be able to quantify the rates of permanent deformation corresponding to different loading intensities with a relatively simple model, preferably possessing only one material parameter. In this study the simple model meeting

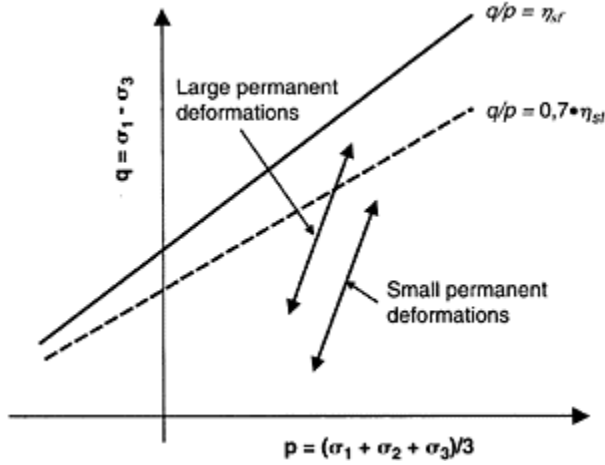


Figure 1. Schematic effect of loading intensity on the accumulation rate of permanent deformation (Kolisoja 1998).

this requirement has been derived from the classical model suggested by Sweere (1990) in the form of Equation 1:

$$\varepsilon_p = a \cdot N^b \quad (1)$$

where ε_p is the accumulated permanent axial strain of a triaxial test specimen; N is the number of load repetitions; and a and b are material parameters.

Bearing in mind the need for a very simple modeling approach it was observed that most of the test results could be described with reasonable accuracy if the accumulated permanent axial strain of the triaxial test specimen was expressed in microstrain units and the parameter a of Equation 1 was given a constant value of 100. Consequently the measurement results were approximated according to Equation 2:

$$\varepsilon_p = 100 \cdot N^b \quad (2)$$

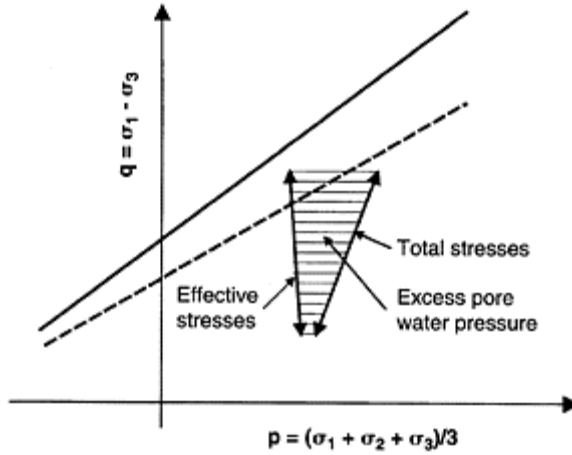


Figure 2. Schematic effect of excess pore water pressure on the accumulation rate of permanent deformation (Kolisoja 1998).

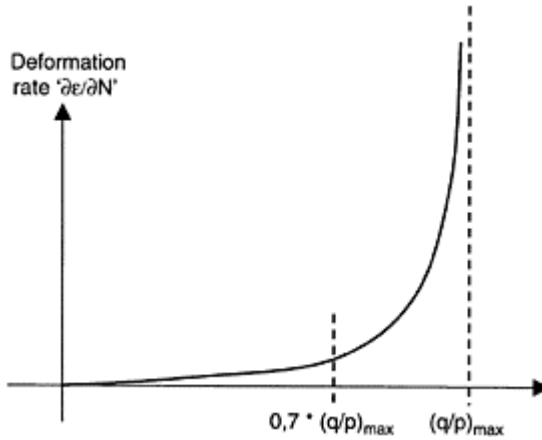


Figure 3. Schematic relation between the cyclic peak stress ratio q/p and the accumulation rate of permanent deformation.

In some cases this robust approximation was somewhat overestimating the development of permanent axial strain under a large number of load repetitions if the permanent deformation was stabilising while rapid and continuous development of permanent strain was in general described reasonably well (Figure 9).

3 TEST SERIES TO BE ANALYSED

The available set of test results included a total number of more than 20 large scale cyclic load triaxial tests of test specimen diameter 200 mm performed with various types of base and sub-base course aggregates consisting of natural gravel, crushed gravel and crushed rock. The tested aggregates represented a range of mineralogical compositions as well as fines contents. All of the test specimens were exposed to a test procedure intended to simulate the effect of seasonal variations as follows:

- The test material was prepared in predetermined grading and fines content and compacted at a water content close to optimum.
- The test specimen was dried in an oven at a temperature of about +45°C for about two weeks (=dry summertime condition).
- Resilient deformation properties of the test specimen were determined.
- The specimen was allowed to adsorb water through the bottom of the specimen for at least one week (=moist autumn time condition).
- Resilient deformation properties of the test specimen were determined again.
- The specimen was exposed to a freeze-thaw cycle while the base of the specimen was connected to a water reservoir (=wet springtime condition).
- Resilient deformation properties of the test specimen were determined for the third time.
- The test specimen was exposed to a permanent deformation test consisting of about 100 000 load repetitions.
- If permanent deformation in the preceding test stages was not significant the specimen was exposed to a monotonic loading triaxial test.

For determination of the resilient deformation properties, the American SHRP P46 test procedure (AASHTO 1992) was applied with the exception that preconditioning was not done for the determination that was made after the freeze-thaw cycle. The test procedure consisted of 15 different combinations of constant confining pressure and repeated axial load (Figure 4). According to the test procedure each of the 15 stress paths is cycled 100 times.

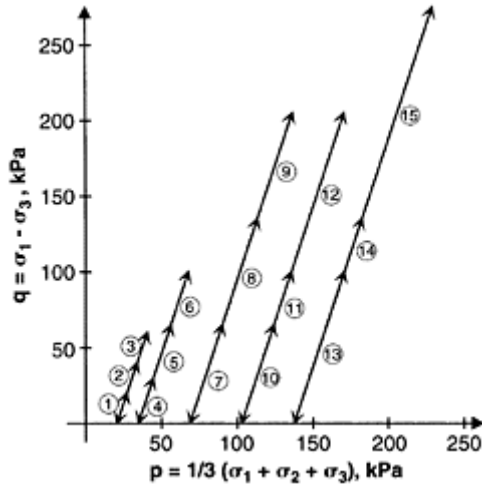


Figure 4. Stress paths included in the SHRP P46 loading procedure (AASHTO 1992).

4 ANALYSIS OF THE TEST RESULTS

4.1 Modeling approach taken

As already mentioned the primary aim of the test series now being utilised was to investigate the effect of seasonal variation on the resilient deformation behaviour of unbound base and sub-base course aggregates. Regarding this aim the test results have already been presented previously (Kolisoja et al 2002, Saarenketo et al 2002) and are therefore not repeated here.

Concerning the analysis of the permanent deformation behavior of the test materials, a curve fitting according to Equation 2 was made at first for the measured values of accumulated permanent axial strain at all of the applied stress paths. The values of parameter b thus obtained and now considered as simple measures of the rate of permanent deformation were then plotted as a function of the applied cyclic peak stress ratio q/p following the idea sketched in Figure 3. As an example of the typical relation between these quantities, Figure 5 presents the results consequently obtained for a crushed intermediary vulcanite from Lepoo in Western Finland having a fines content of 10.7%. The presented results are now derived based on the resilient deformation test performed after a freeze-thaw cycle.

Even though a reasonable amount of scatter exists between the data points and the fitting curve shown in Figure 5, the result can still be considered to support the idea presented in Section 2 fairly well. At this point it must, however, be noted that the parameter b values corresponding to the stress paths Numbers 1 and 2 of Figure 4 have been omitted from the analysis since they were much higher than the general trend. The

reason for this exceptional behavior during the first two stress paths is assumed to be that while the repeated loading was applied along these stress paths a marked amount of extra loosening of the test specimen that had taken place during the freeze-thaw cycle was recovered. Furthermore, it must be noted that at stress paths where the accumulation rate of permanent strain was very low, the limited accuracy of measurement instrumentation was also producing some additional scatter in the values of parameter b .

One more very important observation concerning the way of presenting the results shown in Figure 5 is that parameter b appears as the exponent in Equation 2 and is therefore not a linear measure of the accumulation rate of permanent deformation. Thus, the accumulation rate of permanent deformation per load cycle is in fact increasing much faster as a function of the applied cyclic peak stress ratio q/p than is shown in Figure 5.

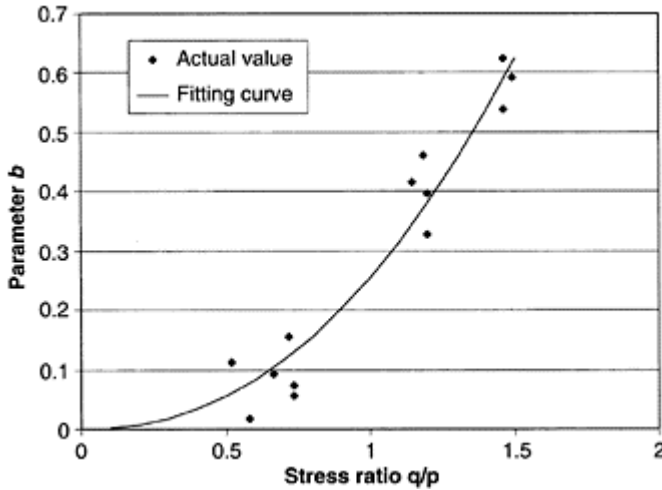


Figure 5. Values of parameter b as a function of the applied cyclic peak stress ratio q/p for a fines rich crushed rock material tested after a freeze-thaw cycle.

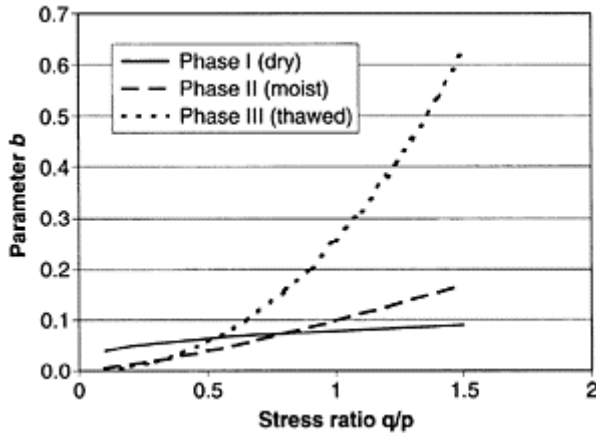


Figure 6. Effect of material condition on the accumulation rate of permanent deformation described in terms of the value of parameter b (Equation 2).

4.2 Effect of moisture content

As a typical example of the effect of material condition on the accumulation rate of permanent deformation Figure 6 presents the respective fitting curves for parameter b at three different conditions of the tested Lepoo crushed rock aggregate i.e. after the drying of the specimen, after it has been allowed to suck in water and after it has been exposed to a freeze-thaw cycle (see Section 3).

Quite expectedly the physical condition of the material, in this case varying in terms of moisture content from 1.5% trough 4.0% and 8.0%, is clearly reflected in the material's ability to resist the development of permanent deformation. Taking into account the limited measurement accuracy, the stress paths of the SHRP loading procedure are practically too mild, as they in fact should be, to produce much useful information concerning the permanent deformation behavior if the test specimen is in a dry or moist condition. However, for a test specimen that has experienced a freeze thaw cycle the resilient deformation test, in spite of the low number of load repetitions included, seems also to give a useful indication concerning the critical stress ratio that should not be exceeded in the respective structural layer of the road pavement when the material is in the weakest condition during the spring-thaw period.

4.3 Effect of fines content

An example of the effect of fines content on the permanent deformation behavior of the same Lepoo crushed rock aggregate in the thawing condition is given in Figure 7. The figure indicates the corresponding fitting curves for parameter b at fines contents of 3.6%, 5.1% and 10.7%, respectively. As can be seen from the results, with this known-to-

be problematic aggregate, even a fines content as low as 5.1% is enough to make it susceptible to the loss of bearing capacity during the critical thawing period.

4.4 Effect of aggregate type

The effect of aggregate type on the susceptibility to permanent deformation during the critical thawing period is visualized in Figure 8, in which the fitting curves for parameter b obtained with five different aggregates are presented. All the aggregates were tested at potentially too high (see Figure 7) fines contents ranging from 8.3% to 10.7%. As can be seen from Figure 8 the differences between the aggregates are obvious and clearly the best performance is that of the Tohmovaara crushed granite known from experience to be a very well performing aggregate in actual road structures.

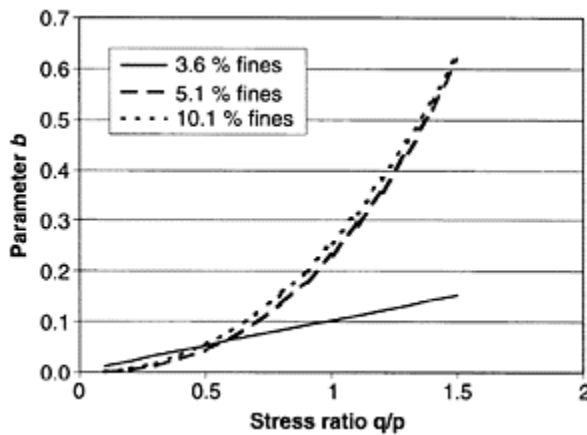


Figure 7. Effect of fines content on the accumulation rate of permanent deformation in the thawing Lepoo crushed rock aggregate described in terms of the value of parameter b .

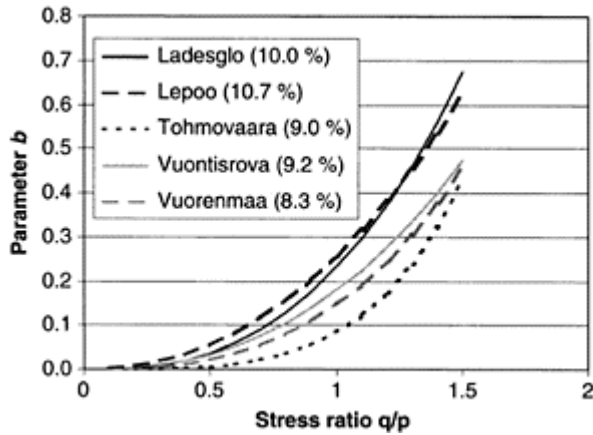


Figure 8. The effect of aggregate type on the accumulation rate of permanent deformation in thawing aggregates described in terms of the value of parameter b .

4.5 Validity of the approach taken at high number of load repetitions

Because the number of load repetitions at each of the stress paths included in the SHRP loading procedure is limited to one hundred, a very important question is how valid are the results above regarding exposure to longer lasting repeated loading. As far as the critical thawing condition is concerned the problem can be assessed reasonably well based on the available set of test results with one type of aggregate, crushed mica gneiss from Vuorenmaa. For this aggregate, three test specimens have, after the resilient deformation tests, been exposed to a continuous repeated axial loading at different intensities under a constant confining pressure of 50 kPa (Figure 9). The respective values of parameter b are indicated in Figure 10 by the black markers, together with the fitting curves determined based on the permanent axial strains measured during the preceding SHRP loading series.

According to Figure 10 the values of parameter b determined based on the SHRP test series tend to somewhat exaggerate the accumulation rate of permanent deformation, but still the trend as a function of stress ratio q/p is the same in both cases. Concerning the other test materials, the difference between the values of parameter b determined using the SHRP test results and the longer lasting

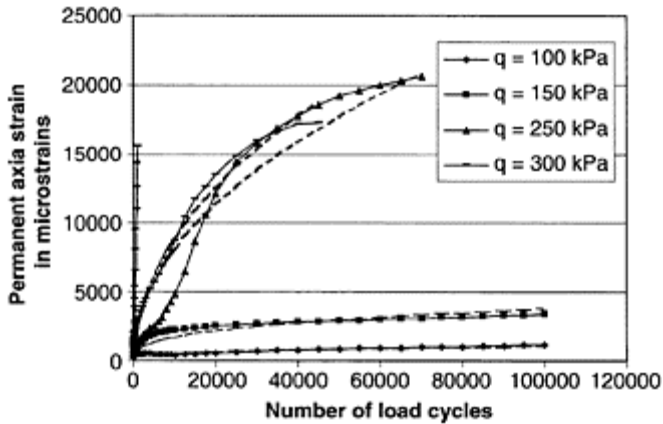


Figure 9. Accumulation of permanent deformation in the fines rich Vuorenmaa aggregate at various intensities of cyclic deviator stress q . Respective curve fittings are indicated by the dotted lines.

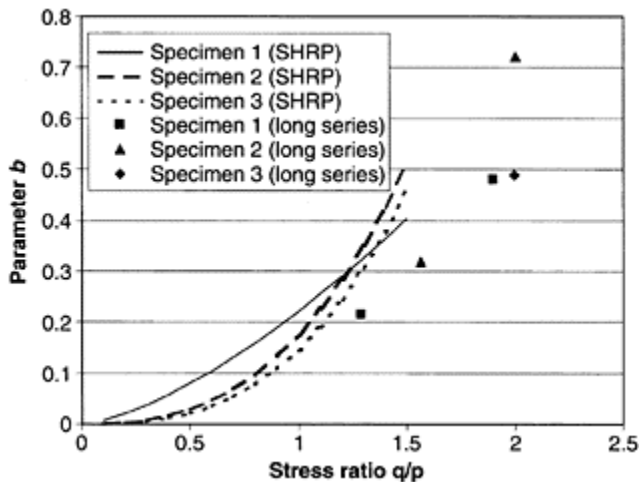


Figure 10. Comparison of the values of parameter b determined from the SHRP test results and those of the longer lasting repeated load series of Figure 9.

repeated loading test series (normally performed using a cyclic deviator stress 300 kPa and a constant confining pressure 50 kPa) was in general of the same order as that indicated in Figure 10.

5 APPLICATION OF THE RESULTS

Figure 11 illustrates the sensitivity of permanent deformation predictions related to different values of parameter b in Equation 2. As can be seen from the figure, and has already been pointed out above, the accumulation rate of permanent deformation is not in a linear relation to the value of b , but the higher the value the faster the permanent deformation develops.

The principal idea of utilizing the type of results presented above concerning the springtime bearing capacity problems on a low volume road could be as follows. Let us assume that the maximum amount of permanent strain allowed to be developed during one thawing period can be defined and the volume of heavy traffic during the critical thawing period can be estimated, then

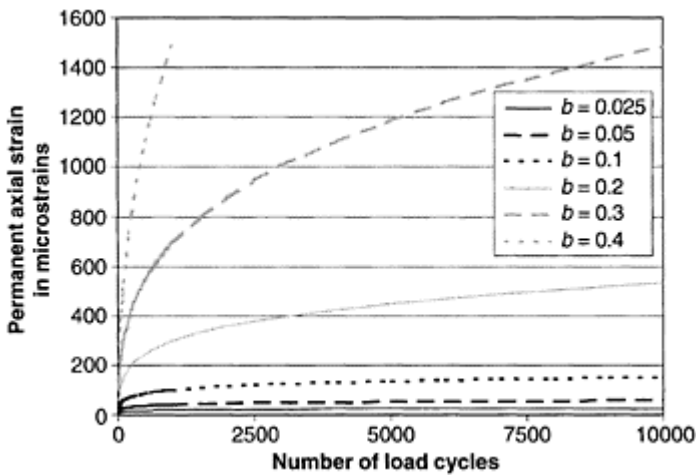


Figure 11. Accumulation rate of permanent deformation corresponding to some values of parameter b in Equation 2.

the respective “allowable” value of parameter b can be read from a diagram like that of Figure 11 or alternatively it can be solved directly from Equation 2. If one then knows the relationship between the value of parameter b and the cyclic peak stress ratio q/p for e.g. the unbound base course material in question, like the one shown in Figure 5, the “allowable” stress ratio q/p corresponding to the “allowable” value b can be determined. By making an analysis on the distribution of stresses in the pavement structure at hand,

one can then decide whether the situation on that road is acceptable or not. In the case of a negative answer the only immediate action that can be taken to protect the road structure is to apply a temporary weight restriction. In that case, based on the same chain of conclusions, it should in principle be possible to derive even the magnitude of the weight restriction to be applied.

At the present level of knowledge the above idea contains of course a number of uncertainties including at least: 1) the currently inadequate information especially concerning the combined effect of aggregate type and fines content on the relation between parameter b and the stress ratio q/p (Figures 7 and 8); 2) lack of correct input parameters and simplifications that must be made in connection with the mechanical modeling of the pavement structure, and last but not least; 3) the large variability in physical conditions, construction materials and layer thicknesses along the road to be analyzed. However, it is believed that the framework of analysis briefly described above could provide a physically justified way of making an assessment of the expected service life of a low volume road exposed to the effect of seasonal frost and of the need for weight restrictions during the springtime thawing period. Regarding especially the later mentioned aspect, the work has been going on (Schneider 2003) and is to be continued in connection with the European Union financed Roadex II project concentrating on the problems of the low volume road network in the Northern periphery areas.

6 CONCLUSIONS

The main conclusions concerning the current paper can be summarized as follows:

- The analysis of a set of repeated load triaxial tests simulating the effect of seasonal variations reveals that, even in connection with a resilient deformation test performed with aggregates in a thawing condition, useful information concerning the permanent deformation behavior of the test material is produced.
- Accumulation rate of permanent deformation is clearly related to the cyclic peak stress ratio q/p . However, the relationship depends very much on the fines content and the type of aggregate.
- In comparison to longer lasting repeated load triaxial tests the prediction of the accumulation rate of permanent deformations made based on the results of the SHRP P46 loading procedure is somewhat overestimated.
- A tentative framework for assessing the need for springtime weight restrictions on a low volume road with a known structure and traffic volume is suggested.

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REFERENCES

- AASHTO T 294–92 I. 1992. *Interim method of test for resilient modulus of unbound granular base/sub-base materials and subgrade soils—SHRP protocol P46*. American Association of State Highway and Transportation Officials.
- Brown, S.F. & Selig, E.T. 1991. The design of pavement and rail track foundations. In: O'Reilly, M.P. & Brown, S.F. (Eds) *Cyclic loading of soils: from theory to design*. Blackie and Son Ltd, London.
- Kolisoja, P. 1998. *Large scale dynamic triaxial tests for Arbeidsfellesskapet KPG, Results of the permanent deformation tests*. Delprosjektrapport KPG 20, Tampere.
- Kolisoja, P., Saarenketo, T., Peltoniemi, H. & Vuorimies, N. 2002. Laboratory testing of suction and deformation properties of base course aggregates. *Transportation Research Record*, Vol. 1787, pp. 83–89.
- Lekarp, F. 1999. *Resilient and permanent deformation behaviour of unbound aggregates under repeated loading*. PhD Thesis, Royal Institute of Technology, Stockholm.
- Saarenketo, T., Kolisoja P., Vuorimies, N. & Peltoniemi, H. 2002. Effect of seasonal changes on the strength and deformation properties of unbound and bound road aggregates. *Proceedings of the 6th International Conference on the Bearing Capacity of Roads and Airfields*, Lisbon. Vol. 2, pp. 1059–1069.
- Schneider, J. 2003. *Permanent deformation behaviour of low volume roads in Nordic countries*. M.Sc. Thesis, Tampere University of Technology, Tampere.
- Sweere, G.T.H. 1990. *Unbound granular bases for roads*. PhD Thesis, University of Delft.
- Werkmeister, S. 2003. *Permanent deformation behaviour of unbound granular materials in pavement constructions*. PhD Thesis, Dresden University of Technology, Dresden.

Shear strength and permanent deformation of unbound aggregates used in brazilian pavements

W.P.Núñez, R.Malysz, J.A.Ceratti' & W.Y.Y.Gehling

Federal University of Rio Grande do Sul, Brazil

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ABSTRACT: Most of studies on unbound aggregates carried out in Brazil have discussed elastic behaviour and little attention has been paid to shear strength and permanent deformation. This paper analyses the results of a laboratory study on the strength and permanent deformation characteristics of unbound aggregates used as bases or sub-bases in Brazil. Three gradations (one well-graded and two open-graded) were chosen, in order to quantify the effects of fine aggregates (percent passing in a #4 sieve), stress state on shear strength and permanent deformation. Failure envelopes and shear strength parameters (c' & ϕ') were obtained in triaxial tests and permanent deformation evolution was measured under dynamic loading. Models relating permanent deformation to number of loading cycles and stress state are proposed and the permanent deformation evolutions of the three gradations are compared. Global results show that the well-graded aggregates presented higher strength and accumulated lower deformation than the open ones.

1 INTRODUCTION

In flexible pavements, especially when unsurfaced or thinly surfaced, granular layers play an important role in the overall performance of the structure. In order to establish more rational pavement design and construction criteria it is essential that the response of granular layers under traffic loading be thoroughly understood and taken into consideration. Most of studies on unbound aggregates carried out in Brazil have discussed elastic behaviour and little attention has been paid to shear strength permanent deformation. However, in thin pavements, rutting, due to volumetric compression and/or shear of granular layers, is frequently the failure mode.

In this context, this paper presents and analyses the results of a laboratory study on unbound aggregates used as bases or sub-bases in southern Brazil. Three gradations (one well-graded and two open-graded) were chosen, in order to quantify the effects of fine aggregates (percent passing in a #4 sieve) and stress state (σ_1 , σ_3 and the ratio $\sigma_d/\sigma_{1,f}$) on shear strength and permanent deformation.

The study was carried out with the purpose of determining:

- Mohr–Coulomb shear strength parameters, and
- initial permanent strain (ϵ_{pi}) and constant strain rate (CSR) for several levels of deviator stress.

Failure envelopes and shear strength parameters (c' & ϕ') were obtained with static triaxial tests and permanent deformation evolution under triaxial dynamic loading was measured.

2 TESTS METHODS

2.1 Shear strength tests

Soils and aggregates strength behaviour may be characterized by Mohr-Coulomb parameters: the cohesive interception (c') and the angle of internal friction (ϕ'). At any level of effective confining stress (σ_3), the failure vertical stress ($\sigma_{1,f}$) is given by

$$\sigma_{1,f} = \frac{(1 + \sin\phi')\sigma_3 + 2c' \cos\phi'}{(1 - \sin\phi')} \quad (1)$$

Several authors like: Lekarp *et al.* (1996), Garg & Thompson (1997), van Niekerk *et al.* (2000) and Theyse (2000), analysed the effects of grain size distribution, aggregate type, moisture content, compaction degree and stress state on the shear strength of unbound aggregates. Table 1 presents Mohr-Coulomb parameters obtained in shear strength tests on compacted specimens (generally 150×300 mm, but 300×600 mm in van Niekerk *et al.*'s study), with confining stresses ranging from 12 to 207 kPa.

In this study consolidated-drained (CD) tests were carried out on cylindrical specimens, of 100-mm diameter and 200-mm height, at a constant strain rate of 0.063%/s. Shear strength envelopes were determined using strain-stress curves. Since pavements are structures subjected to continuous degradation processes, with deformability prevailing over failure, it may be convenient to define shear strength envelopes related to strain levels below failure. Therefore, using stress-strain curves, mobilised shear strength envelopes and parameters corresponding to strains levels of 0.5; 1.0; 1.5 and 2.0% were determined.

2.2 Permanent deformation tests

Permanent deformation behaviour of unbound aggregates may be related to the ratio of the cyclic deviator stress applied (σ_d) to the failure vertical stress in triaxial test ($\sigma_{1,f}$). Some authors used the results of shear strength static tests to define the level of stresses applied in permanent deformation tests. While confining stresses ranged from 12 kPa to 280 kPa, the applied deviator stresses were either a percent of static failure stress ($\sigma_{1,f}$) (Lekarp *et al.*, 1996, Garg & Thompson, 1997, van Niekerk *et al.*, 2000 and Theyse, 2000) or a multiple of the applied confining stress (Werkmeister *et al.*, 2000).

The permanent deformation tests reported in this article were carried out in the same triaxial chamber used in shear strength tests. However, the repeated loadings were applied by a pneumatic system.

In order to define the confining stress (σ_3) for permanent deformation tests, a parametric analysis of some typical Brazilian flexible pavements was carried out. Using ELSYM5, the horizontal compressive stresses acting in the mid-depths of granular bases were estimated. A value of 21 kPa resulted as representative of confining stress. The levels of deviator stress were defined as percentages of a material static failure stress ($\sigma_{1,f}$).

Repeated loadings were applied in multiple stages (multi-stage tests). After every 80,000 cycles, deviator stress was increased in order to quantify its effect on permanent strains. Following

Table 1. Unbound aggregates shear strength parameters.

Author	$\phi'(^{\circ})$	$c'(\text{kPa})$
Lekarp <i>et al.</i> (1996)	58–67	49–145
Garg & Thompson (1997)	31–51	48–124
van Niekerk <i>et al.</i> (2000)	37–44	4–142
Theyse (2000)	48–55	26–121

van Niekerk *et al.* (2000), throughout the test axial strains were recorded at designated load cycles (N=100; 200; ...; 1,000; 2,000; ...; 10,000; 20,000; ...; 80,000).

When testing specimens of open-graded aggregates, deviator stresses corresponding to 20; 40; 60; 80 and 100% of static failure stresses ($\sigma_{1,f}$) were applied. Due to the characteristics of the pneumatic system and the triaxial chamber, when testing specimens of the well-graded aggregate the ratio $\sigma_d/(\sigma_{1,f})$ was limited to 50%. Even with this restriction, the applied cyclic loadings are representative of stresses acting in pavements bases and sub-bases, as shown in a parametric study carried out by Malysz (2004).

3 TESTED MATERIALS

3.1 Characterization

The aggregates were obtained by crushing of basalt rock and characterised by Casagrande (2003), as seen in Table 2.

3.2 Grain size distribution

In a previous research, Casagrande (2003) studied the elastic behaviour and the hydraulic conductivity of well-graded and open-graded aggregates. The shear strength and permanent deformation characteristics of two of those gradations (one well-graded, named GG1, and one open-graded, named GU2) are analysed in this article. A third gradation, also open-graded, named GUM, was chosen in order to study the effect of

grain size in shear strength and permanent deformation. Figure 1 presents the materials grain size distributions as well as Brazilian “A” grading envelope for pavement bases.

Table 2. Aggregates characteristics (Casagrande, 2003).

Characteristic	Basalt aggregate result (%)	Specified values by Brazilian standards (%)
Weight loss in Los Angeles abrasion test	16	≤ 55
Weight loss in soundness test	6.7	≤ 12
Sand equivalent	73.8	≥ 30
Absorption (ME 195/94)	0.5	–

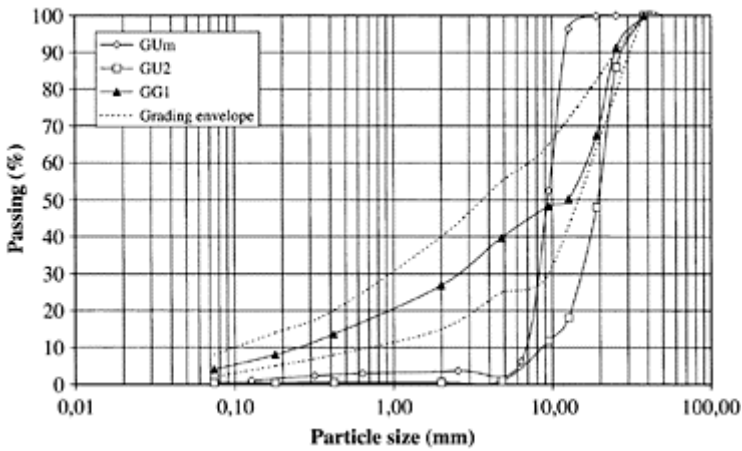


Figure 1. Aggregates grain size distributions and Brazilian “A” grading envelope.

Table 3. Particle size analysis.

Gradation	% passing in #4 sieve	d_{10} (mm)	d_{60} (mm)	d_{90} (mm)	Cu
GUm	6	6.9	10	12	1.5
GU2	1	8.6	21	27	2.4
GG1	40	0.25	16	25	64

Table 4. Compaction and CBR results.

Gradation	Specimen compaction moisture content (%)	γ_d (kN/m ³)	CBR (%)

GG1	5.1	22.8	169
GU2	1.5	17.9	72
GUm	2.0	17.9	37

Table 3 presents the percentages passing in a #4 sieve, particle sizes corresponding to passing percents of 10; 60 and 90% (d_{10} ; d_{60} and d_{90} , respectively) and the coefficient of uniformity (C_u) for each gradation. It may be observed that gradations GU2 and GUm are quite uniform; the former presenting larger grain size.

3.3 Specimens' compaction characteristics and CBR

Table 4 presents the moisture contents and dry unit weight for specimens' compaction, as well as CBR tests results. Specimens were compacted at modified energy (ASTM D1557–00 Method C) and tests were carried out without any scalping.

GG1 gradation, due to its grain size distribution and high bearing capacity, is commonly used in bases construction. GU2 gradation is sometimes used as sub-bases and draining layers materials, while GUm gradation is being tested in permeable pavements (pavements that function as water reservoirs in order to control surface water seepage in urban roads).

4 SHEAR STRENGTH AND PERMANENT DEFORMATION RESULTS

4.1 Shear strength

Figure 2 presents failure envelopes for the three studied gradations. Shear strength parameters corresponding to failure and to some strain levels are shown in Table 5.

For each gradation, failure strains (ε_f) depended on the confining stress applied during the test; therefore average ε_f values were computed, resulting 2.1%; 3.7% and 2.4% for GUm, GU2 and GG1 gradations, respectively.

The shear strength of the well-graded aggregates was quite higher than those of GU2 and GUm gradations at any level of confining stress. Since the angle of internal friction of GG1 and GU2 gradations are very similar, the higher strength of the GG1 gradation was ascribed to its high cohesive interception ($c'=49$ kPa).

Both open-graded aggregates presented quite similar shear strengths at low confining stresses. Conversely, for σ_3 higher than 40 kPa the strength of GU2 exceeds that of GUm, due to the higher angle of internal friction of the larger particle size GU2 gradation.

Figure 3 shows the evolution of mobilised strength as a function of strain level for the three gradations. For strain levels up to 2.0%, specimens of open-graded aggregates (GU2 and GUm) mobilised higher values of Φ' than those of GG1 gradation (see Table 5). On the other hand, the mobilised

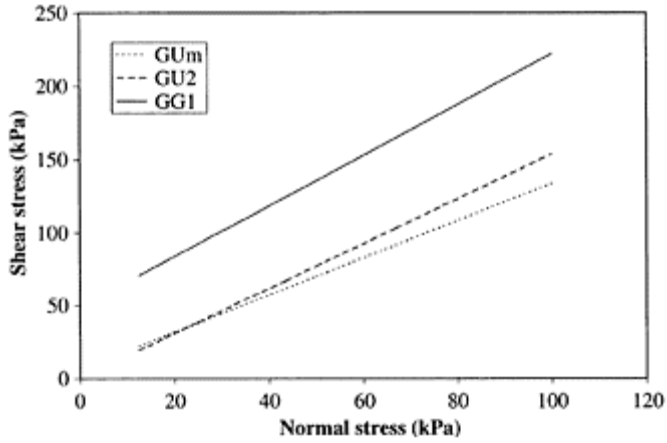


Figure 2. Failure envelopes.

Table 5. Shear strength parameters corresponding to failure and other strain levels.

	$\varepsilon=0.5\%$		$\varepsilon=1.0\%$		$\varepsilon=1.5\%$		$\varepsilon=2.0\%$		Failure	
Gradation	c' (kPa)	ϕ' ($^{\circ}$)	c' (kPa)	ϕ' ($^{\circ}$)	c' (kPa)	ϕ' ($^{\circ}$)	c' (kPa)	ϕ' ($^{\circ}$)	c' (kPa)	ϕ' ($^{\circ}$)
GUm	2	41	5	48	6	50	5	51	6	52
GU2	14	32	7	48	0	54	0	55	0	57
GG1	6	33	35	38	65	48	55	56	49	60

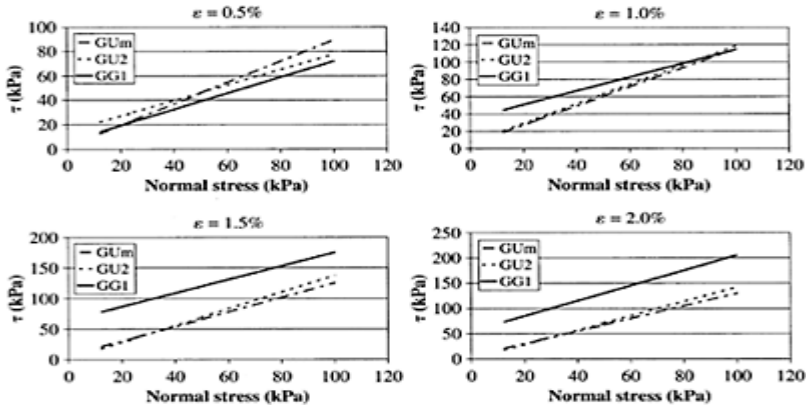


Figure 3. Mobilised strength envelopes at various strain levels.

shear strength of the well-graded aggregates exceeded those of open-graded materials at any strain level from 1.0% on, a fact attributed to the high cohesive interception of the GG1 gradation.

4.2 Permanent deformation

The typical strain response of granular materials under repeated loading and the results of multistage tests for the studied gradations are shown in Figure 4. The numbers in the boxes indicate the deviator stress applied in each stage and the corresponding stress ratio $\sigma_d/\sigma_{1,f}$.

Three stages may be identified:

- an initial permanent strain (ε_{pi}), accumulated in the very beginning of test, reflecting some kind of post-compaction, followed by
- a second stage with permanent deformations accumulating very slowly; for which a constant strain rate (CSR) may be computed, and
- an increasing strain rate stage, observed if σ_d exceeds a certain threshold, which may cause specimen's failure.

Figure 4 reveals better permanent deformation behaviour for well-graded aggregates (GG1 gradation) comparing to open-graded materials (GU2 & GUm gradations). As expected, the well-graded material resisted much higher deviator stresses. Although the shear strength parameters of both open-graded materials were rather similar, their permanent deformation behaviours were quite different. The GU2 specimen accumulated higher permanent deformations. At the end of the first stage, with deviator stresses that barely differed, ε_p in GU2 specimen doubled that in the GUm specimen. After 400,000 loading cycles, the deformation in the GUm specimen was close to 1.5%, while the GU2 had already failed. It is worth to note that the deviator stresses applied in both specimens were rather close.

In spite of its low bearing capacity (CBR=37%), GUm gradation presented a surprisingly good behaviour regarding permanent deformation. All in all, it seems that neither CBR nor shear strength parameters are indicative of the permanent deformation behaviour of unbound aggregates.

The aggregates permanent deformation characteristics (initial permanent deformation and constant strain rate) may be modelled as functions of the applied deviator stress, as in equations (2) and (3), or of the stress ratio $\sigma_d/\sigma_{1,f}$, as in equations (4) and (5).

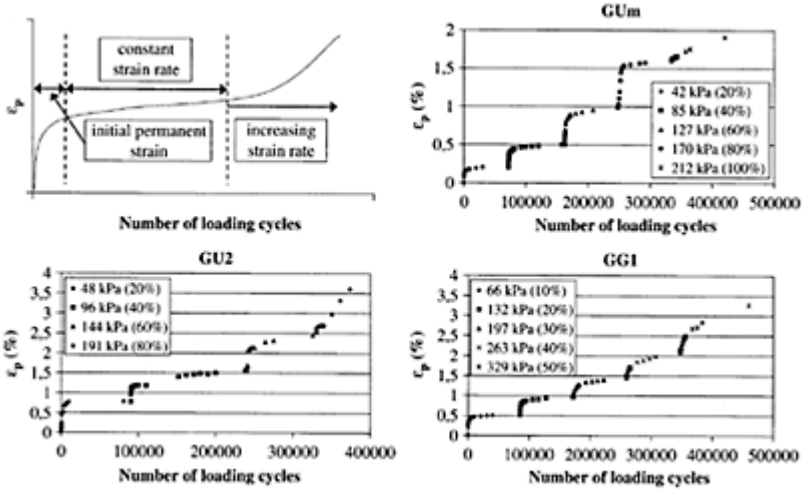


Figure 4. Typical strain response and deformation evolutions in multi-stage tests.

$$\varepsilon_{pi} = a \cdot c^{b \cdot \sigma_d} \quad (2)$$

$$CSR = c \cdot e^{d \cdot \sigma_d} \quad (3)$$

$$\varepsilon_{pi} = f \cdot e^{\frac{g \cdot \sigma_d}{\sigma_{1,f}}} \quad (4)$$

$$CSR = h \cdot e^{\frac{i \cdot \sigma_d}{\sigma_{1,f}}} \quad (5)$$

where ε_{pi} is the initial permanent strain (%); CSR is the constant strain rate (%/cycle); σ_d is the applied deviator stress (kPa); $\sigma_{1,f}$ is the vertical failure stress given by equation 1, considering confining stress of 21 kPa; a, b, c, d, f, g, h and i are models parameters; and $e=2.7183$.

Table 6 presents models parameters and Figure 5 shows the dependency of ε_{pi} and CSR on deviator stress σ_d and stress ratio $\sigma_d/\sigma_{1,f}$, respectively.

Figure 5 shows that, at any level of applied deviator stress, both initial permanent deformation and constant strain rate were higher in the GU2 specimen, which failed at a deviator stress of

Table 6. Models parameters in equations (2) to (5).

Gradation	ε_{pi} model parameters			CSR model parameters		
	a	b	R ²	c	d	R ²
GUm	1.44×10^{-1}	1.28×10^{-2}	0.94	5.58×10^{-8}	1.92×10^{-2}	0.81

GU2	5.32×10^{-1}	8.96×10^{-3}	0.97	3.52×10^{-8}	3.44×10^{-2}	0.97
GG1	3.54×10^{-1}	6.28×10^{-3}	0.98	2.37×10^{-7}	1.00×10^{-2}	0.99
	f	g	R ²	h	i	R ²
GUm	1.44×10^{-1}	2.71×10^{-2}	0.93	5.55×10^{-8}	4.08×10^{-2}	0.81
GU2	5.34×10^{-1}	2.14×10^{-2}	0.97	3.58×10^{-8}	8.20×10^{-2}	0.97
GG1	3.55×10^{-1}	4.12×10^{-2}	0.98	2.37×10^{-7}	6.60×10^{-2}	0.99

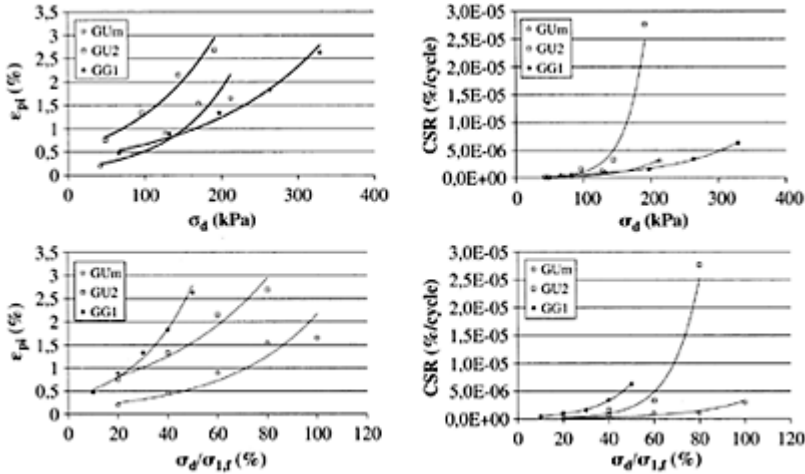


Figure 5. Permanent deformation parameters as functions of deviator stress and stress ratio.

191 kPa. It also shows that for values of σ_d up to 170 kPa the strain rates in GG1 and GUm specimens did not differ. For low deviator stresses (up to 130 kPa) the initial permanent strains in GG1 and GUm specimens were quite similar.

However, different conclusions arise if permanent deformation results are plotted against the stress ratio σ_d/σ_{1f} . It is seen that the GG1 specimen suffered the higher initial permanent deformations at any level of stress ratio up to 50%, and that for the same levels of σ_d/σ_{1f} , deformations accumulated faster in the GG1 specimen (higher CSR). These observations conflict with data presented in Figure 4 and demonstrate that the analysis must be done in terms of absolute values of deviator stress and not in terms of percents of failure vertical compressive stress, as previously concluded by Lekarp *et al.* (1996).

6 CONCLUSIONS

This paper presented and discussed the results of shear strength and permanent deformation tests carried out on unbound aggregates used in southern Brazil for road construction. Three gradations, one well-graded, GG1, and two open-graded, GU2 and GUm, were chosen, in order to quantify the effects of fine aggregates (percent passing in a #4 sieve), particle size and stress state on shear strength and permanent deformation. The study main conclusions were:

- The shear strength of the well-graded aggregates was quite higher than those of GU2 and GUm gradations at any level of confining stress. Since the angle of internal friction of GG1 and GU2 gradations are very similar (60° and 57° , respectively), GG1 higher strength was ascribed to its high cohesive interception ($c' = 49$ kPa).
- Both open-graded aggregates presented quite similar shear strengths at low confining stresses. Conversely, for σ_3 higher than 40 kPa GU2 strength exceeded that of GUm, due to the higher angle of internal friction of the larger particle size GU2 gradation.
- Although the shear strength parameters of both open-graded materials were rather similar, their permanent deformation behaviours were quite different. At any level of applied deviator stress, both initial permanent deformation (ε_{pi}) and constant strain rate (CSR) were higher in the GU2 specimen.
- GUm gradation presented a surprisingly good behaviour regarding permanent deformation. For values of σ_d up to 170 kPa, ε_{pi} and CSR in GG1 and GUm specimens did not differ. It is pointed out that the permanent deformation analysis must be done in terms of absolute values of deviator stress and not in terms of percentage of failure vertical compressive stress.
- All in all, the well-graded material presented the best results in terms of shear strength and permanent deformation. Therefore, its use in pavement bases is justified. The two open-graded materials presented rather high angles of internal friction and acceptable permanent deformation behaviours at stress levels normally acting in sub-bases. GUm gradation, despite its low CBR, presented a surprisingly good behaviour regarding permanent deformation, suggesting that neither CBR nor shear strength are good indicators of deformation in unbound aggregates.
- Considering the high ε_{pi} accumulated in multi-stage tests, it might be advisable to allow machine traffic on recently compacted granular layers before constructing an upper layer. This is especially recommended when using uniformly graded aggregates, of problematical compaction.

REFERENCES

- Arnold, G. 2000. Performance Based Specifications for Road Construction and Materials. *Unbound Aggregates in Road Construction*. Rotterdam:A.A.Balkema, pp. 183–191.
- Casagrande, F. 2003. *Study of the influence of fine content on the hydraulic conductivity and elastic deformability of unbound aggregates*. MSc Thesis. Porto Alegre: Federal University of Rio Grande do Sul, Brazil, 145 pp. (In Portuguese).
- Garg, N. & Thompson, M.R. 1997. Triaxial Characterization of Minnesota Road Research Project Granular Materials. *Transportation Research Record* Washington DC. No. 1577, pp. 27–36.
- Lekarp, F., Richardson, I.R. & Dawson, A. 1996. Influences on Permanent Deformation Behaviour of Unbound Granular Materials. *TR Record* No. 1547, pp. 68–75.
- Lekarp, F. & Dawson, A. 1998. Modeling Permanent Deformation Behaviour of Unbound Granular Materials. *Construction and Building Materials*. Volume 12 No. 1, pp. 9–18.
- Lekarp, F. & Isacsson, U. 2001. The Effects of Grading Scale on Repeated Load Triaxial Tests Results. *International Journal of Pavement Engineering*. Volume 2, No. 2, pp. 85–101.
- Malysz, R. 2004. Mechanical Behaviour of Unbound Aggregates Used in Pavements. MSc Dissertation. Porto Alegre:Federal University of Rio Grande do Sul, Brazil. 158 pp. (In Portuguese)
- van Niekerk, A.A., van Scheers, J., Muraya, P. & Kisimbi, A. 2000. The Effect of Compaction on the Mechanical Behaviour of Mix Granulate Base Course Materials and on Pavement Performance. *HERON*. vol. 45, No. 3, pp. 197–218.
- Theyse, H.L. 2000. The Development of Mechanistic-Empirical Permanent Deformation Design Models for Unbound Pavement Materials from Laboratory and Accelerated Pavement Test Data. *Unbound Aggregates in Road Construction*. Rotterdam: A.A.Balkema, pp. 285–293.
- Werkmeister, S., Numrich, R. & Wellner, F. 2000. Resilient and Permanent Deformation of Unbound Granular Materials. *Unbound Aggregates in Road Construction*. Rotterdam: A.A.Balkema, pp. 171–180.

Modeling of material crushing in granular road bases

S.Lobo-guerrero & L.E.Vallejo

*Department of Civil and Environmental Engineering University of
Pittsburgh, USA*

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ABSTRACT: Crushing of a granular material was investigated. The material was subjected to compressive loads and a combination of compressive and shear loads. These type of loads are effective in granular bases under flexible pavements. The crushing tests used a weak material (sugar) and standard geotechnical engineering equipment (a compression and a direct shear apparatus). Sieve analysis and photographs obtained using a microscope were made before and after testing in order to evaluate the level of crushing in the samples. The Young's modulus of elasticity, E , of the samples was evaluated from the compression tests. E increased in value as a result of particle rearrangement and particle abrasion during compression and was found to decrease slightly as a result of particle crushing. The results from the direct shear tests show that the angle of shearing resistance of the samples decreased slightly as a result of crushing. A combination of low values of shear and compressive stresses produced a degree of crushing in the samples that was similar to that produced when a high level of compression was used alone.

1 INTRODUCTION

Granular materials forming part of the base of flexible pavements experience crushing as a result of static and dynamic loads. Figure 1 shows the type of stresses experienced by a granular base when a wheel travels on the pavement surface (Jessberger & Dorr 1981). Before the wheel reaches a point D on the granular base, this point is subjected to a combination of normal and shear stresses. When the wheel is directly on top of point D, the granular base is subjected to a normal stress only. After the wheel passes point D, the granular base at this point is subjected again to a combination of normal and shear stresses. This study reports the results of crushing testing on granular materials when subjected to compressive loads only, and to a combination of compressive and shear loads. The granular material used is sugar. This material breaks easily under loads

exerted by standard geotechnical engineering equipment. Sugar has been used previously by researchers interested in the formation and the breakage of granular materials located in fault zones (Mandl et al. 1977). Thus, sugar simulates well the fragmentation behaviour experienced by granular bases.

The main focus of this study is to determine if the level of crushing experience by granular materials under compression only can also be obtained using smaller values of compressive loads in conjunction with shear loads. Also, the evolution of crushing in the granular materials was assessed from photographs of the materials before and after crushing obtained using a microscope. Changes in the Young's elastic modulus as a result of compression and changes in the friction angle in the material during the combination of compressive and shear loads were also evaluated.

2 JUSTIFICATION OF THE RESEARCH

Granular materials forming part of the base of flexible pavements are subjected during their engineering lives to both static and dynamic loads. As a result of these loads particle breakage occurs.

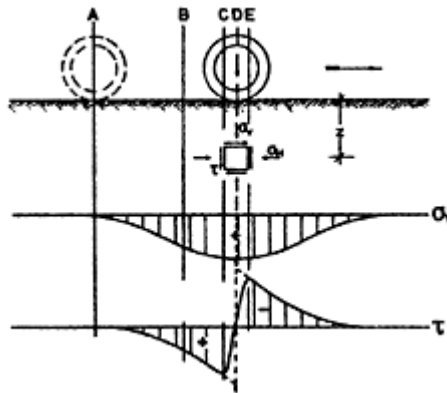


Figure 1. Stresses induced in a granular base by a moving wheel (Jessberger & Dorr 1981).

Particle breakage causes settlements of the pavement structure. Also, as a result of grain breakage, the granular base will experience a reduction in its hydraulic conductivity, and its elastic moduli will change from their original values. Because of crushing, the original engineering properties with which the pavement was designed will change during its engineering life. Changes in the original engineering properties could affect the stability of the pavement reducing its serviceable life. The objective of this study was to conduct a laboratory investigation on the evolution of crushing in a granular material as a result of: (a) compression loads, and (b) a combination of compression and shear loads. The influence of crushing on the Young's modulus of elasticity and on the friction angle of

the materials was also investigated. The level of crushing in the granular material was evaluated from sieve analysis and photographs.

3 PREVIOUS WORK

Based on laboratory tests, some factors have been associated with the occurrence of crushing in granular materials (Lee & Farhoomand 1967), (Hagerty et al. 1993), (Hardin 1985), (Lad et al. 1996):

- Crushing is directly related to particle hardness.
- A uniform granular material composed by big particles exhibits more crushing than one composed by smaller particles of the same material.
- Angular particles exhibit more crushing than rounded particles.
- Uniform soils exhibit more crushing than well graded soils.
- Crushing of the granular media continues with time.

Also, the references cited above highlight the influence of the ratio between the principal stresses on the crushing occurrence. Most of the tests that were carried out to arrive to these conclusions used triaxial compression and uniaxial compression machines mainly on glass beads and sands.

Other efforts have focused on understanding the relation between shear strength and crushing. Shear strength tests on glass beads and sands established that these materials develop a non linear failure envelope because of crushing (Fedá 2002). For granular materials, a clear relationship was found to exist between the void ratio and the imposed vertical stress in confined uniaxial tests (Terzaghi & Peck 1948), (McDowell & Bolton 1998), (Cheng et al. 2003). From these tests, engineering parameters such as the compression index, C_c , were found to be affected by the occurrence of crushing.

4 MATERIALS USED AND TESTING PROGRAM

Studying crushing of granular materials has always been limited by the large capacity machines needed to develop considerable loads that can lead to the grain fragmentation in granular assemblies. One approach to solve this problem is to use standard geotechnical equipment with weak materials (sugar, corn flakes) (Mandl et al. 1977), (McDowell & Bolton 1998). In this study standard geotechnical laboratory equipment and a weak material (sugar) were used. The sugar used had an average diameter equal to 1.015 mm (material passed the No. 16 sieve and was retained in No. 20 sieve). The specific gravity of this sugar, G_s , was equal to 1.5. The natural angle of repose of this material (α) was found to be 40°. This angle of repose is a measure of the angle of friction of the material. Samples of this sugar were used in the normal compression and direct shear tests. During the testing program, the humidity of the air was equal to 15%. At this level of humidity the sugar did not experience any visible change in its original structure.

4.1 Compression tests

A Versa Loader machine was used in the compressive tests. The sugar was placed in a loose state inside a Plexiglas cylinder having an internal diameter equal to 5 cm. The samples were subjected to loading and unloading conditions in the cylinder (loading and unloading rate=0.063 in/min). The vertical deformation of the sample was continuously recorded using a LVDT transducer. The samples were subjected to 9 different vertical compressive stresses. The vertical stresses used range in value from 100 kPa to 1548 kPa. At the end of each test, the samples were subjected to a sieve analysis. Photographs of the samples before and after crushing were also obtained. These photographs were taken using a microscope.

The relationship between the applied vertical compressive stress and the void ratio for the samples subjected to five of the vertical stresses used is showed in Figure 2.

Figure 2 represents a typical trend exhibited by a granular material when subjected to vertical compressive stresses as reported by other researchers (Terzaghi & Peck 1948), (McDowell & Bolton 1998), (Cheng et al. 2003). In Figure 2, three different stages can be distinguished. In the

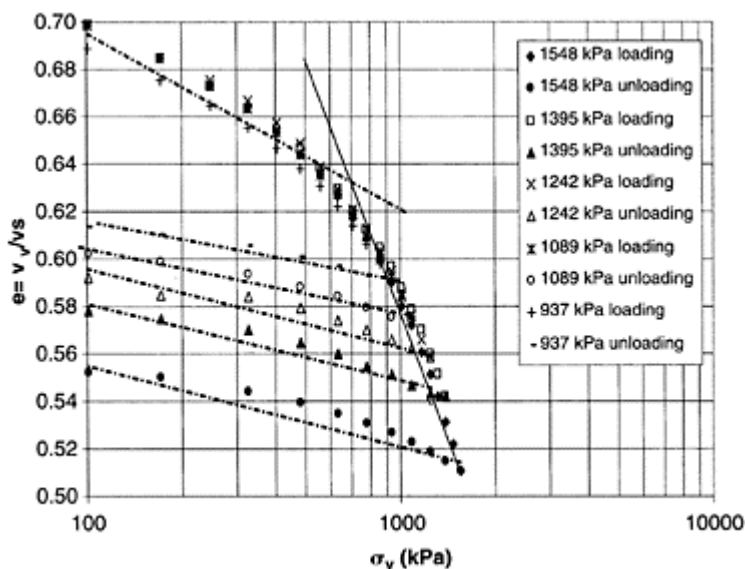


Figure 2. Vertical stress vs. void ratio in normal compression.

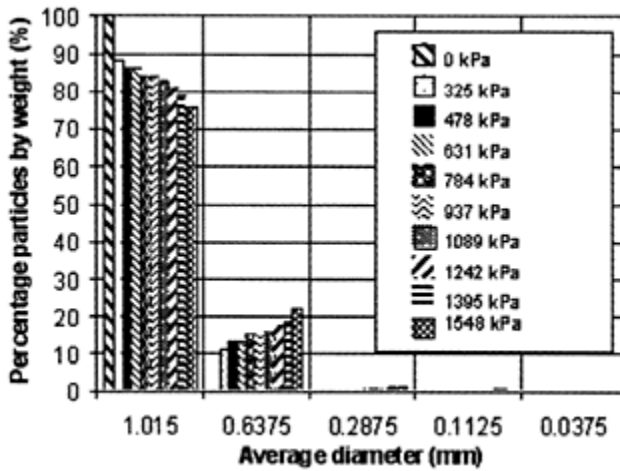


Figure 3. Particle size distribution after the test.

first stage (for a vertical stress between 100 and 700 kPa), the void ratio changes very little (from 0.695 to 0.635). This small change in void ratio is the result of grain rearrangement during compression and a slight level of crushing in the form of abrasion of the particles. The second stage (for a vertical stress between 700 and 1548 kPa), the void ratio changed substantially (from 0.635 to 0.51). This change in void ratio was the result of crushing of the sugar. The third stage takes place during the unloading of the samples. The void changes in the void ratios in this unloading stage are the result of the elastic rebound of the samples.

Sieve analysis was carried out on the samples before and after the compression tests depicted in Figure 3. The sugar grains had an average diameter equal to 1.015 mm before they were subjected to the compressive loads. After the compressive loads were in effect on the samples, some of the grains broke. Figure 3 indicates the percentages of the different sizes in the samples after the compression tests. This figure shows that as the compression levels increased, the percentage of the grains with the original size decreased, and the percentage of the grains with size smaller than the original size (1.015) increased.

The Young's modulus of elasticity can be obtained from the compression-uniaxial strain relationships. The elastic modulus, E , changes during compression and can be obtained at different points of the compression-axial strain curve from the ratio between a small increment in vertical stress ($d\sigma$) and the corresponding increment in vertical strain ($d\varepsilon$). Since the increments in vertical strain and vertical load were recorded at every step of the compression tests, a relationship between the vertical stress and $d\sigma/d\varepsilon$ was established. Figure 4 shows a plot of the elastic modulus as it changes with the levels in the compression. An analysis of Figure 4 indicates that the elastic moduli increases with the levels of compression reaching a maximum at a value of compressive stress equal to 700–800 kPa. After this compressive stress is reached, the modulus of elasticity decreases slightly. If one considers Figure 2, the compressive stress separating the stages of particle

rearrangement and crushing is equal to 700 kPa. Beyond the stress equal to 700 kPa, the sugar grains seem to experience crushing. If one now considers Figure 4, at stresses greater than 700 kPa, the elastic moduli decreased slightly. From a comparison of Figure 2 and 4 it can be concluded that the elastic moduli of the granular material increases as a result of particle rearrangement and particle abrasion and tends to decrease slightly as a result of particle crushing.

The obtained results can be used to understand the implications of having rearrangement of particles, abrasion, and crushing in granular bases and the effects that these phenomena can generate on the pavement structure: reduction in bulk volume and associated settlements, reduction in permeability due the generated fines and the reduction in the pore spaces, and a reduction in the

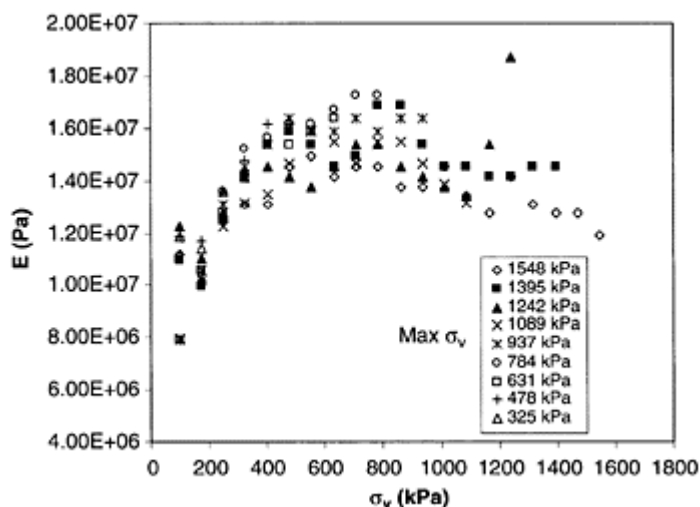


Figure 4. Vertical stress, σ_v , vs
Young's Modulus of Elasticity, E .

elastic modulus. Nevertheless, granular bases may not be subjected to vertical stresses as high as the ones reported above, and perhaps is more likely that granular materials are subjected to normal stresses that are smaller than the ones used in the compression tests in conjunction with moderate shear stresses. The next section describes direct shear tests on the sugar in order to investigate if the same degree of crushing obtained in the compression tests can be obtained under direct shear conditions using stresses that are smaller than those used in the compression tests.

4.2 Direct shear tests

A circular direct shear box having an inside diameter of 6.35 cm and a sample height of 2.1 cm was used in the direct shear tests. For the direct shear tests, normal stresses that range in value between 110 kPa and 225 kPa were used. These normal stresses are substantially smaller than those used in the compression tests (Figures 2 to 4). The

samples were prepared by placing the sugar in a loose state inside the shear box; then, the shear box was vertically loaded to normal stresses that varied between 110 and 225 kPa. These normal stresses were kept on the samples for a period of 10 minutes. During this time, the samples deformed vertically. After this, the samples were subjected to shear stresses. Changes experienced by the samples were recorded at 3, 6 and 9 mm horizontal deformation of the samples. After each of these horizontal deformations were reached by the samples, they were removed from the shear box and were subjected to sieve analysis and microscopic inspection. For each normal stress used, three different samples (deformed to 3, 6 and 9 mm) were used for the shear-deformation analysis.

Figure 5 shows the curves relating the friction coefficient (τ/σ) and the level of horizontal deformation for the five normal stresses used in the direct shear tests. Figure 5 indicates very little variation of the coefficient of friction with respect to the normal stresses used in the tests. An average value for maximum coefficient of friction regardless of the normal stress used seems to be equal to 0.82. This coefficient of friction corresponds to an angle of shearing resistance equal to 39.4 degrees. This angle of shearing resistance is slightly lower than the angle of friction before crushing (measured by the angle of repose of the original sugar grains which is equal to 40 degrees). Thus, crushing seems to affect very little the shear strength of the sugar.

The changes experienced by the sugar grains during the direct shear testing was evaluated using sieve analysis and photographs obtained before and after crushing using a microscope.

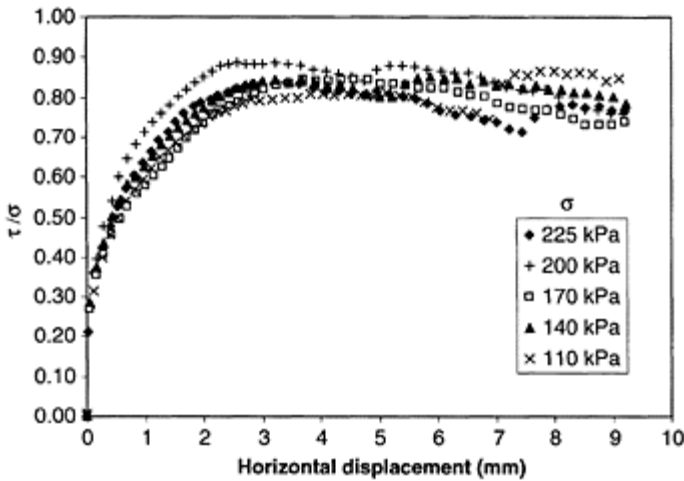


Figure 5. Horizontal deformation vs. friction coefficient.

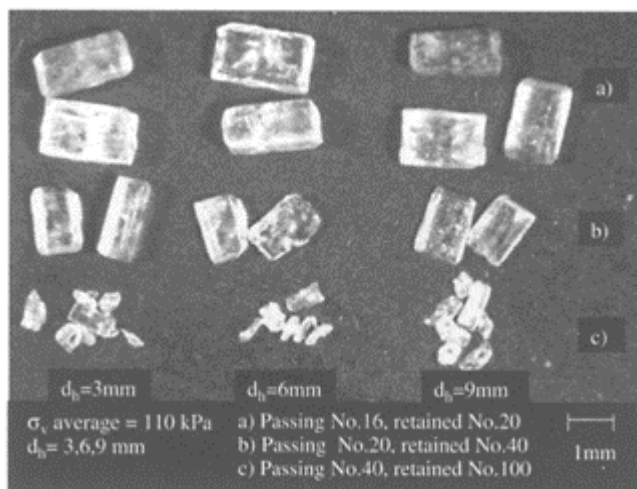


Figure 6. Samples subjected to an average vertical stress of 110 kPa.

4.3 Photographs

Figure 6 shows a picture of some particles from the three different samples subjected to the same average vertical stress but to a different horizontal deformation. Figure 6 is a photograph of the grains for the case in which the samples were subjected to the smallest of the normal stress value. This normal stress was equal to 110 kPa. Figure 7 shows a picture of some particles from the three different samples all subjected to a normal stress equal to 225 kPa. This normal stress was the largest one used in the direct shear tests. In Figures 6 and 7, the sugar grains located in a row represent the size of the grains passing and being retained on certain sieves. The sugar grains in the column of the photographs represent the different sugar grain sizes that the samples developed at different values of the horizontal deformation in the shear tests.

An analysis of Figures 6 and 7 indicates that the samples break and develop fines as the normal stress is increased on the samples or as the amount of deformation is also increased. During the combination of normal and shear stresses, the sharp corners of grains break producing the smaller

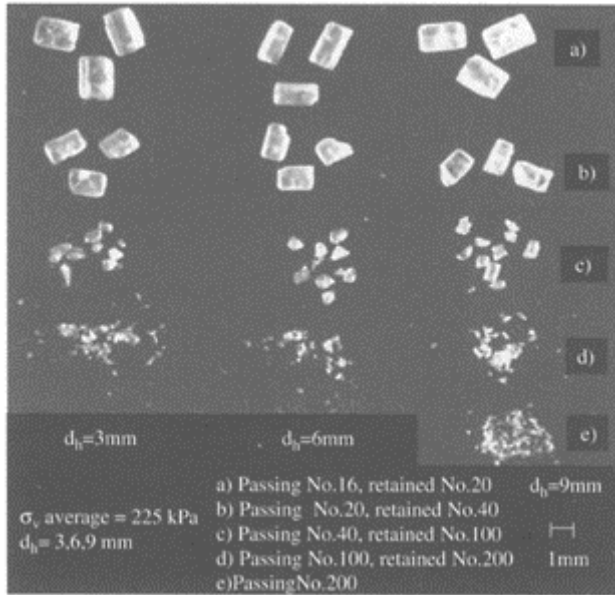


Figure 7. Samples subjected to an average vertical stress of 225 kPa.

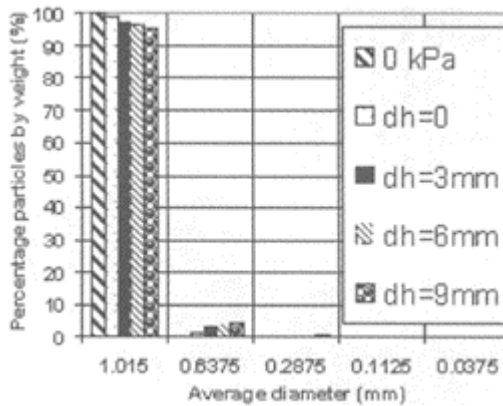


Figure 8. Particle size distribution, $\sigma_v=110\text{ kPa}$.

particles in Figures 6 and 7. The larger particles also break in two large pieces. The fragmentation of the sugar during the direct shear tests changes the sugar from a uniform granular material into a somewhat well graded material. The breakage of the sugar grains caused a decrease in the volume of the material during shear. This volume decrease was caused when the smaller material resulting from the breakage moved into the voids

located in between the original larger sugar grains that did not break during shear. The smaller material located in between the larger grains also served as roller bearings during shear. This roller bearing effect seems to explain the decrease in frictional resistance that the samples experienced as a result of shearing.

4.4 Sieve analysis

The samples subjected to the direct shear tests were subjected to a sieve analysis. The results of the sieve analysis are depicted in Figures 8, 9 and 10. Figure 8 represents the sieve analysis for the

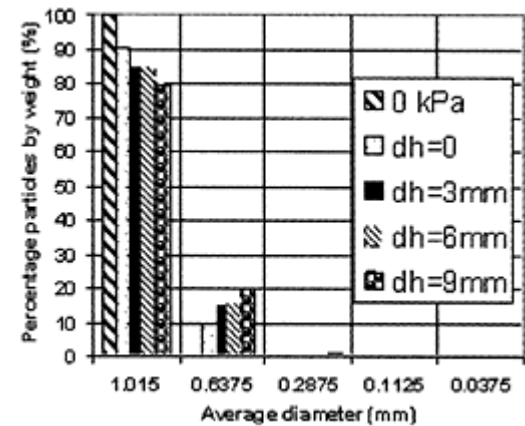


Figure 9. (left) Particle size distribution, $\sigma_v=200$ kPa.

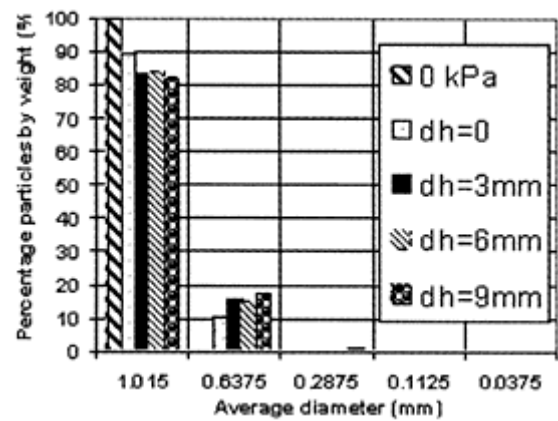


Figure 10. (right) Particle size distribution, $\sigma_v=225$ kPa.

samples subjected to a 110 kPa normal stress. Figure 9 for the samples subjected to a 200 kPa normal stress. And Figures 10 for the samples subjected to a 225 kPa normal stress. Figures 8 to 10 also indicate the size distribution of the grains for the different values of horizontal deformation in the direct shear tests. Under 110 kPa, the samples experienced very small amount of breakage. Figures 9 and 10 show the results for similar samples subjected to a vertical stress of 200 kPa and 225 kPa. A comparison of Figures 9 and 10 shows that the two plots are very similar. The similarity of these plots is a sign that no crushing is generated by an increment in the vertical stress. The results of Figures 9 and 10 seem to indicate that the samples tend to reach a stable structure after which very little fragmentation takes place. These results are in agreement with those obtained by Lang (Lang 2002) who modeled crushing in granular materials under direct shear conditions using the Discrete Element Method.

Comparing the amount of crushing obtained under a uniaxial compression stress of 1395 kPa (Figure 3) and the crushing results obtained in the direct shear test under vertical stresses of 200 kPa or 225 kPa and a horizontal deformation of 9 mm, it can be established that the same degree of crushing was reached in the samples. However, the level of vertical stress used in the compression test was about six times greater than the normal stress used in the direct shear testing. Thus crushing can be generated in granular materials under low values of normal stress if this stress is used in conjunction with a shear stress.

5 CONCLUSIONS

The crushing of a granular material was investigated in the laboratory. The crushing was produced when a granular material was subjected to compressive loads and a combination of compressive and shear loads. These types of loads are effective on granular bases under flexible pavements. The crushing tests used a weak material (sugar) and standard geotechnical engineering equipment (a compression and a direct shear apparatus). Even though sugar does not accurately represent a granular base, its crushing behaviour simulates well the one experienced by granular bases. Sieve analysis and photographs obtained using a microscope were made before and after testing in order to evaluate the level of crushing in the samples. The Young's modulus of elasticity, E , of the samples was evaluated from the compression tests and was found to increase with higher levels of compression. E increased in value as a result of particle rearrangement and particle abrasion during compression and was found to decrease slightly as a result of particle crushing. The angle of shearing resistance of the samples decreased slightly as a result of crushing and was found to be relatively constant regardless of the value of the normal stress used in the direct shear testing. A combination of low values of shear and compressive stresses produced a degree of crushing in the samples that was similar to that produced when a high level of compression alone was exerted on the samples.

ACKNOWLEDGEMENTS

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REFERENCES

- Cheng, Y.P., Nakata, Y., Bolton, M.D. 2003. Discrete element simulation of crushable soil, *Geotechnique* 53(7): 663–641.
- Feda, J. 2002. Notes on the effect of grain crushing on the granular soil behavior, *Engineering Geology* 63:93–98.
- Hagerty, M.M., Hite, D.R., Ulrich, C.R. 1993. One dimensional high pressure compression of granular media, *Journal of Geotechnical Engineering* 119(1): 1–18.
- Hardin, B.O. 1985. Crushing of soil particles, *Journal of Geotechnical Engineering* 111(10): 1177–1190.
- Jessberger, H.L., Dorr, R. 1981. Behaviour of dynamically loaded granular materials, *Proc. of the 10th Int. Conf. On Soil Mech. And Found. Eng.*, Stockholm, vol. 1:655–660.
- Lade, P.V., Yamamuro, J.A., Bopp, P.A. 1996. Significance of particle crushing in granular materials, *Journal of Geotechnical Engineering* 122(4): 309–316.
- Lang, R.A. 2002. *Numerical simulation of comminution in granular materials with an application to fault gouge evolution*, (Unpublished Master of Science Thesis, Texas A&M University).
- Lee, K.L., Farhoomand, I. 1967. Compressibility and crushing of granular soil in anisotropic triaxial compression, *Canadian Geotechnical Journal* IV(1): 68–86.
- Mandl, G., Jong, L.N.J., Maltha, A. 1977. Shear zones in granular material, *Rock Mechanics* 9:95–144.
- McDowell, G.R., Bolton, M.D. 1998. On the micromechanics of crushable aggregates, *Geotechnique*, 48(5): 667–679.
- Terzaghi, C., Peck, R. 1948. *Soil mechanics in engineering practice*, New York, Ed Jhon Wiley & sons: 58–61.

Fractal analysis of the abrasion and crushing of gravels

L.E.Vallejo, Z.Chik & S.Tucek

*Department of Civil and Environmental Engineering, University of
Pittsburgh, USA*

B.Caicedo

*Department of Civil and Environmental Engineering, Universidad de los
Andes, Bogota, Colombia*

Pavements Unbound—Dawson (ed.)

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ABSTRACT: Gravels forming part of the base of flexible pavements experience abrasion and crushing as a result of static and dynamic loads. Abrasion takes place when the sharp corners of the particles of gravel are removed as a result of compressive and shear loads. As a result of abrasion, the particles change in shape. Crushing is caused by the fragmentation of the particles into a mixture of many small particles of varying sizes. In this study, the abrasion and crushing of gravels are evaluated experimentally and analytically. The laboratory component of this study involves gravels that were subjected to abrasion and dynamic compression tests. The evaluation of the abrasion and crushing experienced by the gravel was carried out using fractals. In this study, the fractal dimension concept from fractal theory is used to evaluate: (a) the changes in shape, and (b) the crushing (fragmentation) of the original particles of gravel. It was determined that the fractal dimension of the profile of the particles decreased as a result of abrasion. With respect to crushing, the fragmentation fractal dimension was found to increase with the degree of breakage of the gravel. To understand the influence of crushing on the permeability of the gravels, the hydraulic conductivity of the gravels was measured before and after crushing. The hydraulic conductivity of the gravels was found to decrease with an increase in their level of crushing.

1 INTRODUCTION

Gravels form part of the base of flexible pavements. These gravels are subjected during their engineering lives to either static and dynamic loads (Brown and Pappin, 1981). As a

result of these loads, the gravels may experience abrasion and crushing. Because of sustained abrasion and crushing, the original engineering properties with which the base of a pavement structure was designed (i.e., hydraulic conductivity, shear strength, elastic moduli) will change during its engineering life. Changes in the original engineering properties could affect the stability of the structure and could make it unsafe. Thus, there is a need to understand the evolution of abrasion and crushing in granular materials. In this study, the evaluation of abrasion and crushing of gravels is conducted using fractal theory. Laboratory experiments in the form of abrasion and dynamic compression tests are used to induce abrasion and crushing in the gravels.

1.1 *The abrasion and crushing of granular materials*

Granular materials form part of engineering structures such the base of flexible pavements, highway embankments, and foundations. The granular materials forming part of these structures are subjected during their engineering lives to either static or dynamic loads. As a result of these loads, particle abrasion and particle breakage occur (Lee and Farhoomand, 1967; Lade et al., 1996; and Raymond, 2000). According to Lee and Farhoomand (1967), particle breakage or crushing seems to be a general feature for all granular materials. Grain crushing is influenced by grain angularity, grain size, uniformity of gradation, low particle strength, high porosity, and by the stress level and anisotropy (Bohac et al., 2001).

When a granular mass is subjected to a compressive load, the particles resist the load through a series of contacts between the grains. The particles with highly loaded contacts are usually aligned in chains (Cundall and Strack, 1979). Crushing starts when these highly loaded particles fail and break into smaller pieces that move into the voids of the original material. This migration causes the settlement of a granular assembly (Figure 1). Also, on crushing, fines are produced and the grain size distribution curve becomes less steep. Consequently, with continuing crushing, the granular material becomes less permeable and more resistant to crushing. Grain size distribution is a suitable measure of the extent of crushing (Lade et al., 1996).

Lade et al. (1996) found that if a uniform granular material is crushed, the resulting grain size distribution approaches that of a well graded soil for very large compressive loads. McDowell et al. (1996) established that the grain size distribution of a granular assembly that has been crushed under large compressive loads is a fractal distribution. A well graded particle distribution or a fractal distribution represents a granular structure that is made of grains of all sizes including the original unbroken grains.

These original large grains do not break based on the fact that with more small size particles surrounding them, the average contact stress acting on these large grains tends to decrease (Lade et al., 1996). However, before the granular structure reaches a well graded or a fractal particle size distribution, the granular structure will experience gradual changes in particle sizes depending on the magnitude of the compressive load applied to it.

Pavements are the most unusual structures designed by civil engineers. Water enters through their tops, bottoms, and sides, but because pavements are relatively flat, the water flows out again very slowly unless they are well drained under their full width (Cedergreen, 1994). The most serious problems occur in asphalt pavements when their

granular bases are unable to remove the water that enters the pavement. Figure 1(a) represents a well drained granular base assuming drainage goes vertically or laterally. In Figure 1(b) the loose zones that drain the water are interconnected (the dense zones filled with crushed material are not connected). Thus, drainage in the vertical or horizontal direction is still possible. In Figures 1(c) and 1(d), the loose zones that drain the granular base in either the vertical or horizontal direction are no longer connected. These loose zones must be interconnected in order for water to drain from underneath the pavement. In Figures 1(c) and 1(d),

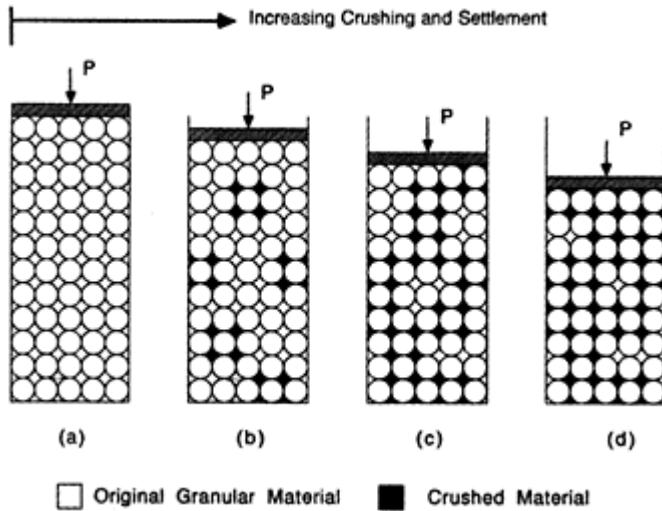


Figure 1. Evolution of crushing in a confined granular material under compression.

the dense zones made of crushed material are the ones that are interconnected. The dense zones made of crushed granular material surround and isolate the loose zones that promoted drainage.

Thus, when the granular base reaches the conditions of Figures 1(c) and 1(d) as a result of crushing, serious problems will develop in pavements. Due to traffic loads, the material in the loose isolated zones will act as closed hydraulic systems that will develop excess pore water pressures, ultimately producing the failure of the granular base as well as the pavement (Cedergreen, 1994). Next, a theoretical method based on fractal theory for evaluating abrasion and complete fragmentation in gravels is presented.

2 FRACTALS AND THE CONCEPT OF THE FRACTAL DIMENSION

The shape of forms in nature is usually analyzed using Euclidean geometry. According to this kind of geometry, straight lines are perfectly straight lines and curves are arcs of

perfect circles. However such perfection is seldom found in natural forms. Most of the time, the shapes of natural forms are irregular. Fractals are a relatively new mathematical concept to describe the geometry of irregularly shaped objects in terms of a fractional number (the fractal dimension) rather than an integer. In this study the fractal dimension concept from fractal theory is used to measure the degree of irregularity of particle profiles. Fractals are also used to evaluate the size distribution in a granular material subjected to varying crushing levels.

2.1 The fractal dimension of closed (particle) profiles: abrasion measurement

Many methods have been developed to measure the fractal dimension of open and closed form profiles such as those constituting part of rock joints, geomembranes, pavements, sands, gravels, and voids in soils (Yeggoni et al., 1996; Vallejo, 2001;). The most commonly used methods are: (a) the divider method, (b) the box method, (c) the area-perimeter method, and (d) the spectral method (Hyslip and Vallejo, 1997). Next, the divider method is presented as a way of measuring the fractal dimension of a closed profile (Figure 2).

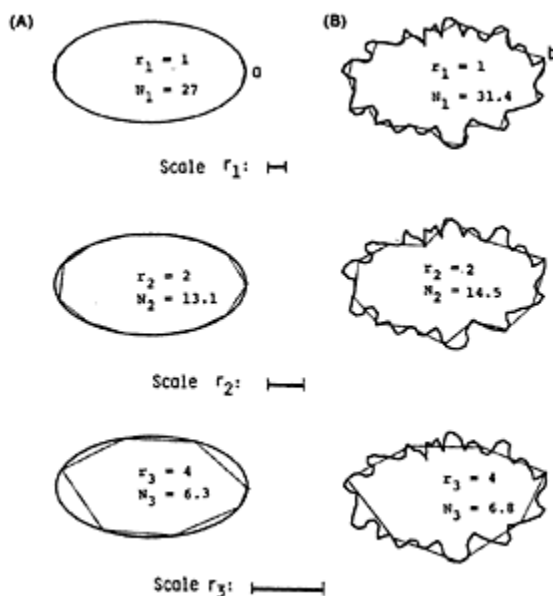


Figure 2. Smooth and rough particle for fractal analysis.

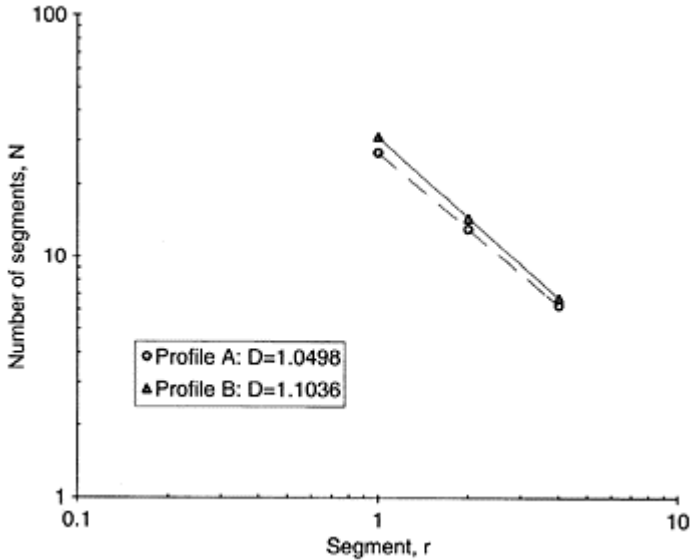


Figure 3. Fractal dimension, D , for particles shown in Figure 2.

Figure 2 represents the profile of two particles having the same cross sectional area but different profiles. Fig. 2(A) shows the two dimensional profile of a smooth, ellipsoidal particle repeated twice. Figure 2(B) shows the profile of a rough, ellipsoidal particle, also repeated twice. Suppose we wish to measure the length L of the simple and complex closed profiles shown in Figure 2 using a ruler or yardstick of fixed length, r . We may begin by setting two arms of a divider to a known distance (step or segment length r) and step off the outline of the profiles as shown in Figure 2. The length of the profiles, L , is obtained from the product of the number of segments, N , and the chosen segment length, r . Three different segment lengths, r , were used to measure both the simple and complex closed profiles. The scales for the length of these segments are shown in Figure 2. The number of segments, N , of each length, r , to cover the profile of the particles is also shown in Figure 2. According to Mandelbrot (1977), if a linear relationship develops between the values N and r when plotted on log-log paper, the profiles analyzed are fractal profiles. The absolute value of the slope of the linear relationship between N and r values represents the fractal dimension, D , of the profiles. The number of segments, N , and the corresponding length of the segments, r , are plotted on log-log paper (Figure 3). The slope of the best fit line passing through the points relating N and r represents the fractal dimension D of the profiles. As expected, the fractal dimension, D , of the rough profile [Figure 2(B)] is greater than the fractal dimension, D , for the smooth profile [Figure 2(A)]. The fractal dimension of the rough profile is equal to 1.1036, and the fractal dimension of the smooth profile is equal to 1.0498 (Figure 3).

Figure 2(B) can represent the profile of one particle *before abrasion* occurs. Figure 2(A) can represent the profile of one particle *after abrasion* occurs. Thus, the fractal

dimension concept can be used to measure abrasion in the particles forming part of granular bases under flexible pavements.

Figure 3 indicates the way to evaluate the fractal dimension, D , for the case of one particle. To evaluate the average fractal dimension of a group of particles, the area-perimeter method is recommended (Hyslip and Vallejo, 1997). The area perimeter-method involves the measurement of the areas and the respective perimeters of the multiple particles forming a group. One then plots on log-log paper the areas and the perimeters of the individual particles. The slope, m , of the best fit line passing through the plotted points is used to calculate the average fractal dimension, D , of the group of particles analyzed. The average fractal dimension, D , is equal to the ratio $(2/m)$ (Hyslip and Vallejo, 1997). The area-perimeter method will be used to measure abrasion levels in gravels.

2.2 Fractal dimension of the grain size distribution: fragmentation measurement

Grain size distribution of naturally occurring soils has been found by Tyler and Wheatcraft (1992) and Hyslip and Vallejo (1997) to be fractal. Tyler and Wheatcraft (1992) have developed a relationship that uses the results of a standard sieve analysis to calculate the fractal dimension, D_F , of the size ditribution of natural soils. This relationship is:

$$\frac{M(R < r)}{M_T} = \left(\frac{r}{r_L} \right)^{3-D_F} \quad (1)$$

where $M(R < r)$ is the cumulative mass (weight) of particles with size R smaller (finer) than a given comparative size r ; M_T is the total mass (weight) of particles; r is the sieve size opening; r_L is the maximum particle size as defined by the largest sieve size opening used in the sieve analysis; and D_F is the fragmentation fractal dimension. The results of a sieve analysis tests using Equation. (1) can be plotted on log-log paper. The slope, m , of the best fitting line through data obtained using Equation (2) and the fractal dimension, D_F , are related as follows:

$$D_F = 3 - m \quad (2)$$

Equations (1) and (2) are used to obtain the fractal dimension of the size distribution in a gravel subjected to crushing. The crushing is the result of dynamic compressive loads. Changes in the degree of crushing of the gravel are reflected in the value of the fractal dimension, D_F , of the size distributions.

3 LABORATORY TESTS AND RELATED FRACTAL DIMENSIONS

3.1 Abrasion tests

A fine gravel ($d_{50}=7$ mm) and a specific gravity, G_s , equal to 2.67 was subjected to abrasion tests. The abrasion tests were performed in a cylindrical jar mill made of ceramic material. The jar mill's diameter was equal to 15.24 cm, and its length was also equal to 15.24 cm. Inside the jar mill, there were 65 ceramic balls (charges), each measuring 2 cm in diameter. Twenty pieces of the gravel under dry conditions were placed inside the cylinder together with the ceramic balls. After this was done, the cylinder was rotated for a period of 1 hour. The frequency of rotation was 50 rpm. During rotation, the interaction between the ceramic balls and the gravel caused the abrasion of the surface of the particles of gravel. This abrasion produced small changes in the profile of the rock samples. Figure 4 shows a photograph of six rock pieces before the abrasion test. Figure 5 shows the same rock pieces after the abrasion tests.

These photographs were then used to obtain the fractal dimension of the group. The fractal dimension of each group of gravel shown in Figures 4 and 5 was obtained using the area-perimeter method, as previously explained. As a result of abrasion, the fractal dimension of the gravel profiles decreased from 1.0588 to 1.0106.

3.2 Crushing tests

For the crushing tests, the same dry gravel was used. The gravel was placed in a metallic cylinder having a diameter equal to 15 cm. The height of the sample in the cylinder measured 20 cm. Using a Standard Proctor hammer, the gravel was subjected to 100, 300, and 500 compressive blows. As a result of the dynamic compressive stresses induced by the falling of the hammer on the samples, the gravel experienced fragmentation. A sieve analysis was conducted on the samples before and after the dynamic compression. From the sieve analysis and the use of Equations (1) and (2), Figure 6 is obtained. Figure 6 shows the fragmentation fractal dimension values, D_F , for the

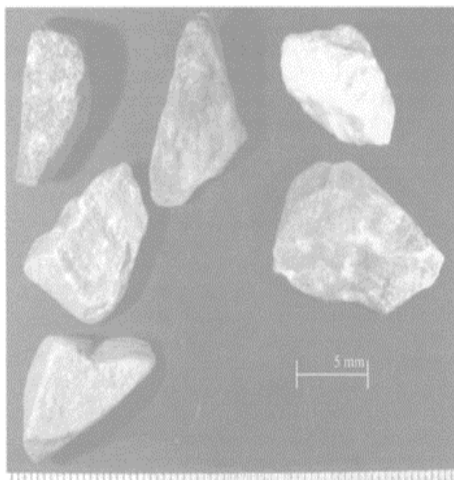


Figure 4. Gravel before the abrasion test. Using the area-perimeter method, the average fractal dimension of the profiles is $D=1.0588$.

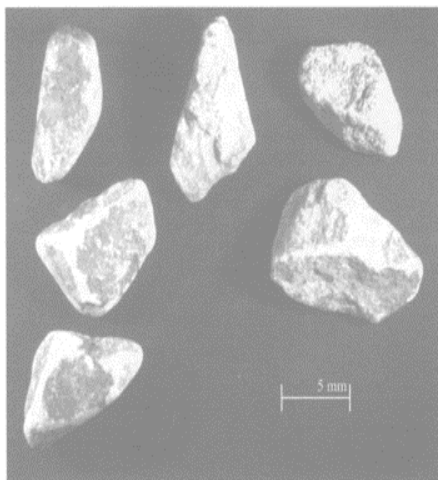


Figure 5. Gravel after the abrasion test. Using the area-perimeter method, the average fractal dimension of the profiles is $D=1.0106$.

different values of compressive blows used to break the gravel. An analysis of Figure 6 indicates that the fragmentation fractal dimension values, D_F , increased in value with the number of blows delivered to the samples. The fragmentation fractal dimension, D_F , for the original sample was equal to 1.892. It increased to 2.3849 after 100 blows, to 2.5237 after 300 blows, and to 2.5795

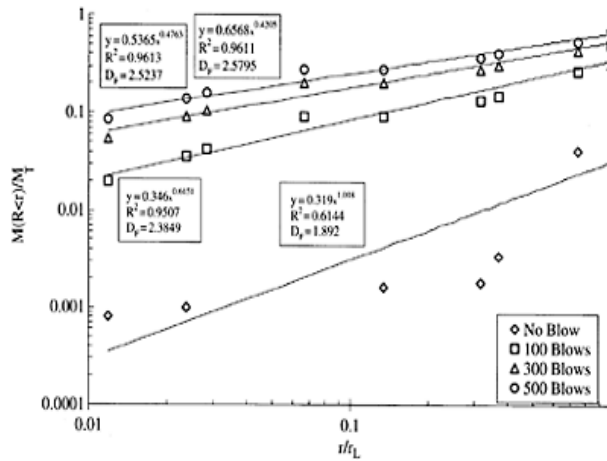


Figure 6. Fractal fragmentation calculation for the gravels before and after their dynamic crushing.

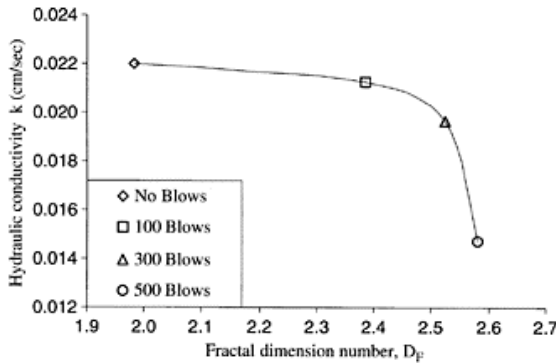


Figure 7. Relationship between the hydraulic conductivity of the samples and their fragmentation fractal dimension.

after 500 blows. In other words, the fractal dimension of size distribution increased in value as a result of the breakage induced in the gravel by the dynamic compression.

3.3 Hydraulic conductivity tests

Before and after each of the crushing tests, constant head hydraulic conductivity tests were performed on the samples of gravel. Figure 7 shows a plot of the hydraulic conductivity tests and the fragmentation fractal dimension values. Figure 7 indicates that the hydraulic conductivity values decreased with the degree of fragmentation of the samples measured by their fragmentation fractal dimension.

4 CONCLUSIONS

The fractal dimension concept from fractal theory has been presented to evaluate abrasion and fragmentation of granular materials. Abrasion tests changed the profile of the gravel from a rough profile to a smoother one. These changes in profile were reflected by their fractal dimension values. Fragmentation was produced by conducting dynamic compression tests on fine gravel. As a result of the compressive loads, the size distribution of the gravel changed from that of a low fractal material to that of a high fractal one. The changes in the particle size distribution in the sand had a large influence on the hydraulic conductivity. The hydraulic conductivity decreased as the particle size distribution changed from a low fractal distribution to a high fractal one.

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REFERENCES

- Bohac, J., Feda, J., and Kuthan, B., 2001. Modelling of grain crushing and debonding. *Proceedings of 15th Int. Conference on Soil Mech. And Geotech. Eng.*, Istanbul, Turkey, 1:43–46.
- Brown, S.F., and Pappin, J.W., 1981. Analysis of pavements with granular bases. *Transportation Research Record*, NRC, 810:17–23.
- Cedergren, H.R., 1994. America's pavements: world's longest bathtubs. *Civil Engineering*, ASCE, September Issue, pp. 56–58.
- Cundall, P.A., and Strack, O.D.L., 1979. A discrete numerical model for granular Assemblies. *Geotechnique*, 29(1): 47–65.
- Hyslip, J.P., and Vallejo, L.E., 1997. Fractal analysis of the roughness and size distribution of granular materials. *Engineering Geology*, 48 (3–4): 231–244.
- Lade, P.V., Yamamuro, J.A., and Bopp, P.A., 1996. Significance of particle crushing in granular materials. *J. of Geotechnical Eng., ASCE*, 122 (4): 309–316.

- Lee, K.L., and Farhoomand, J., 1967. Compressibility and crushing of granular soils in anisotropic triaxial compression. *Canadian Geotechnical J.*, 4(1): 68–86.
- Mandelbrot, B.B., 1977. *Fractals: forms, chance and dimension*. San Francisco, Freeman.
- McDowell, G.R., Bolton, M.D., and Robertson, D., 1996. The fractal crushing of granular materials. *Int. J. of Mechanics and Physics of Solids*, 44(12): 2079–2102.
- Raymond, G.P., 2000. Track and support for a mine company railroad. *Can. Geotech. J.*, 37:318–332.
- Tyler, S.W., and Wheatcraft, S.W., 1992. Fractal scaling of soil particle-size distribution analysis and limitations. *Soil Science Society of America Journal*, 56 (2): 47–67.
- Vallejo, L.E., 2001. Fractal assessment of the surface texture of pavements. *Intern. J. of Pavement Engineering*, 2(2): 149–156.
- Yeggoni, M., Button, J.W., and Zollinger, D.G., 1996. Fractals of aggregates correlated with creep in asphalt concrete. *J. of Transp. Eng., ASCE*, 122(1): 22–27.

Comparative analysis of compaction procedures of unbound traditional and non-conventional materials

M.Pasetto & N.Baldo

Construction and Transport Dept., University of Padova, Padova, Italy

Pavements Unbound—Dawson (ed.)

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ABSTRACT: The results are presented of a laboratory experiment, conducted according to the prEN 13286–2 standards, on the physical and mechanical characterisation of unbound granular mixes, comprising a traditional quarried limestone aggregate and an aggregate originating from construction and demolition work. The objective was to check the applicability of the regulations to all types of materials, including natural or artificial “soft” ones (in the case in point an alternative aggregate), verifying any particle size variations at the post-compaction stage. In addition to the Proctor method, a gyratory compactor was used to make the samples, which is not customary in Italy in the formulation of unbound mixes, but which can better express the behaviour of the tested materials in. The gyratory compactor also allowed useful information to be gained on the shearing strength and density achieved by the different tested materials during compaction, demonstrating that the traditional procedures (Proctor) could have selective applicability depending on the mix being studied.

1 INTRODUCTION

In the study of granular unbound mixtures to be used in the sub-bases and sub-grades of road infrastructures, the Proctor compaction method is the one most widely used in road laboratories, by virtue of its simplicity and the lack of bulky equipment required, meaning that speedy inspection tests can be done using portable tools which can also be taken on site. The method has also been well tested and encoded in different Italian and international standards, and has recently been adopted in the proposal of regulation prEN 13286–2 formulated by CEN.

However, the Proctor method also has two drawbacks: the first is that the “impulsive” compaction characteristic does not simulate well the compaction done when the granular mix is being laid; the second is that the compaction methods may cause post-compaction particle-size variations, which behave differently according to the material used.

In bituminous mix formulations the impulsive Marshall technique is contrasted by the gyratory one, which can reproduce a more similar compaction to the real in situ compaction using rollers. Similarly, for aggregate compaction, it would appear reasonable to use the gyratory compactor with, or as an alternative to, the Proctor test. A comparative trial was therefore set up between the Proctor test and an un-conventional compaction methodology based on the use of a gyratory compactor, considering unbound granular mixes produced with both natural and recycled aggregate.

The latter type of aggregate, characterised by the presence of “soft” material (fragments of bricks, ceramics, etc.), should allow a critical element of the Proctor test to be verified, i.e., the particle-size variation, due to the crushing of the aggregate, which occurs during compaction of the solids.

This work forms part of a wider context of theoretical, applied and regulatory studies being done at the University of Padova to learn more about the characteristics of recycled materials from building construction and demolition work (C&D) and their possible applications in the building of roads, railways and airports.

2 MATERIALS USED

The investigation considered both natural quarried aggregate (Type 1 aggregate), and recycled material from C&D (Type 2 aggregate). Obviously, with the latter material all foreign elements, such as metals, plastic residues, glass, etc. were eliminated prior to testing.

Grading analysis, performed dry, provided the grading curves reported in Figure 1.

With reference to regulation UNI 10006 (which uses the HRB-AASHTO classification), Table 1 reports the values of the elements that allowed both materials to be classified as A1-a. The aggregates examined therefore have the requisites for the construction of road embankments and subgrades according to the above specification according to the above specification.

A further indication of the good quality of the materials used in the tests is provided by the coefficient of uniformity U , usually considered in the geotechnical sector, which has values of 42 and 43 for the Type 1 and Type 2 mixtures, respectively. Therefore, based on the classification reported in Table 2, it can be confirmed that these are well-graded materials.

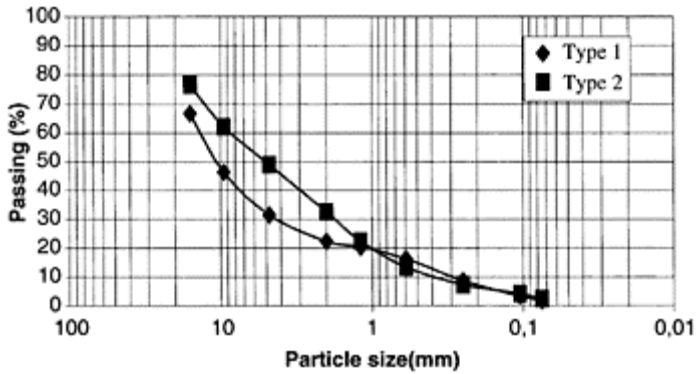


Figure 1. Grading curve of Type 1 and Type 2 mixtures.

Table 1. Properties of Type 1 and Type 2 aggregate.

Elements of classification—UNI 10006	Aggregate Type 1	Aggregate Type 2
% Passing through 10 ASTM sieve (2.00 mm)	22.3%	32.6%
% Passing through 40 ASTM sieve (0.42 mm)	13.4%	10.8%
% Passing through 200 ASTM sieve (0.075 mm)	1.8%	2.5%
Liquid limit W_L	—	—
Plastic limit w_p	—	—
Plasticity index I_p	0	
Group I_g index	0	

Table 2. Classification of the material, based on the coefficient of uniformity.

Classification of the material	Coefficient of uniformity U
Material practically uniform	$1 < U \leq 2$
Material not well graded	$2 < U \leq 6$
Material well graded	$6 < U \leq 15$
Material very well graded	$U > 15$

3 PROCTOR COMPACTION

The Type 1 mixture, compacted using the Proctor technique following the procedures reported in regulation prEN 13286–2, provided very well-defined results, with the

optimum water content (5.0%), and corresponding dry density (23.77 kN/m^3) being aided by the good particle-size mix and the high quality of the material used.

The Type 2 aggregate did not have the same characteristics of linearity in the curve of compaction envelope, and it was only possible to determine the optimum water content (10.0%) and corresponding dry density (20.48 kN/m^3) after many tests. This can be ascribed to the inclusion of brick and concrete fragments in the mixture, which, being highly porous, absorb a great deal of water.

For the recycled material to reach the maximum density level it was therefore necessary to use exactly twice the amount of water required for the conventional material.

Figures 2 and 3 report the Proctor compaction curves.

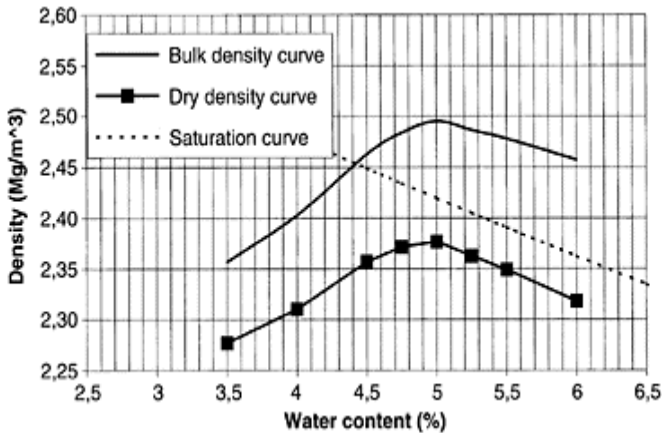


Figure 2. Proctor compaction curves of Type 1 mixture.

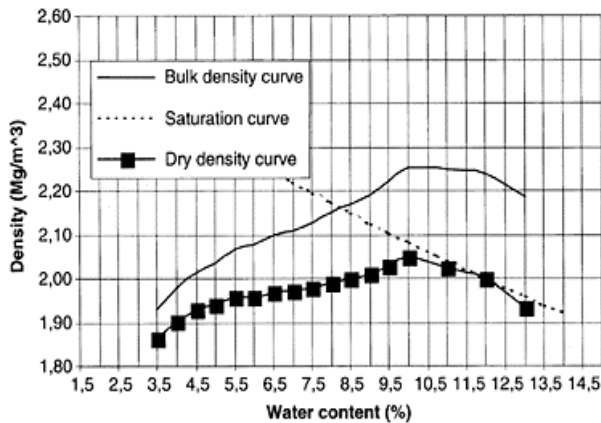


Figure 3. Proctor compaction curves of Type 2 mixture.

4 GYRATORY COMPACTION

Table 3 reports the values of the main operating parameters of the gyratory compaction equipment. In detail, relating to the vertical pressure on the specimen and the number of revolutions, values were considered that would allow comparison between the compaction energy supplied to the mixture during the gyratory compaction process with that of AASHTO standard compaction. As there are no reference regulations, similar experiments by other authors (Dondi et al., 2000) were considered, who identified a pressure of 600 kPa and 130 revs as being ideal to ensure a density level comparable with the Proctor one.

It was then verified whether the drained conditions characteristic of Proctor compaction, would also be guaranteed by gyratory compaction. For this it was unnecessary to use any particular operating expedient, given that the mould and base plates, while guaranteeing the confinement of the material, do not constitute a sealing system and therefore allow drainage.

Figures 4 and 5 report the curves relative to the determination of optimum moisture content (4.75% and 9%) and the corresponding bulk density (25.61 kN/m³ and 22.85 kN/m³) for Type 1 and Type 2 materials, respectively.

Regarding the results obtained, Figures 6, 7, 8 and 9 give the curves of compaction and resistance to shearing of the Type 1 and Type 2 aggregates, at different numbers of revolutions and for different levels of moisture content, obtainable thanks to the possibility offered by the gyratory compactor to continuously monitor the compaction process.

For less than optimum water contents (4% and 4.5% for Type 1, 7% and 8% for Type 2) the curves of compaction are almost parallel to the curves at $\omega = \omega_{opt}$, but shifted towards lower density levels. This behaviour means that when the water content is low, there is a great deal of attrition between the particles that hinders efficient compaction of the material. In fact, at $\omega < \omega_{opt}$, resistance to shearing increases progressively until the end of the compaction, denoting a poor capacity of the mixes to achieve a state of maximum density.

Table 3. Gyratory compaction equipment capability.

Angle of rotation	1.25° ± 0.02°
Rotation speed	30 rpm
Number of revolutions	130
Diameter of the mould	150 mm
Vertical pressure	600 kPa
(constant during compaction)	

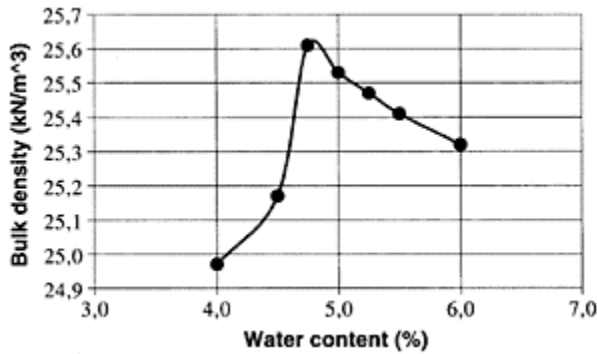


Figure 4. Type 1 mixture: optimum moisture content by gyratory compaction.

Increasing water content (5% and 5.25% for Type 1 mixture, 9.5% and 10% for Type 2 mixture), reduced suction allowing easier compaction than when suction is higher and hinders particle movements; at the same energy used in the compaction, it leads to a better density. In fact, the curves of compaction for these water contents are again similar to the preceding curves, but shifted towards higher density values. In this case resistance to shearing increases until a peak is

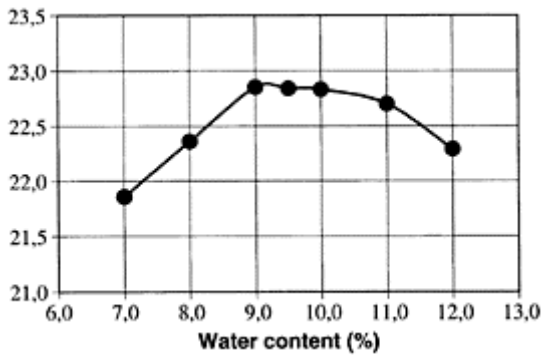


Figure 5. Type 2 mixture: optimum moisture content by gyratory compaction.

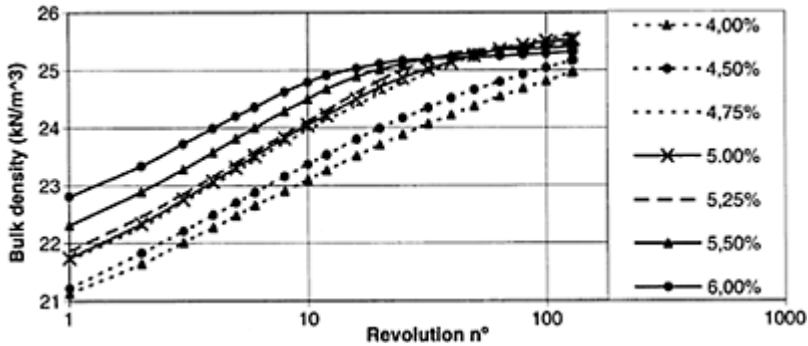


Figure 6. Type 1 mixture: density vs. number of revolutions of gyratory compactor.

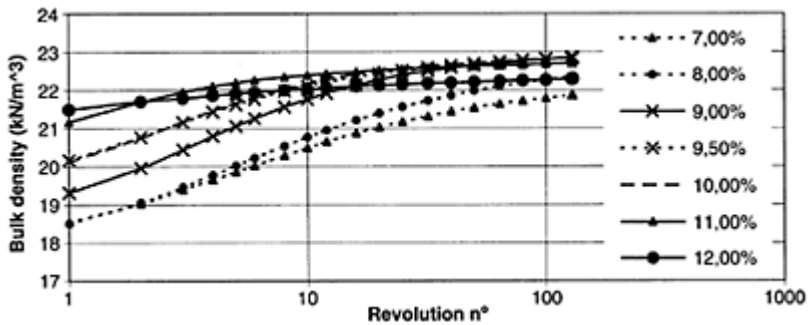


Figure 7. Type 2 mixture: density vs. number of revolutions of gyratory compactor.

reached, past which it diminishes somewhat. This behaviour represents the best compromise solution between good density and sufficient internal friction, so as not to cause in situ liquefaction phenomena in the mixtures.

In terms of density and resistance to shearing, the performances of the mixes with a higher than optimum water content are comparable with those at $\omega = \omega_{opt}$, up to a 0.25% and 0.5% greater moisture content for Type 1 and Type 2 aggregate, respectively. This suggests that during the laying on site, there is a certain safety margin as regards the use of higher water contents than the optimum determined in the laboratory.

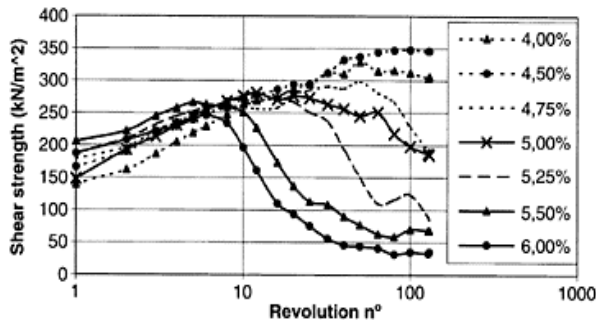


Figure 8. Type 1 mixture: shear strength vs. number of revolutions of gyratory compactor.

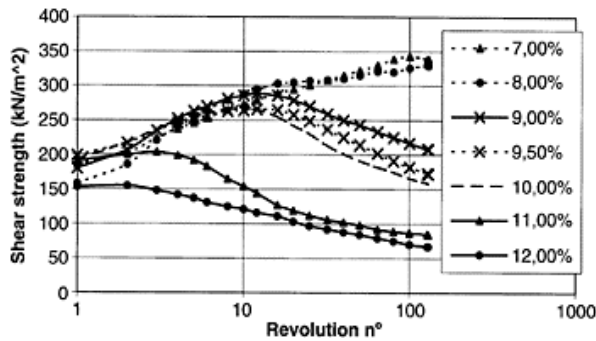


Figure 9. Type 2 mixture: shear strength vs. number of revolutions of gyratory compactor.

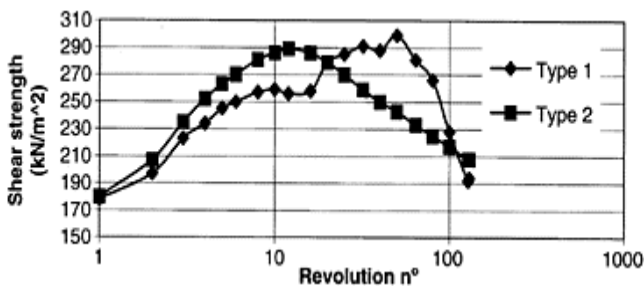


Figure 10. Type 1 and Type 2 mixture: shear strength vs. number of revolutions of gyratory compactor.

When the water content is further increased (5.5% and 6% for Type 1 mixture, 11% and 12% for Type 2 mixture), clear and hazardous negative effects appear, as the water fills the voids between the particles and reduces the space available for air. This must be compacted at increasing pressure to be able to obtain the same density level as that reached with less water. Otherwise, given that the energy used remains the same, as the water content increases, above a limit of circa 50 revs, there is a small reduction in the density of the material. In terms of shear strength, the peak reached after a few revolutions of compaction is followed by a sharp fall of internal friction within the compacted material, which could lead to phenomena of decompaction on site.

It should be pointed out that, although the optimum moisture contents are very different for the two materials, the maximum and final resistances to shearing are similar: 299 and 194 kN/m² for Type 1 aggregate, 289 and 208 kN/m² for Type 2 aggregate (Figure 10).

As well as water content, the moment of reaching maximum shear strength also distinguishes the two materials; 50 revolutions for the Type 1 mixture, just 12 for the Type 2 mixture. The different behaviour of the second mixture is due to the fact that the specimen more quickly becomes a series of individual stones “floating” in a matrix of fines.

5 COMPARATIVE ANALYSIS

Figure 11 gives the comparison between the bulk density levels reached by the two materials at $\omega=\omega_{opt}$, with the Proctor and gyratory compaction. It shows that a higher final compaction level was obtained with the gyratory compactor than the Proctor test: 2.65% and 1.42% for Types 1 and 2, respectively. In addition, with the gyratory compactor, density levels similar to the Proctor ones can be reached after only 32 revolutions for Type 1 mixture (24.95 kN/m³) and 35 for Type 2 aggregate (22.53 kN/m³).

The Type 1 material reaches somewhat higher compaction levels than Type 2.

The pressure values and number of revolutions chosen for compaction with the gyratory technique were therefore more than adequate.

Regarding the determination of optimum water content, the two methods provide almost identical results for Type 1 (0.25% deviation), while there is a quite clear difference for Type 2 mixture (1% deviation—c.f. Figure 2 with Figure 4 and c.f. Figure 3 with Figure 5).

The post-compaction particle-size variations were studied considering the samples produced with optimal water content. Tables 4 and 5 report the values of passing through the different sieves,

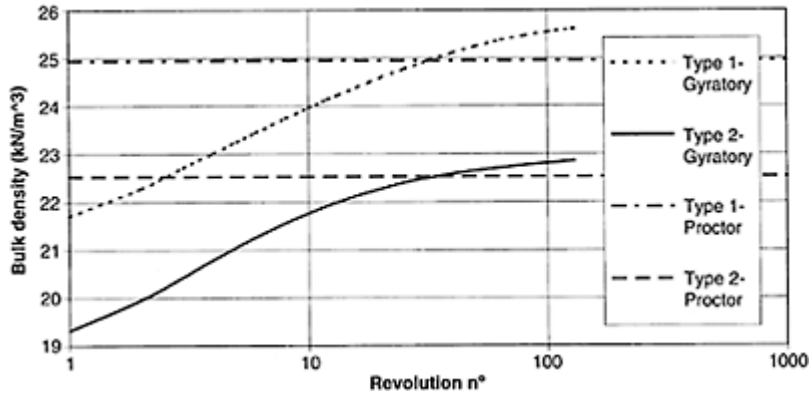


Figure 11. Type 1 & 2 mixtures: comparison of density level by gyratory and Proctor compaction.

Table 4. Type 1 mixture: sieve passing, before and after the Proctor and gyratory compaction.

Sieves [mm]	Initial passing [%]	Post Proctor passing [%]	Post gyratory passing [%]	Post Proctor variation [%]	Post gyratory variation [%]	Δ Proctor to gyratory [%]
15.9	66.6	77.3	68.3	10.7	1.7	9
9.51	46.2	54.7	48.9	8.5	2.7	5.8
4.76	31.4	37.6	33.7	6.2	2.3	3.9
2.00	22.3	28.4	25.9	6.1	3.6	2.5
1.19	20.4	26.3	24.1	5.9	3.7	2.2
0.595	16.4	24.2	22.2	7.8	5.8	2
0.25	8.8	16.4	16.8	7.6	8	-0.4
0.105	3.4	11.6	12.1	8.2	8.7	-0.5
0.075	1.8	9.7	10.1	7.9	8.3	-0.4

Table 5. Type 2 mixture: sieve passing, before and after the Proctor and gyratory compaction.

Sieves [mm]	Initial passing [%]	Post Proctor passing [%]	Post gyratory passing [%]	Post Proctor variation [%]	Post gyratory variation [%]	Δ Proctor to gyratory [%]

15.9	76.5	88.0	80.1	11.5	3.6	7.9
9.51	62.1	71.1	68.3	9	6.2	2.8
4.76	48.9	59.5	57.6	10.6	8.7	1.9
2.00	32.6	50.2	48.3	17.6	15.7	1.9
1.19	22.4	46.8	45.1	24.4	22.7	1.7
0.595	13.4	43.0	41.3	29.6	27.9	1.7
0.25	7.4	39.0	37.5	31.6	30.1	1.5
0.105	4.3	35.7	34.2	31.4	29.9	1.5
0.075	2.5	33.9	32.7	31.4	30.2	1.2

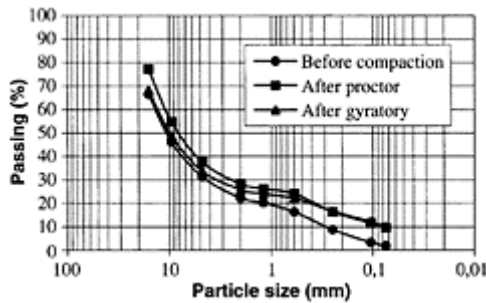


Figure 12. Type 1 mixture: grading curve, before and after the Proctor and gyratory compaction.

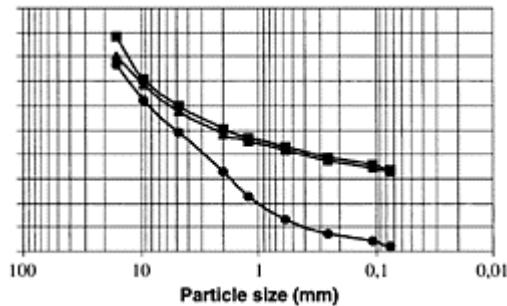


Figure 13. Type 2 mixture: grading curve, before and after the Proctor and gyratory compaction.

before and after the Proctor and gyratory compaction, for Type 1 and Type 2 aggregate, respectively. Figures 12 and 13 show the relative grading curves.

There are two important elements:

- the recycled material is affected by the larger particle size variations compared to the natural material, certainly caused by the crushing of the brick and concrete fragments present in the Type 2 mix;
- the differences in grading variations were not great between the two compaction methods, and in fact for Type 1 mixture, when considering to the smallest particle diameters, the gyratory caused higher variation than the Proctor.

6 CONCLUSIONS

In the determination of ω_{opt} , the two analysed methods (gyratory and Proctor compaction) provided substantially similar results for the natural aggregate. For the C&D inerts the gyratory compactor gave a much higher value of optimum mixture content. The validity of this result can be confirmed by the information relating to the pattern of internal friction during compaction, which the gyratory compactor allows to be monitored continually. Unlike the Proctor technique, the gyratory method also allows any critical performances of the mixes to be displayed.

The feared post-compaction particle-size variation was marked for the “soft” material, but the two compaction methods did not lead to important differences.

The experiments have demonstrated the usefulness of the gyratory compactor, originally used for bituminous concrete, in the study of unbound granular mixes, in particular non-conventional materials, while both methods have been demonstrated as substantially equivalent for traditional applications.

REFERENCES

- AIPCR/PIARC, 1989, “*Marginal Materials. State of the art*”, Paris.
- C.N.R. B.U. (standard) no. 23, 1971, “*Analisi granulometrica di una terra mediante crivelli e setacci*” (*Grading analysis of soil by sieving*).
- C.N.R. B.U. (standard) no. 64, 1978, “*Determinazione della massa volumica reale dei granuli di un aggregato*” (*Determination of the true volumetric mass of an aggregate*).
- C.N.R. B.U. (standard) no. 69, 1978, “*Norme sui materiali stradali; Prova di costipamento di una terra*” (*Road Material Standard: Compaction test for soil*).
- G.Dondi, A.Simone & A.Bonini, 2000, “*Metodologie di impiego della press giratoria (2^a parte)*” (*Methodology for use of the gyratory compactor—Part 2*), *Rassegna Del Bitume*, no. 35, SITEB, Rome, pp. 39–48.
- EN 13242 Standard, 2003, “*Aggregates for unbound and hydraulically bound materials for use in civil engineering work and road construction*”.
- prEN 13285 Standard, “*Unbound mixtures—Specification*”.
- prEN 13286–1 Standard, “*Unbound and hydraulically bound mixtures—Part 1: Test method for the determination of the laboratory reference density and water content – Introduction, general requirements and sampling*”.
- prEN 13286–2 Standard, Last version, “*Unbound and hydraulically bound mixtures – Part 2: Test method for the determination of the laboratory reference density and water content—Proctor compaction*”.

- UNI 10006 Standard, 2002, “*Costruzione e manutenzione delle strade—Tecniche di impiego delle terre*” (*Construction and maintenance of roads—Techniques for use of soil*).
- UNI EN 933–1 Standard, 1999, “*Prove per determinare le caratteristiche geometriche degli aggregati—Determinazione della distribuzione granulometrica; Analisi granulometrica per setacciatura*” (*Tests to determine the geometric characteristics of aggregates – grading determination by sieving*).
- UNI EN 933–2 Standard, 1997, “*Prove per determinare le caratteristiche geometriche degli aggregati—Determinazione della distribuzione granulometrica; Stacci di controllo, dimensioni nominali delle aperture*” (*Tests to determine the geometric characteristics of aggregates—grading determination—Nominal dimension of openings of control sieves*).
- UNI EN 1097–5 Standard, 2000, “*Prove per determinare le proprietà meccaniche e fisiche degli aggregati; Determinazione del contenuto d’acqua per essiccazione in forno ventilato*” (*Determination of the moisture content by desiccation in a ventilated oven*).

Cyclic plasticity based model for flexible pavements

C.Chazallon & F.Allou

*Laboratory of Mechanical Modelling of Materials and Structures of Civil Engineering,
Université de Limoges, France*

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ABSTRACT: Rutting is one of the main causes of damage in low traffic flexible pavements and recent studies, dealing with the improvement of design methods for flexible pavements, have pointed out the key role played by permanent deformations in the unbound granular layers. In spite of this, adequate methods for predicting permanent deformations in unbound pavement materials are lacking. The purpose of this paper is to show that the monotonic and cyclic mechanical behaviour of unbound granular materials for roads can be modelled within the framework of soils plasticity. The incremental method is a step by step calculation based on an elastoplastic model, and a simplified method is developed for finite element (2D and 3D) modelling of flexible pavements.

1 INTRODUCTION

Rutting is one of the main causes of damage in low traffic flexible pavements and recent studies, dealing with the improvement of design methods for flexible pavements, have pointed out the key role played by permanent deformations in the unbound granular layers. In spite of this, adequate methods for predicting permanent deformations in unbound pavement materials are lacking. In France, pavement design is based on linear elastic calculations, and to limit the risk of rutting, a strain criterion, limiting the vertical elastic strain is used. On the contrary, in soil mechanics, the plasticity framework is widely used, and good results have been obtained in the modelling of the cyclic behaviour of soils submitted to earthquakes (Arulanandan et al (1993)). However, the mechanical behaviour of unbound granular materials has to be described for up to 105 to 106 cycles, with ratchetting and elastic or plastic shakedown, leading to rutting of the pavement, whereas in earthquake problems, the accelerogram represents less than 100 load cycles.

Three methods can be used to take into account the mechanical behaviour of unbound granular materials for roads. The first one is the incremental method. It is based on step by step calculation with an elastoplastic model. Interpolation laws coupled with incremental calculations simplify the calculation of the plastic strains under large numbers of cycles. This method developed by Bonaquist et al (1997) is presently studied by Desai (2002). We have used results obtained by Chazallon et al (2002) comparing experiments and calculations for triaxial loadings on a clayey sand, and a simplified method for finite element calculation of flexible pavement have been developed. The second method is based on analytical models (Barksdale (1972)), (Paute et al (1988)), (Lekarp et al (1998)), which can be coupled with Boussinesq theory (de Buhan et al (2002)). Finally, the third method is based on simplified methods based on the shakedown theory (Sharp et al (1984)), (Yu et al (1998)), (Habiballah et al (2003)). Advantages and drawbacks of the first method will be presented in this paper.

2 ELASTOPLASTIC MODEL

2.1 Resilient behaviour of Unbound Granular Materials (UGM)

These materials exhibit a complex non-linear response under repeated loading. The resilient response of granular materials is usually defined by resilient modulus and Poisson's ratio or shear and bulk modules. Two main models are used for modelling the resilient behaviour: the K- θ model (Hicks et al (1972)) and the Boyce model (Boyce (1980)). Later, Hornyh et al (1998) proposed an anisotropic version of Boyce model. The expression of Boyce model was modified with the multiplication of the principal stress σ_1 by a coefficient of anisotropy γ . The model equations are given by the stress-strain relationships, as:

$$\varepsilon_v = \frac{p^*n}{p_a^{n-1}} \left[\frac{\gamma+2}{3Ka} + \frac{n-1}{18Ga} (\gamma+2) \left(\frac{q^*}{p^*} \right)^2 + \frac{\gamma-1}{3Ga} \left(\frac{q^*}{p^*} \right) \right] \quad (1)$$

$$\varepsilon_q = \frac{2}{3} \frac{p^*n}{p_a^{n-1}} \left[\frac{\gamma-1}{3Ka} + \frac{n-1}{18Ga} (\gamma-1) \left(\frac{q^*}{p^*} \right)^2 + \frac{2\gamma+1}{6Ga} \left(\frac{q^*}{p^*} \right) \right] \quad (2)$$

with:

$$p^* = \frac{\gamma\sigma_1 + 2\sigma_3}{3}, \quad q^* = \gamma\sigma_1 - \sigma_3 \quad (3)$$

See the section 5 for the notations. This model is used as the elastic part of the elastoplastic model.

2.2 Permanent deformation behaviour of UGM

In comparison with resilient behaviour, less research has been devoted to permanent deformation. One reason is that it requires much more time for experiments, each stress path requires one new specimen, and the number of cycles is large. The main objective of

research in long term behaviour study is to establish a constitutive relationship of the amount of permanent strain at any number of cycle and at any stress level. Consequently, empirical and rheological models have been developed.

Empirical models are based on repeated load triaxial test results. Several workers have attempted to relate the permanent deformation with the number of cycles and/or the stress level (COST337 (2003)).

In soil mechanics, elastoplastic models have been developed to describe the behaviour of UGM which are used in road pavements. These models describe the behaviour of UGM under monotonic loading but they require modifications to reproduce the behaviour under large numbers of cycles. Richer et al (1999) and Chazallon (2000) developments are based on Hujeux's model (Hujeux (1985)) and the critical state concept (Schoffield et al (1968)). Mayoraz (2002), employed an analogy between the evolution of permanent strain in the cyclic load triaxial test and the evolution of permanent strain of viscous materials submitted to creep. Simplified analyses have been developed either with the plasticity framework for finite element modelling of pavements (Bonaquist et al (1997)), (Desai (2002)) or with an analytical framework (de Buhan et al (2002)). We have to underline that these models have never been used for long term behaviour pavement modelling.

Recently, some authors apply the shakedown theory to study the long term pavements behaviour (Sharp et al (1984)), (Yu et al (1998)), (Habibalah et al (2003)).

2.3 The elastoplastic constitutive model

The model has been presented in Chazallon (2000), and some modifications have been added. To take into account the unsaturated state of the material, leading to a macroscopic cohesion, a constitutive parameter C_0 has been added. It appears in the expression of the yield surface, plastic potential, the kinetic laws and the elasticity law, by adding C_0 to the mean stress p to obtain p^* . The elasticity is considered non linear and the author has used the anisotropic hyperelastic formulation proposed by Hornych et al (1998) (equation (1)). The yield function f is written:

$$f = \left(\frac{27}{2}\right)^{1/2} \frac{S_0(\bar{\sigma} - \bar{X})}{M I_1(\bar{\sigma} - \bar{X})} + r_c b \ln \left[\frac{I_1(\bar{\sigma} - \bar{X})}{3 p_c^*} \right] \quad (4)$$

where b is a parameter which controls the shape of the yield surface in the (p, q) plane. M is the slope of the critical state line in the (p, q) plane. See the Section 5 for the notations, $\varepsilon_{v_i}^p$ is the critical pressure corresponding to the actual void ratio. The hardening is given by:

$$p_c = p_{c0} \exp(\beta \varepsilon_{v_i}^p) \quad (5)$$

where $\varepsilon_{v_i}^p$ is the volumetric plastic strain. p_{c0} and β are constants which determine the position of the critical state line in the $(e, \ln(p))$ plane. p_{c0} is the initial critical pressure corresponding to the initial critical void ratio e_{c0} .

r_c is a hardening variable associated to the deviatoric plastic strain ε_d^p .

$$\mathbf{r}_c = \frac{\mathbf{e}_d^p}{\mathbf{a} + \mathbf{e}_d^p} + \mathbf{r}^{el} \text{ and } d\mathbf{r}_c = d\lambda \cdot 3(1 - r_c)^2 / a M I_1(\tilde{\sigma} - \tilde{X}) \quad (6)$$

Initially, under monotonic loading $\mathbf{r}_c = \mathbf{r}_m^{el}$, where $\mathbf{r}_m^{el}$ represents the initial elastic domain, ($1 \geq r_c$) and $a = a_m$ governs the evolution of the hardening variable $\mathbf{r}_c$. When unloading occurs, $\mathbf{r}_c = \mathbf{r}_c^{el}$ represents the initial elastic domain and $a = a_c$ governs the evolution of the hardening variable $\mathbf{r}_c$. $d\lambda$ is the plastic multiplier, and it can be determined by the consistency condition $df=0$ and $d\lambda \geq 0$. The former equations are completed by the definition of a non associated plastic potential g and the following kinetic:

$$d\tilde{\epsilon}^p = d\lambda \frac{\partial g(\tilde{\sigma} - \tilde{X})}{\partial \tilde{\sigma}} \text{ and } g = \left(\frac{27}{2}\right)^{1/2} S_{II}(\tilde{\sigma} - \tilde{X}) / M I_1(\tilde{\sigma} - \tilde{X}) + \ln \left[\frac{I_1(\tilde{\sigma} - \tilde{X})}{3p_c^*} \right] \quad (7)$$

$\tilde{X}$ is the tensorial kinematic hardening variable. Its kinetic is given by

$$d\tilde{X} = \mu d\tilde{\sigma} \text{ and } 0 \leq \mu \leq 1 \quad (8)$$

If one follows a stress path AB from A to B, and one unloads from B to C, $\tilde{X}$ is the stress of the origin of the new plane (p, q).

$$\tilde{X} = \tilde{\sigma}_B + P_{uc}/P_c^* P_c^* \tilde{I} \text{ and } 0 \leq P_{uc}/P_c^* \leq 1 \quad (9)$$

where $\tilde{I}$ is the second order identity tensor, P_{uc} and P_{lc} are two parameters which take into account the position of the yield surface and the plastic potential during unloading (subscript uc) or reloading (subscript lc). The parameters of the model are divided into monotonic and cyclic and are uncoupled. The parameters $(\mu, \mathbf{r}_m^{el})$ have been fixed and $a_c = a_m$.

2.4 Parameters identification

The parameters identification, on a clayey sand (sand of Miscillac) of 0/4 millimeters grading, has been presented in Chazallon et al (2002). The values of the parameters of the Missillac sand are presented in Table 1. The elasticity parameters (K_a , G_a , n) and the anisotropic parameter γ are determined using the results of cyclic triaxial tests performed at various (q/p) ratios (Batard et al (2002)). Then, monotonic triaxial shear tests are performed till failure at different initial confining pressures: 0 kPa, 10 kPa and 20 kPa, and C_0 , M are determined. The initial critical pressure p_{c0} and the plastic compressibility modulus β have been determined using correlations (Hicher et al (1994)).

Table 1. Model parameters (Chazallon et al (2002)).

Elasticity parameters		Plasticity	parameters
n	0.55	β	50
Ka(MPa)	25	$a_m(10^{-3})$	4.2

Ga(Mpa)	30.7	b	0.25
γ	0.64	r_m^{el}	0.02
<i>Critical state parameters</i>		$P_{uc}(10^{-3})$	2
C_0 (kPa)	15.5	$P_{lc}(10^{-3})$	1.5
M	1.7	r_c^{el}	0.48
P_{c0} (Mpa)	10		

The hardening parameters a , b have been determined using the results of monotonic triaxial tests. Then, the cyclic parameters (P_{uc} , P_{lc} , r_c^{el}) are determined using cyclic triaxial tests.

3 PAVEMENT MODELLING

The object of this study is to develop a simplified calculation to estimate the permanent strains of unbound granular materials layers subjected to large cycles numbers. The following method has been used:

- The estimation of the stress field in a flexible pavement structure with the anisotropic Boyce model is performed. The asphalt layer was assumed to be linear elastic ($E=5400$ Mpa, 0.35). The granular layer and subgrade are considered non linear. The anisotropic Boyce model is used with the values of material properties given in Table 1, elasticity parameters line.
- Then the calculation of the accumulated plastic strains under repeated loading (80 000 cycles) is performed with the proposed elastoplastic model.
- Finally, the integration of the curve (depth—plastic strains) is calculated, which corresponds to the plastic deflexion.

3.1 Pavement modelling

Our study deals with the feasibility of 2D and 3D finite element modelling of pavements when one studies the long term pavement behaviour. Consequently, the same materials have been used for the base layer and the subgrade.

- 2D finite element modelling: the pavement, for this study, consists of a 0.12 m thick asphalt layer as the surfacing course, a 0.40 m thick granular layer (Missillac sand) as the base course, a 0.270 m thick soil layer (Missillac sand) as the subgrade. The pavement configuration is shown in Figure 1.b with axisymmetric conditions.
- 3D finite element modelling: the pavement block of 3.22 m thickness is shown in Figure 2. A quarter of the pavement has been modelled and 4 meters in the longitudinal direction have been taken into account.

3.2 Loading

The loading is the cyclic load plate test which represents the French standard single axle load (dual wheel half-axle loaded at 65 kN having an inflation pressure of 0.56 MPa). Pavement-tyre contact stresses were considered to be uniform and applied near the centre.

- 2D finite element modelling: the contact pressure is applied on a radius whose length is equal to 0.192 m (under axisymmetric conditions)
- 3D finite element modelling: the contact pressure is applied on an area of 0.192 m $\times$ 0.192 m which represents the quarter of the dual wheel contact surface.

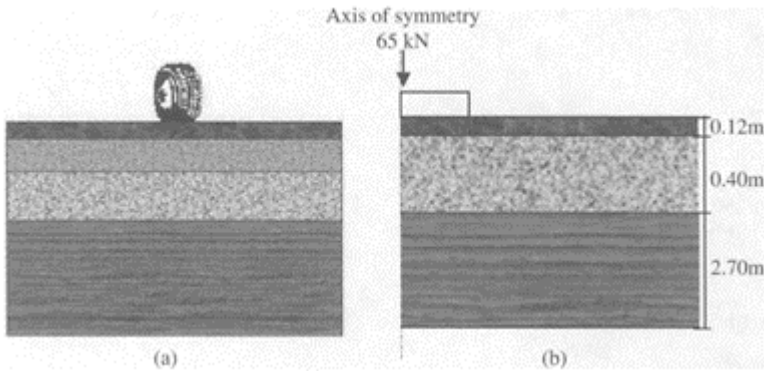


Figure 1. Pavement in 2D analysis.

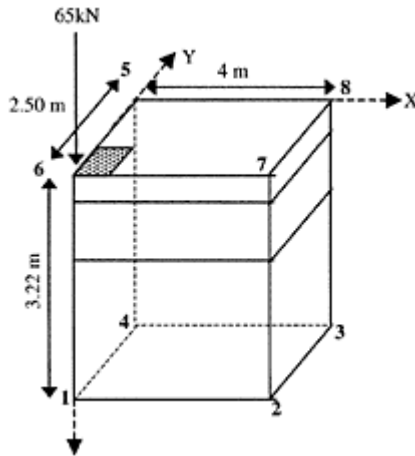


Figure 2. Pavement in 3D analysis.

3.3 The finite element mesh

The pavement modelling was performed using the finite element program CAST3M.

- 2D finite element analysis with axisymmetric idealisation is adopted. The finite elements used are the quadrilateral plane elements with 9 nodes.
- the 3D mesh used for the analysis represents the quarter of the dual wheel, half axle loaded. For the discretization of the geometrical model, we used hexahedral tridimensional finite elements. The total number of elements was 9000.

3.4 Boundary conditions

The following conditions are applied, for 2D and 3D analyses.

- 2D finite element modelling: along the line of symmetry, all nodes are constrained horizontally, but are free to move in the vertical direction. The bottom of the subgrade layer is fixed.
- 3D finite element modelling: with reference to Figure 2, the bottom surface (plane defined by the points 1–2–3–4) is fixed. The displacements according to the x axis of the plane defined by the points 1–4–5–6 and to the y axis of the plane defined by the points 1–2–7–6 are prevented.

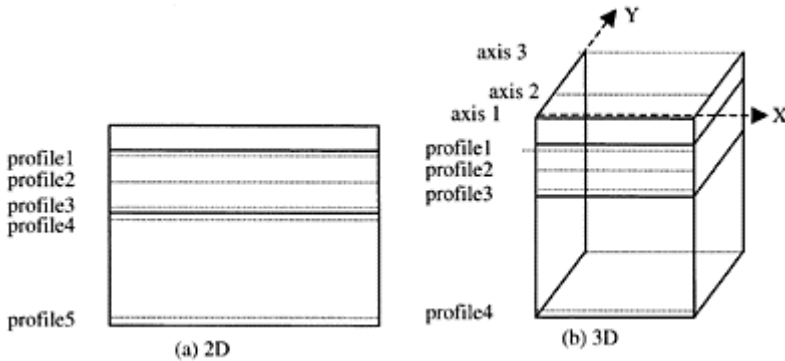


Figure 3. Position of the stress paths.

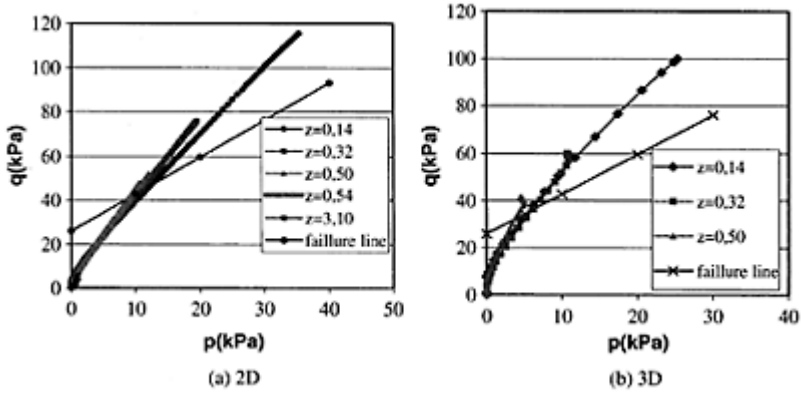


Figure 4. Stress paths.

3.5 Results

Figures 3(a) and (b) show the stress paths obtained along the direction of wheel movement (x axis) in 2D analysis, at 0.14, 0.32, 0.50, 0.54, 3.10 m depths; along the direction of the wheel movement (x axis) and the direction perpendicular to the wheel movement (y axis) in 3D analysis, at 0.14, 0.32, 0.50, 3.10 m depths.

Figures 4(a) and (b) represent the stress paths $q=f(p)$ (p : pressure, q : deviatoric stress). The stress paths obtained for both analyses show differences of 20% for the deviatoric stress. However, such a difference may be acceptable, because the 3D analysis consumes more time and effort. It can be seen that the stress paths are fairly linear, with a slope close to 3.

Figures 5(a) and (b) show the permanent strain calculated with the simplified method for 2D and 3D analyses. We recall that 80 000 cycles have been performed for each stress path.

In 3D analysis, the permanent strains are presented in the (ϵ_z^p, z) plane, at three axis positions in the direction y (direction perpendicular to the wheel movement). One can see that the permanent strain decreases when one considers axis 1, axis 2, axis 3. We have to underline that the model can not estimate the permanent strain when the stress path is too far from the failure line, thus, for those stress paths in 2D and 3D analyses, the permanent strains have been fixed at 10%. The permanent strains for both analyses do not show significant difference.

Plastic deflexions, computed at 0.50 m and 3.10 m depths by a trapezoidal method for both 2D and 3D analyses, are presented graphically in Figure 6. In 2D analysis, we obtain just one point but

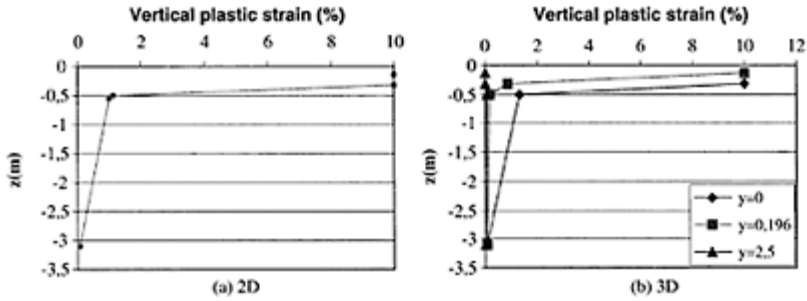


Figure 5. Vertical plastic strain.

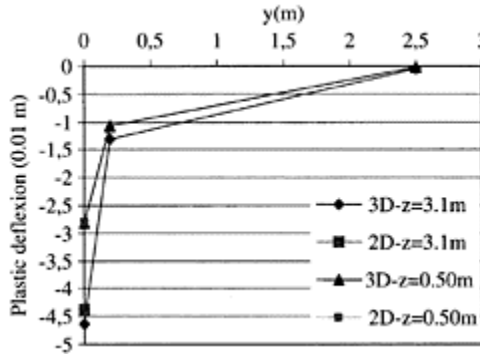


Figure 6. Plastic deflexions.

in 3D analysis, we obtain the bowl of plastic deflexion. The plastic deflexions for both analyses do not show significant differences. One can see that the deflexion under the load application is important and decreases when the load is remote from the application point, nevertheless the influence zone of the plastic deflexion bowl exists till 2.5 m. One can see that the maximum plastic deflexion contribution for $z=0.50\text{ m}$ represents 40% and 42% respectively for the 2D and 3D cases.

4 CONCLUSION

This simplified incremental method is based on an elastoplastic model with an anisotropic hyperelastic part and with kinematic hardening. It requires parameters, which must be determined using monotonic triaxial tests till failure and cyclic triaxial tests with a resilient behaviour study and a permanent deformation study. The calculation is performed with two steps. First an elastic calculation is performed to initialise the stress field with the anisotropic hyperelastic model, then stress paths are obtained at various depths. Two methods have been proposed for finite element modelling of the long term behaviour of flexible pavements. The results show that under cyclic load plate tests, 2D and 3D analyses give nearly the same results if we are interested in the maximum plastic

deflexion, but the 3D analysis gives the plastic deflexion bowl and its influence zone which exists till 2.5 m. This method is less expensive and time consuming than the traditional incremental calculations. Thus, calculation time remain acceptable for the large numbers of load cycles encountered in pavements. Now, this method has to be tested on full scale instrumented experiments and validated using results of monotonic and cyclic triaxial test performed on materials in the same mechanical state and under various stress paths ($1 \leq q/p \leq 3$).

5 NOTATION

$\varepsilon_v, \varepsilon_q$: volumetric strain and deviatoric strain respectively.

n, p_a : coefficient of non linear elasticity and reference pressure equal to 0.1 Mpa respectively.

K_a, G_a : bulk and shear moduli at the reference pressure.

$\tilde{\sigma}, \tilde{\chi}, \tilde{\varepsilon}^p$: stress tensor, kinematic hardening tensor and plastic strain tensor respectively.

I_I, S_{II} : trace operator and deviatoric stress operator respectively.

REFERENCES

- Arulanandan K., Scott R.F., 1993, *Proceeding of the International Conference on Verification of Numerical Procedures for the Analysis of soil Liquefaction Problem*, Vol 1. And Vol 2.
- Batard G., 2002, Etude du comportement des sols support de chaussées a l'essai triaxial a chargement répétés. *Mémoire de fin d'études*, INSA Rennes.
- Boyce H.R., 1980, "A non linear model for the elastic behaviour of granular materials under repeated loading", *Proceedings International Symposium on Soil under Cyclic and Transient Loading*, Swansea, UK, Vol 1, pp 285–294.
- Chazallon C., 2000, "An elastoplastic model with kinematics hardening for unbound aggregates in road", *Unbound Aggregates in Road Construction UNBAR5*, pp 265–270, Nottingham.
- Chazallon C., Habibalah T. and Hornych P., 2002, "Elastoplasticity framework for incremental or simplified methods for unbound granular material for roads", *6th International Conference on the Bearing Capacity of Roads, Railways, and Airfields*, pp 31–39, Lisbonne.
- COST 337 Final Report, "Unbound granular materials for road pavement", European Commission, 2003, pp 350.
- De Buhan P., Abdelkrim M. and Bonnet G., 2002, "A numerical method for predicting the residual settlement of rail-road track under repeated traffic loading", *Numerical Methods in Geotechnical Engineering*, Presses de l'ENPC-LCPC, pp 125–130, Paris.
- Desai C.S., 2002, "Mechanics pavement analysis and design using unified material and computer models", *Proceedings of the 3rd International Symposium on 3D Finite Element for Pavement Analysis, Design and research*, pp 1–63, Amsterdam.
- Habibalah T., Chazallon C. and Petit C., 2003, "Simplified method based on plasticity for the permanent strains of unbound granular materials of flexible pavements", *International Symposium on Deformation Characteristics of Geomaterials*, pp 343–348, Lyon.
- Richer P.Y., Rahma A., 1994, "Micro—Macro correlations for granular media. Application to the modelling of sands", *European Journal of Mechanics of Solids*, 13, 6, pp 763–781.

- Hicks, R.G., Monismith, C.L., 1972, "Prediction of the resilient response of pavement containing granular layers using non linear elastic theory", *International Conference on Asphalt Pavement*, 1, pp 410–429.
- Hornych P., Kazai A. and Piau J.M., 1998, "Study of the resilient behaviour materials", *Proc. 5th Int. Conf. On the Bearing Capacity of Road and Airfield*, pp 1277–1287, Trondheim.
- Lekarp F., Dawson A., 1998, "Modelling permanent deformation behaviour of UGM", *Const. And Building Mat.*, 12 (1), pp 9–18.
- Mayoraz F., 2002, "*Comportement mécanique des milieux granulaires sous sollicitations cycliques: Application aux fondations des chaussées souples*", These de Doctorat, EPFL.
- Paute J.L., Jouve P., Martinez J., and Ragneau E., 1988, "Modèle de calcul pour le dimensionnement des chaussées souples", *Bulletin de liaison des laboratoires des ponts et chaussées*, 156, pp 21–36.
- Schofield AN, Wroth CP., 1968, Critical state soil mechanics. McGraw-Hill.
- Sharp R., Booker J., 1984, "Shakedown of pavements under moving surface loads", *Journal of Transportation Engineering*, pp 1–14, n°1
- Yu H.S., Hossain M.Z., 1998, "Lower bound shakedown analysis of layered pavements discontinuous stress fields", *Computer Methods in Applied Mechanics and Engineering*, 167, pp 209–222.

Fundamental study on permanent deformation analysis of granular base course material using elasto-plastic model

Y.Takeuchi, M.Koyanagawa & T.Maki

Tokyo University of Agriculture, Japan
T.Nishizawa

Ishikawa National College of Technology, Japan
K.Endo

The Nippon Road Co., Ltd, Japan

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ABSTRACT: One of the authors had revised the modified Cam-Clay model using the extended SMP criterion and a hardening parameter, and shown that the static tri-axial compression test of the subgrade material, base course material and the open grade asphalt mixture can be simulate by that model. In this study, the further revised Cam-Clay model, which took in the rotational hardening concept proposed by Hashiguchi, was developed. And the simulation of the cyclic loading tri-axial compression test of a granular base course material was carried out. As the result, it was found that this model could be simulated the increase of the plastic strain with cyclic loading. Consequently, it seems that the elasto-plastic model developed in this study is effective for the permanent deformation analysis of the granular base course materials.

1 INTRODUCTION

Traffic loads causes the permanent deformation of base course and subgrade materials, resulting in severe rutting on asphalt pavement surface, and void formation under concrete pavement slabs, which accelerates fatigue cracking of the slabs [1]. Therefore, it is important to predict the permanent deformation of soil-based materials in the pavement design and evaluation.

One of the authors proposed a model by revising the modified Cam-Clay model using the extended SMP (Spatially Mobilized Plane) criterion which proposed by Matsuoka et al. [2] and a hardening parameter, and showed that the model is able to simulate the mechanical behaviors of compacted and normally consolidated clays for subgrade granular materials for base course and open grade asphalt mixtures under the static tri-axial compression test [3].

The objective of this study is to analyze the accumulated plastic strain of granular base course material by cyclic loading using the elasto-plastic model based on the modified Cam-Clay model. In this study, the further revised Cam-Clay models, which take into account the evolution rule of the rotational hardening concept proposed by Hashiguchi and Chen [4], were developed. And the simulations of the cyclic loading tri-axial compression test of a granular base course material using the revised model were carried out.

2 CALCULATION OF PLASTIC STRAIN

2.1 Revision of modified Cam-Clay model

As shown in Figure 1, Matsuoka et al. [5] transformed the SMP criterion into circle by using stress conversion Equation (1) and (2). And they adopted this transformed SMP criterion into the

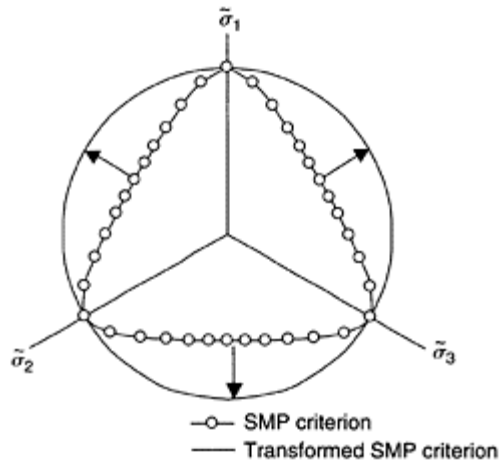


Figure 1. Transformed SMP criterion.

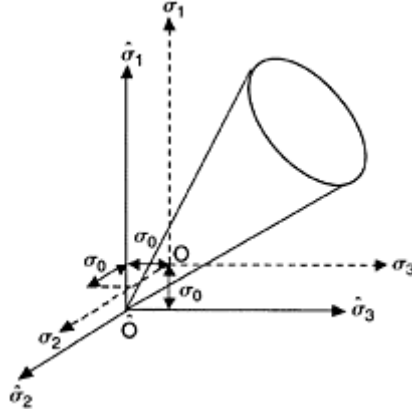


Figure 2. Coordinate transformation of failure criterion.

modified Cam-Clay model, and showed that the analytical results were more accurate than the original ones. The stress conversion equations are shown as following:

$$\tilde{\sigma}_{ij} = p \cdot \delta_{ij} + \frac{q_c}{q} (\sigma_{ij} - p \cdot \delta_{ij}) \quad (1)$$

$$q_c = \frac{2I_1}{3\sqrt{(I_1 \cdot I_2 - I_3)/(I_1 \cdot I_2 - 9I_3)} - 1} \quad (2)$$

where $\tilde{\sigma}_{ij}$ is transformed stress tensor, δ_{ij} is Kronecker's delta and $I_1 \sim I_3$ are the effective stress invariants, and $p = \sigma_{ii}/3$, $q = \sqrt{3(\sigma_{ij} - p\delta_{ij})(\sigma_{ij} - p\delta_{ij})}/2$

As shown in Figure 2, in order to take into consideration the cohesion in the SMP criterion (the extended SMP criterion), the coordinate is transformed (shifted) according to Equation (3):

$$\hat{\sigma}_{ij} = \sigma_{ij} + c \cdot \cot \phi \cdot \delta_{ij} = \sigma_{ij} + \sigma_0 \cdot \delta_{ij} \quad (3)$$

where c is the cohesion, ϕ is the angle of internal friction and the symbol “ $\wedge$ ” means the coordinate transformation. Equations (1) and (2) can be written using Equation (3) as follows:

$$\tilde{\hat{\sigma}}_{ij} = \hat{p} \cdot \delta_{ij} + \frac{\hat{q}_c}{\hat{q}} (\hat{\sigma}_{ij} - \hat{p} \cdot \delta_{ij}) \quad (4)$$

$$\hat{q}_c = \frac{2\hat{I}_1}{3\sqrt{(\hat{I}_1 \cdot \hat{I}_2 - \hat{I}_3)/(\hat{I}_1 \cdot \hat{I}_2 - 9\hat{I}_3)} - 1} \quad (5)$$

Equation (6) is the yield function of the modified Cam-Clay model expressed by the transformed stress of Equation (4).

$$f = \tilde{q}^2 + M^2 \tilde{p}^2 - M^2 \tilde{p} \tilde{p}_x = 0 \quad (6)$$

In the Cam-Clay model the $e \sim \ln p$ relationship is adopted as the relation of volumetric change with pressure. But Hashiguchi [6] pointed out that the swelling index κ and the compression index λ , which are calculated from the $e \sim \ln p$ relationship, depended on the initial void ratio e_0 , and e becomes negative when the pressure is very high. Therefore, Hashiguchi presented the $\ln V \sim \ln p$ relationship in terms of volumetric change with pressure, and introduced a swelling index γ and a compression index ρ . Based on this study, the hardening function $\tilde{p}_x$ in Equation (6) can be expressed as follows:

$$\tilde{p}_x = \tilde{p}_0 \cdot \exp\left(\frac{H}{\rho - \gamma}\right) \quad (7)$$

where $\tilde{p}_0$ is the transformed pre-consolidation pressure. H is the hardening parameter defined by Sun et al. [7] in order to calculate dilatancy (+ and -) in the Cam-Clay model and can be expressed as follows:

$$dH = \frac{M^4}{M_f^4} \cdot \frac{M_f^4 - (\tilde{q} / \tilde{p})^4}{M^4 - (\tilde{q} / \tilde{p})^4} \cdot d\varepsilon_v^p \quad (8)$$

where M_f is calculated using the internal friction angle from Equation (9), and $M = \tilde{q} / \tilde{p}$ when the plastic volumetric strain increment $d\varepsilon_v^p = 0$ [7]

$$M_f = \frac{6 \sin \phi}{3 - \sin \phi} \quad (9)$$

In the modified Cam-Clay model, if the associated flow rule is used in this study, the plastic strain increments are calculated from Equation (10):

$$d\varepsilon_{ij}^p = \Lambda \frac{\partial f}{\partial \tilde{\sigma}_{ij}} \quad (10)$$

where Λ is a positive scalar and f is the yield function, which can be expressed by Equation (11):

$$f = \tilde{q}^2 + M^2 \tilde{p}^2 - M^2 \tilde{p} \left\{ \tilde{p}_0 \exp\left(\frac{H}{\rho - \gamma}\right) \right\} = 0 \quad (11)$$

Λ can be obtained from the compatibility condition:

$$df = \frac{\partial f}{\partial \tilde{p}} d\tilde{p} + \frac{\partial f}{\partial \tilde{q}} d\tilde{q} + \frac{\partial f}{\partial H} dH = 0 \quad (12)$$

a to c are defined as follows:

$$dH = a \cdot d\varepsilon_v^p \quad (13)$$

$$b = \frac{\partial f}{\partial \tilde{p}} = 2M^2 \tilde{p} - M^2 \left\{ \tilde{p}_0 \exp\left(\frac{H}{\rho - \gamma}\right) \right\} \quad (14)$$

$$c = \frac{\partial f}{\partial H} = -M^2 \frac{\tilde{p}}{\tilde{p}_0} \frac{1}{\rho - \gamma} \exp\left(\frac{H}{\rho - \gamma}\right) \quad (15)$$

Consequently, the plastic strain increment can be easily calculated as follows:

$$d\varepsilon_{ij}^p = \Lambda \frac{\partial f}{\partial \tilde{\sigma}_{ij}} = \Lambda \left(\frac{\partial f}{\partial \tilde{p}} \cdot \frac{\partial \tilde{p}}{\partial \tilde{\sigma}_{ij}} + \frac{\partial f}{\partial \tilde{q}} \cdot \frac{\partial \tilde{q}}{\partial \tilde{\sigma}_{ij}} \right) = \Lambda \left\{ \frac{b}{3} \delta_{ij} + 3(\tilde{\sigma}_{ij} - \tilde{p} \delta_{ij}) \right\} \quad (16)$$

$$d\varepsilon_v^p = d\varepsilon_{ii}^p = \Lambda \cdot b \quad (17)$$

$$\Lambda = - \frac{b \cdot d\tilde{p} + 2\tilde{q} \cdot d\tilde{q}}{a \cdot b \cdot c} \quad (18)$$

2.2 Introduction of rotational hardening concept

In Equation (16), when $\tilde{\sigma}_{ij} - \tilde{p} \delta_{ij}$ is replaced with $\tilde{\sigma}_{ij}^* \eta_{ij}$ and $\tilde{\sigma}_{ij}^*$ is replaced with η_{ij} , Equation (19) is obtained.

$$\frac{\partial f}{\partial \tilde{\sigma}_{ij}} = \frac{\tilde{p}}{3} \left\{ 2M^2 \delta_{ij} - M^2 \frac{\tilde{p}_0}{\tilde{p}} \exp\left(\frac{H}{\rho - \gamma}\right) \delta_{ij} + 9\eta_{ij} \right\} \quad (19)$$

In Equation (19), replacing η_{ij} with $\eta_{ij}^* = \eta_{ij} - \beta_{ij}$, Sekiguchi and Ohta proposed this idea in order to account for the anisotropy of soil [8]. And Hashiguchi and Chen named this transformation the rotational hardening concept and proposed β 's evolution rule [4]. At first, Hashiguchi introduced this evolution rule into the Subloading Surface model. Since this evolution rule uses the stretching (time dependence) concept, β_{ij} increases with its evolution velocity. And this evolution rule is not directly applicable to the Cam-Clay model, because this model is time independent. Consequently in this study, in order to apply the rotational hardening concept to the Cam-Clay model, the evolution rate of β_{ij} is transposed into the increment of $d\beta_{ij}$ as follows:

$$d\beta_{ij} = b_r \cdot \left\| d\varepsilon_{ij}^{p*} \right\| \cdot \left\| \eta_{ij}^* \right\| \cdot \bar{\eta}_{ij} \quad (20)$$

$$d\varepsilon_{ij}^{p*} = d\varepsilon_{ij}^p - \frac{1}{3} d\varepsilon_{ii}^p \cdot \delta_{ij} \quad (21)$$

$$\bar{\eta}_{ij} = m_b \cdot t_{ij} - \beta_{ij} \quad (22)$$

$$m_b = \frac{2\sqrt{6} \sin \phi_b}{3 - \sin \phi_b \cdot \sin 3\theta_\sigma} \quad (23)$$

$$t_{ij} = \frac{\dot{\eta}_{ij}}{\left\| \dot{\eta}_{ij} \right\|} \quad (24)$$

$$\sin 3\theta_\sigma = -\sqrt{6} \frac{\dot{\eta}_{ii}}{\left\| \dot{\eta}_{ij} \right\|^3} \quad (25)$$

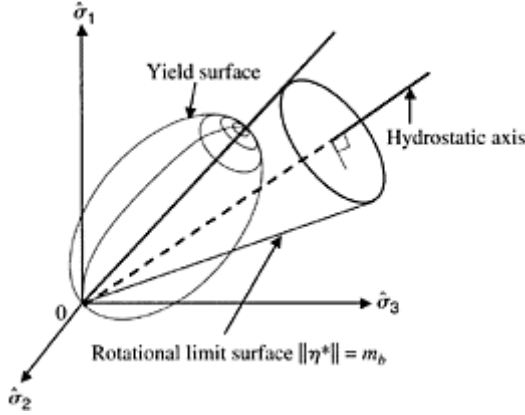


Figure 3. Rotational hardening in principal stress space.

where b_r and ϕ_b are material constants, and the symbol $\| \|$ stands for magnitude. As shown in Figure 3, the rotational limit surface is described by $\|\eta_{ij}^*\| = m_b$. And it is assumed that the initial value of β_{ij} is zero.

As a cyclic loading is proceeding, the value of the hardening parameter H increases and the value of b in Equation (14) becomes negative from positive. However, since the axial plastic strain of base course material under cyclic loading is very small and the volumetric strain is compressive, it can be assumed that a in Equation (13) is positive during the cyclic loading. And $c < 0$ is obvious from Equation (15).

$$H < (\rho - \gamma) \ln \left(2 \frac{\bar{p}}{\bar{p}_0} \right) \quad (26)$$

3 ANALYSIS OF CYCLIC TRIAXIAL COMPRESSION TEST

3.1 Cyclic triaxial compression test of base course material

In this study, in order to measure the axial plastic strain the cyclic triaxial compression test (see Figure 4) of granular base course material (the mechanical stabilized material) was carried out under the conditions as shown in Table 1. Since, the plastic strain data could not be measured until 10 times loading ($N=10$) in these tests, the plastic strain at $N=10$ were assumed zero. The resilient modulus (M_R) was obtained according to the AASHTO DESIGNATION T294-94 in this study. M_R of the granular material is plotted against the stress invariant θ ($=\sigma_1+2\sigma_3$) on logarithmic scale and a relationship between M_R (MPa) and θ (kPa) was obtained as follows:

$$M_R = 6.1226\theta^{0.6984} \quad (27)$$

3.2 Analytical condition

The swelling index γ and the compression index ρ of soil material can be measured by the isotropic consolidation test under static condition. The elastic modulus of that material can be calculated using the swelling index γ using Equations (28) to (31).

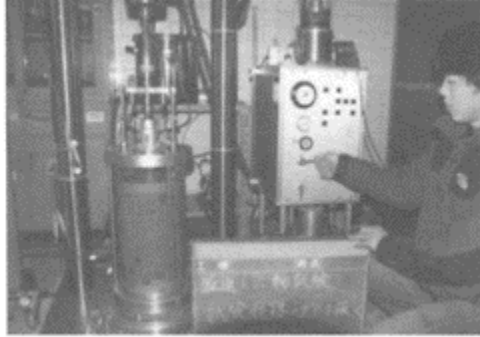


Figure 4. Cyclic triaxial compression test apparatus.

Table 1. Condition of cyclic triaxial compression test.

	Sign	Detail
Material		Mechanical stabilized (M-30)
Water content	w	4.8%
Specimen size		ϕ p150mm*300mm
Compaction		Electric compactor 15 s/Layer * 5 Layers
Loading waveform		HaveF sine
Frequency		1 Hz
Loading repetitions	N	1000 times
Confining pressure	σ_3	20, 40, 60, 80 kPa
Axial stress	σ_1	98 kPa

According to the Hook's law, the elastic strain increments can be calculated as follows:

$$d\varepsilon_{ij}^e = \frac{1+\nu}{E} d\sigma_{ij} - \frac{3\nu}{E} dp \cdot \delta_{ij} \quad (28)$$

where $d\varepsilon_{ij}^e$ is the elastic strain increment, ν is the Poisson's ratio, E is the elastic modulus and dp is the mean principal stress increment. From Equation (28), the volumetric elastic strain increments $d\varepsilon_v^e$ can be calculated as follows:

$$d\varepsilon_v^e = d\varepsilon_{ii}^e = \frac{3(1-2\nu)}{E} dp \quad (29)$$

In the $\ln V \sim \ln p$ relationship, the volumetric elastic strain is defined as follows:

$$\varepsilon_v^e = \gamma \cdot \ln \frac{p}{p_0} \quad (30)$$

Therefore, when differentiating Equation 30 with respect to p and substituting its result into Equation (29), the elastic modulus can be obtained as follows:

$$E = \frac{3(1-2\nu)}{\gamma} p \quad (31)$$

The elasto-plastic model in this study cannot take the effect of loading rate into consideration. Therefore, the loading condition affect to the material constants. For example, it is well known that

Table 2. Parameters used in analysis.

	Sign	Value
Material constants	M_f	1.64
	M	1.40
	σ_0	128 kPa
	γ	0.0004
Fitting parameters	ρ	0.0005 ~ 0.0007 @0.0001
	b_r	500,1000
	ϕ_b	30, 60°
Poisson's ratio	ν	0.3
Confining pressure	σ_3	20, 40, 60, 80 kPa
Axial stress	σ_1	98 kPa

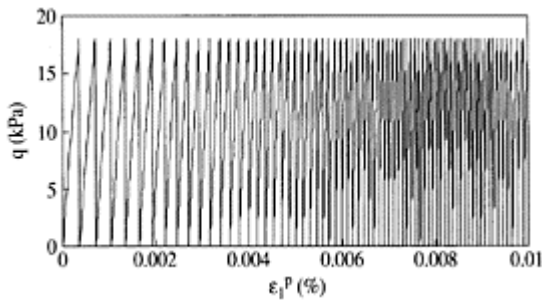


Figure 5. An example of analytical result with cyclic loading.

the elastic modulus is different when the loading condition is static or cyclic, i.e. as shown in Equation (31) the swelling index γ is change with the loading condition.

Since the swelling index γ and the compression index ρ cannot be measured in the cyclic loading test directly, it was assumed that the elastic modulus E equals the resilient modulus M_R and γ was derived using Equation (31). And ρ , b_r and ϕ_b are obtained by fitting the calculated parameters to the measured ones.

Table 2 shows the parameters used in the analysis. All parameters except γ , ρ , b_r and ϕ_b were measured from a static triaxial compression test result [3]. In the static test analysis, the value of γ was about 0.001. But in the cyclic test analysis, that was about half of the static one. A decrease of the swelling index means an increase of the loading rate. In this analysis the axial plastic strain was calculated until 500 loading repetitions.

3.3 Analytical results

Figure 5 shows an example of the axial plastic strain analysis results. From this figure, it is found that the axial plastic strain increments decreased with cyclic loading suggesting the effect of the rotational hardening.

Figures 6 to 9 shows the comparisons of the predicted axial plastic strain and the experimental ones for each confining pressure. In those figures, the axial plastic strain at 10th repetition was assumed 0.

From all figures, it is found that the analytical results increase with ρ increasing. When $\sigma_3=40$ and 60 kPa, the analytical and the experimental result of the axial plastic strain increments showed a slightly different tendency. For all cases of $\sigma_3=20$ kPa and $\sigma_3=40$ kPa, the analytical results agree well with the experimental ones, when $\rho=0.0005 \sim 0.0006$. On the whole, it is said that the

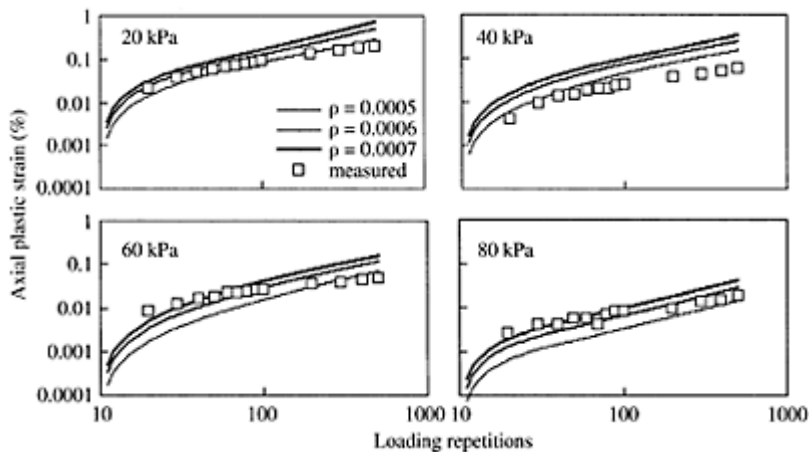


Figure 6. Comparison of the axial plastic strain when $b_r=1000$, $\phi_b = 60^\circ$.

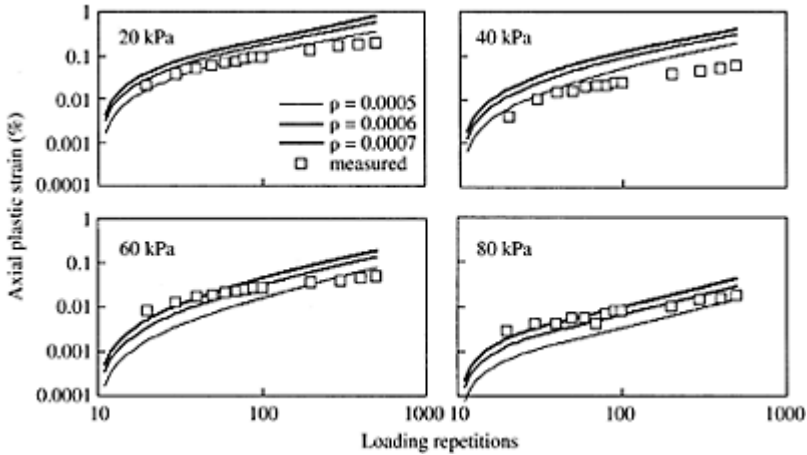


Figure 7. Comparisons of the axial plastic strain when $b_r=1000$, $\phi_b = 30^\circ$.

analytical results of the axial plastic strain agree well with the experimental ones when $\rho=0.0005\sim0.0006$.

From Figures 6 to 9 it is found that the predicted plastic strains slightly increase with ϕ_b increasing. Also, the predicted plastic strain slightly increases with b_r increasing. However, the increments of the predicted strains by b_r and ϕ_b are smaller than those by ρ . From these results, it can be said that the effect of b_r and ϕ_b on the analytical result is smaller than the effect of ρ . Consequently, in the cyclic plastic strain analysis of granular base course material, the effect of ρ seems to be dominant in the analytical result.

4 CONCLUDING REMARKS

In order to simulate the axial plastic behavior of base course and subgrade materials under cyclic loading, the modified Cam-Clay model revised using the evolution rule of the rotational harden-

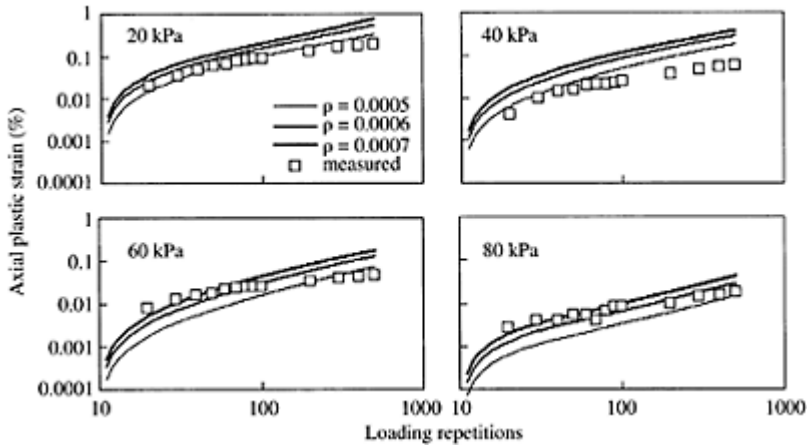


Figure 8. Comparisons of the axial plastic strain when $b_r=500$, $\phi_b = 60^\circ$.

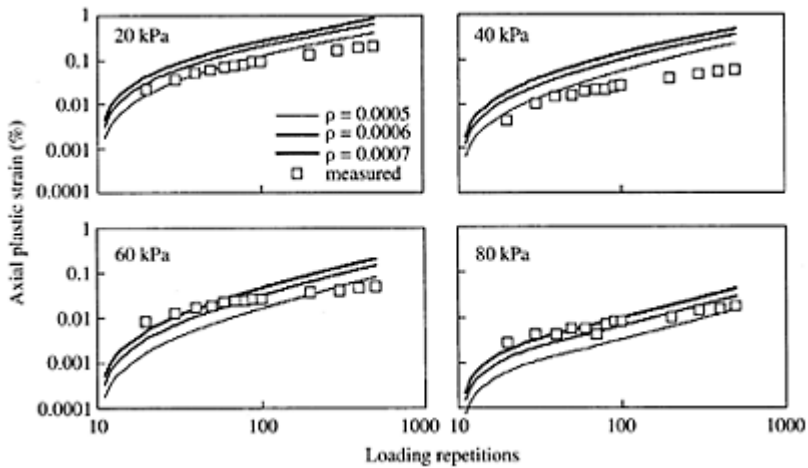


Figure 9. Comparisons of the axial plastic strain when $b_r=500$, $\phi_b = 30^\circ$.

ing concept proposed by Hashiguchi was developed. And the analytical results were compared with the experimental results of triaxial compression test of granular base course material under cyclic loading. The main results are summarized as follows:

- The elasto plastic model developed in this study was effective to predict the axial plastic strain of the granular base course material under cyclic triaxial compression condition. It is expected that this model would be applicable for the cyclic triaxial compression analysis of the subgrade materials.

- The swelling index γ can be calculated from the resilient modulus M_R . But in the cyclic test analysis, the value of γ was about half of the static one.
- The compression index ρ and the material constants b_r, ϕ_b cannot be measured directly from the cyclic loading test. Therefore, it seems to be desirable that these parameters should be treated as fitting parameters.
- In the cyclic plastic strain analysis, the effect of b_r and ϕ_b on the analytical result is very small while the effect of ρ seems to be dominant.

In this paper, the volumetric strain was not measured, because the sensitivity of the burette for volumetric strain measurement was insufficient. So we are planning to measure the volumetric strain and obtain the parameters of the elasto-plastic model for the cyclic plastic strain analysis.

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REFERENCES

- 1) Takeuchi, Y. et al., 2001, "Permanent deformation of base course under concrete pavement and its effect to fatigue failure", *Proc. 7th International Conference on Concrete Pavements*, No.0047, pp 1–14.
- 2) Matsuoka, H. et al., 1990, "A general failure criterion and stress-strain relation for granular materials to metals", *Soils and Foundations*, Japan Society of Soil Mechanics and Foundation Engineering, Vol.30, No.2, pp 119–127.
- 3) Takeuchi, Y., 2002, "Fundamental study on elasto-plastic analysis model for pavement materials", *Journal of Pavement Engineering*, Japan Society of Civil Engineers, Vol.7, No.24, pp 1–12.
- 4) Hashiguchi, K. & Chen, Z.P., 1998, "Elastoplastic constitutive equation of soils with the subloading surface and the rotational hardening", *International Journal for Numerical and Analytical Methods in Geomechanics*, 22, pp 199–227.
- 5) Matsuoka, H. et al., 1999, "The Cam-Clay models revised by the SMP criterion", *Soils and Foundations*, Japanese Geotechnical Society, Vol.39, No.1, pp 81–95.
- 6) Hashiguchi, K., 1974, "Isotropic hardening theory of granular media", *Proceedings of Japan Society of Civil Engineers*, No.227, pp 45–60.
- 7) Sun, D.A. et al., 2001, "A transformed stress based on extended SMP criterion and its application to elastoplastic model for geomaterials", *Journal of Geotechnical Engineering*, Japan Society of Civil Engineers, No.680/III-55, pp 211–224.
- 8) Sekiguchi, H. & Ohta, H., 1977, "Induced anisotropy and time dependency of clay", *International Conference of Soil Mechanics and Foundation Engineering*, pp 229–239.

Shakedown analysis of unbound road pavements—an experimental point of view

P.S.Ravindra

University of Sydney, NSW Australia

J.C.Small

Department of Civil Engineering, University of Sydney, NSW, Australia

Pavements Unbound—Dawson (ed.)

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ABSTRACT: Shakedown behaviour of road pavements is investigated under laboratory controlled conditions using the Sydney University Pavement Testing Facility. The Pavement Test Facility was upgraded to provide improved data acquisition, storage and analysis for the experiment. Three different unbound pavement configurations sealed with bitumen emulsion seal have been tested so far. Observed results are consistent with the pavement shakedown. Wheel loads lower than the shakedown load generated low permanent deformations for a larger number of load cycles in comparison with high permanent deformations for a lower number of load cycles for wheel loads higher than the shakedown load.

1 INTRODUCTION

Pavement design decisions are primarily based on the pavement life cycle cost together with an acceptable pavement maintenance regime for the particular road under consideration. Riding comfort (increase in roughness or pavement serviceability-performance) is an important factor in the design process although the pavement needs to be designed against structural failure in order to carry the design loads. The design inputs range from axle load and traffic analysis, environment factors, pavement material properties, improvement of locally available material, properties of the subgrade, properties of bases and sub bases, available construction standards and equipment, surfacing material and fundamental stress-strain analysis.

Observed performance under actual conditions is the final criterion to establish the validity of a design methodology. Experimental pavements laid on public roads and subjected to normal road traffic are essential to the development and proving of structural design standards. (Performance observations carried out on existing road sections in the UK from Boroughbridge (1949) to Conington and Cambridgeshire (1965) by TRRL fall into this category of tests.) In such experiments, however, performance can be assessed only in terms of the axle-load spectrum to which the sections are exposed and little

quantitative information can be gained about the relative damaging effects of the different axle loads constituting the traffic. A separate study of the relationship between pavement damage and axle loading is necessary to enable the results from full-scale road experiments to be generalized and applied to other traffic conditions. Such studies are made using test tracks on which the loading can be strictly controlled. The track may use full-scale road pavements on which the traffic is restricted to repetitions of specific axle loads applied by normal road vehicles. The WASHO Road Tests in Idaho (1952–54) and AASHO Road Tests in Illinois (1958–60) were of this type.

An analysis which incorporates the substantial strength existing prior to the point of static collapse has been suggested by (Sharp and Booker, 1984) who pioneered the application of Melan's (1936) Shakedown Theory to road pavements. The shakedown model identified a critical load level below which shakedown occurs, but above which permanent strains continue to occur. Extensions for Melan's lower bound approach to calculate the shakedown load was presented by (Raad et al, 1988; 1989), (Hossain and Yu, 1996) and (Shiau and Yu, 2000). Upper estimates of the shakedown load were obtained by (Collins and Cliffe, 1987) employing the dual kinematic theorem due to Koiter (1960). They have shown that in the two dimensional case results were identical with Sharp and Booker's lower bound approach. Sharp and Booker (1984) have shown AASHTO (1950) experimental results agree with their parametric study, they concluded that there is a clear need for further experimental work so that such design criteria could be validated.

Werkmeister S, Dawson AR and Wellner F, (2001) reported the results of RLT (Repeated Load Triaxial) tests performed on unbound pavement material at the University of Nottingham's Centre for Pavement Engineering from 1999 to 2002 and concluded that shakedown limit calculations in combination with FE-calculations of insitu stresses can be used to predict whether or not stable behaviour occurs in the UGL (Unbound Granular Layers) of a pavement construction.

In this paper, it is intended to discuss a laboratory experiment presently underway at the University of Sydney Pavement Testing Facility in Australia, to examine the application of shakedown theory by means of measuring accumulated plastic deformation, after applying traffic loads below and above the theoretical shakedown load. To facilitate the data acquisition, storage and analysis, a new data acquisition system was developed and installed for the Pavement Test Facility as a part of this experiment.

2 SYDNEY UNIVERSITY PAVEMENT TESTING FACILITY

2.1 *Introduction*

The testing facility was initially developed by Wong and Small (1994) to test model pavements and was modified in order to change the position of the tyre across the pavement randomly (Moghaddas-Nejad and Small (1996)). The facility consists of three main structural components, namely, the test tank, the overhead track and the loading carriage. The support and guidance for the moving loading carriage is provided by the overhead rails. The test section of pavement is constructed inside the test tank and positioned below one of the straight sections of the overhead track. The test wheel runs

on a plywood track when outside of the test section of pavement. A conductor rail system supplies power to the motor which drives the wheel. The wheel passes over the test section of pavement once during each revolution around the track and triggers a micro switch that starts a micro computer recording data. Test facility specifications are shown in Table 1.

Table 1. Test facility specifications.

Feature	Specification
Speed range (km/h)	0–7.2
Wheel load (kN)	0–1.4
Maximum tyre pressure (kPa)	500
Tyre width (mm)	45
Tyre diameter (mm)	220
Length of test section (m)	1.4
Width of test section (m)	0.5
Maximum depth of tank (m)	0.8
Length of test track (m)	12.15
Time to complete one circuit at 1 km/h (s)	44.0

2.2 Instrumentation and data acquisition

All subsurface settlements are measured by buried Perspex discs (30mm diameter) that are connected to wires that pass through the base of the tank and are connected to LVDT (Linear Voltage Differential Transducers). Data acquisition from the transducer output is carried out only during the passage of the wheel across the test section of pavement. Data acquired were directly written to a relation data base, comprised of subsurface pavement settlement data from the LVDTs and time stamps, pavement position location details, number of test cycles, spring load monitoring data, subsurface permanent deformation details and test facility calibration data. To cater for the large number of cross section measurements envisaged in the project, a new laser transducer based surface deformation measurement system was designed and developed to measure and record the surface deformation measurements.

3 MATERIAL PROPERTIES

3.1 Sand subgrade

Loose Silica Sand was used as the subgrade. Particle size distribution is as shown in Table 2.

3.2 Base material

Base material used in the experiment was made up of recycled crushed concrete aggregate obtained from Randwick Yarra Bay Material stockpile site. This material mainly consisted of crushed concrete obtained from council building demolition sites. This material is used as a base material in road rehabilitation works in the City Council area and is found to be easy to handle at the site with respect to compaction and spreading.

All material tests carried out were compared with current AUSPEC specifications and relevant ARRB recommended specifications for recycled crushed concrete specifications. Direct Shear Box tests and Texas Triaxial tests were carried out to determine the angle of internal friction and cohesion values. Test results are shown in Tables 3 and 4.

4 CONSTRUCTION OF PAVEMENT

4.1 Pavement

A Standard Proctor hammer was used to compact the pavement layers. Moisture content was maintained at OMC and the numbers of blows applied were evenly spread across the layer and were made equal to give the same energy level to the base material as in the Standard Compaction Test. The pavement surface was allowed to dry at 20°C before lightly brushing off the loose fines

Table 2. Particle size distribution of subgrade sand
(Test specification AS 1289 3.6.1).

Sieve size (mm)	4.75	2.36	1.18	0.600	0.425	0.300	0.15
% Passing	100	99	99	98	89	54	2

Table 3. Soil strength test results for base material.

Texas Triaxial	Direct Shear Box
$\phi = 46^{\circ}$	$\phi = 51^{\circ}$
c=45 kPa	c=56 kPa

Table 4. Particle size distribution test results for
base material.

Sieve size (mm)	Sample				ARRB specification
	1	2	3	4	
26.5	100	100	100	100	100
19	93	97	98	95	95–100
13.2	77	76	82	81	75–95

9.5	65	66	69	72	
6.7	58	59	61	65	
4.75	51	53	54	58	42–76
2.36	44	44	46	50	28–60
1.18	38	39	40	43	
0.6	32	33	34	37	
0.425	27	27	27	31	10–28
0.30	17	18	17	20	
0.150	6	7	6	8	
0.075	4	4	4	5	2–10

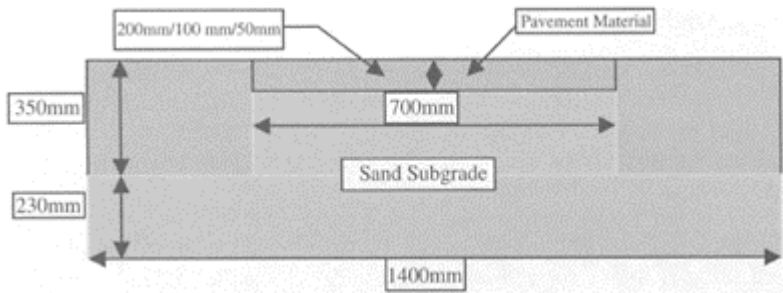


Figure 1. General pavement configuration.

in order to apply the bitumen emulsion seal coat with 5 mm single sized cover aggregate. Emulsion was applied with a roller brush and spread evenly across the pavement. After application of the cover aggregate, a light compaction pass was applied to embed the cover aggregate into the emulsion layer as a single layer of thickness equal to the aggregate's least dimension. A further 24 to 48 hours drying period was allowed before brushing the loose cover aggregate off the surface. To date, three different test pavements have been tested in the experiment. Wheel entry and exit sections were kept at 350 mm thickness for all tests and the 700 mm length mid section thickness was varied to obtain different theoretical shakedown loads. The general pavement configuration is shown in Figure 1.

5 CALCULATION OF SHAKEDOWN THICKNESS

5.1 General

The shakedown theorem due to Melan (1936) as presented by Maier (1969), described the main limitations of the classical theory as: “(a) Inviscid perfectly-plastic (non-hardening) laws govern the local deformability, and involve convex yield surfaces, associative flow rules and constant elastic moduli (the term inviscid rules out time-effects, such as creep and rate-sensitivity); (b) geometric changes do not significantly affect the equilibrium relations; (c) temperature changes have negligible influence on the material properties; (d) external agencies act so slowly that the system

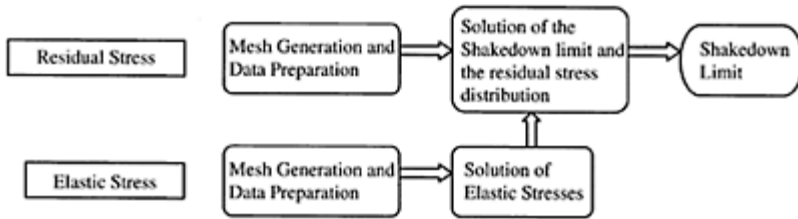


Figure 2. Calculation of shakedown limit.

behaves in a quasi-static way (with negligible inertia and viscous forces); (e) adaptation guarantees the survival of the structure, i.e., rules out structural crises by excessive deformation or local failure.”

The question of the validity of the shakedown theorems for materials with non-associated flow rules has been examined by Maier (1973). He showed that the bounds given by Koiter’s theorem are still upper bounds to the shakedown load, even though the real material has a non-associated flow rule. Melan’s theorem can be used to obtain a lower bound to the shakedown limit for a nonstandard material, but the yield-surface must be replaced by a “potential surface” which lies inside the yield surface.

Sharp and Booker (1984) applied the linear programming technique adopted by Maier (1969). They assumed a plane strain model with a trapezoidal pressure distribution under a roller. The material of the half space was assumed to be isotropic and homogeneous and the resulting permanent deformation and residual stress distribution were assumed independent of horizontal distance and dependent on the depth. The tangential shear load was taken as a trapezoidal distribution. The failure criterion was Mohr-Coulomb and material properties were assumed to be linear elastic—perfectly plastic.

5.2 Calculation of shakedown limit

Calculation of shakedown limits for the various pavement configurations tested in this experiment is based on a lower bound calculation procedure developed by S.H.Shiau

(2001) in his PhD thesis. Calculation procedure is depicted in Figure 2. This procedure assumes that both elastic stresses and residual stresses required by the lower bound shakedown analysis are linearly distributed across the continua and the resulting deformation is plane strain by replacing the wheel load as an infinitely wide roller. A trapezoidal load distribution was selected as the contact load distribution. The finite element and the linear programming approach are used in this procedure.

Results of the shakedown limit calculations are presented in the form of a dimensionless shakedown limit ($\lambda P_v/C$) where P_v is the vertical pressure at which the elastic stresses are calculated, C is the cohesion of the base material and λ is the shakedown load factor. The shakedown limit for the two layered system depends on a number parameters such as stiffness ratio (E_b/E_s), strength ratio (C_b/C_s), thickness of the base course to contact length (h/B), surface friction ($\mu=P_h/P_v$), Poisson's ratios ν_b and ν_s , friction angles ϕ_b and ϕ_s . Subscripts b , s , h and v stands for base, subgrade, horizontal and vertical respectively. Computer software used for the mesh generation, elastic analysis and shakedown analysis could be found at LSHAKE (2001).

6 TEST RESULTS AND DISCUSSION

6.1 Database

Data acquired comprised subsurface pavement settlement data from LVDTs, the time of reading, the lateral location of the wheel (as the wheel can be moved laterally relative to the pavement),

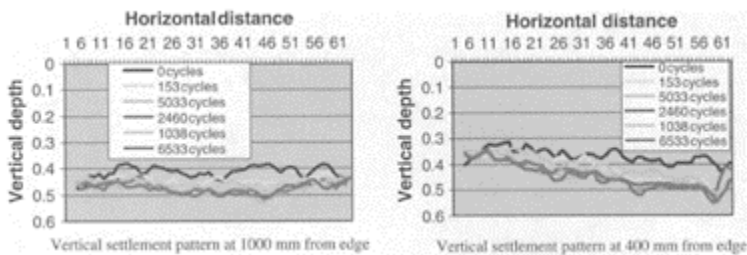


Figure 3. Pavement settlement pattern when wheel load (80 N) is more than the theoretical shakedown load (5 N) of the test pavement (Pavement thickness 50 mm).

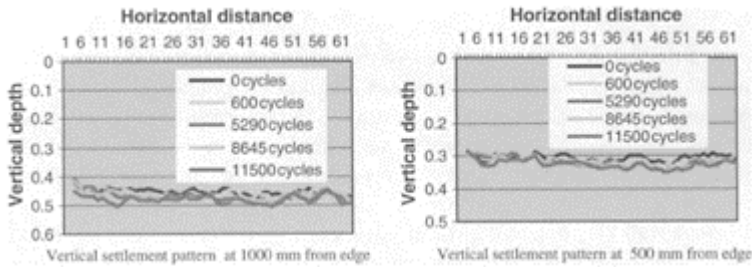


Figure 4. Pavement settlement pattern when wheel (80 N) load is less than the theoretical shakedown load (229 N) of the test pavement (Pavement thickness 200 mm).

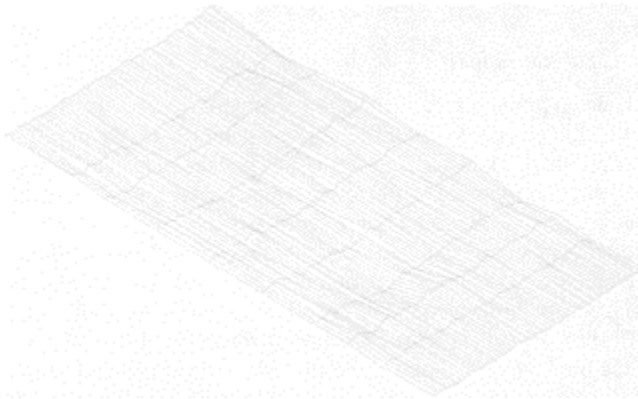


Figure 5. 3D mesh representation of pavement vertical surface deformation (VSD) plotted directly from the test results data base (after 4800 cycles when wheel load (80 N) is more than shakedown load (5 N)) (Pavement thickness 50 mm).

number of test cycles, spring load monitoring data, subsurface permanent deformation details and transducer calibration data. Results were written to a relational database enabling online analysis and processing (OLAP) of test data. Some typical pavement settlement patterns from the cross-section measurement database are shown in Figures 3 to 4 and a deformed surface plot is shown in Figure 5. Results so far indicate that larger

deformations occur for fewer load cycles when the wheel load is more than the shakedown load.

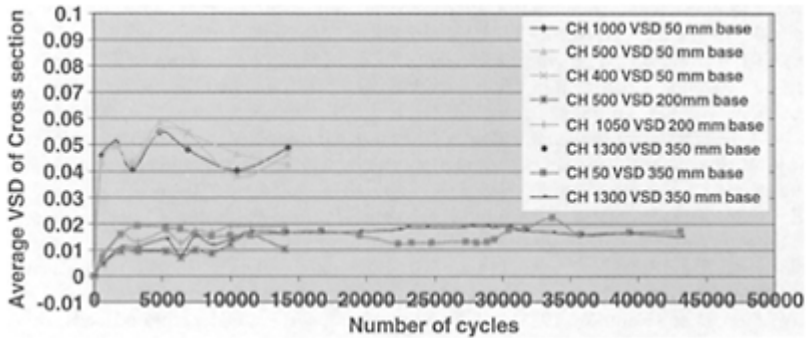


Figure 6. Average VSD of the particular cross section plotted against number of cycles.

6.2 Test results

Vertical surface deformation (VSD) of the pavement surface was measured at nine cross sections initially. The number of cross sections was increased to cover the progression of VSD and to produce 3D mesh images of the pavement surface (Figure 5). This made it possible to visualize the settlement pattern of the total pavement rather than individual cross sections. Standard measures such as pavement rutting and roughness are related to VSD but do not have the same degree of reliability. Usually rutting is determined by calculating or measuring the depth of a rut from a straight edge placed across the wheel path and is affected by any heaving at the edges. Roughness represents the variation of VSD along the wheel path. Progression of the average VSD at each cross section with the load cycles is shown in Figure 6.

Results obtained so far by this experiment indicate that there is a rapid increase in VSD in the case of wheel loads more than the shakedown load calculated by the two dimensional shakedown load for the test pavement (case of 50 mm base pavement in Figure 6). In the current tests, trafficking was terminated when the pavement reached a shakedown state or when VSD increased more than 10 mm in a particular cross section (this corresponds to a 40 mm depth in the prototype). More tests are needed to verify the long term behaviour of the pavement.

7 CONCLUSIONS

Preliminary testing of pavements with the University of Sydney Pavement Testing Facility have indicated that the shakedown loads predicted by 2-D shakedown theory analyses are a good indicator of whether a pavement will undergo continued deformation under cyclic wheel loading. Results so far have indicated that at loading levels above the

shakedown limit of the test pavement, VSD increased rapidly to form a rut indicating pavement failure and when the loading levels were lower than the shakedown limit, after an initial vertical settlement VSD remained constant for a sufficiently large number of cycles.

REFERENCES

- AASHTO (1994), *Guide for Design of Pavement Structure*, American Association of State Highway and Transportation Officials.
- AUSTROADS (2000), *Pavement Design: A Guide to the Structural Design of Road Pavement*, Sydney, Australia.
- Collins, I.F. and Cliffe, P.F., 1987, Shakedown in frictional materials under moving surface loads, *International Journal for Numerical and Analytical Methods in Geomechanics*, 11, pp 409–420.
- Hossain, M.Z. and Yu, H.S., 1996, Shakedown analysis of multi layer pavements using finite element and linear programming, *Proc. 7th Australia-New Zealand Conference on Geomechanics*, Adelaide, pp 512–520.
- Koiter, W.T., 1960, General Theorem for Elastic-Plastic Solids, in *Progress in Solid Mechanics*, Eds. Sneddon, J.N. and Hill, R., 1, North Holland, Amsterdam, The Netherlands.
- LSHAKE, 2001. *A Lower Bound Program for the Shakedown Analysis of Continua under Moving Surface Loads*, Geotechnical Group, Department of Civil, Surveying and Environmental Engineering, The University of Newcastle, NSW 2308, Australia.
- LR 132, 1984, *The Design of Bituminous Roads*, Transport and Road Research Laboratory, UK.
- Maier, G., 1969, Shakedown theory in perfect elastoplasticity with associated and non-associated flow-laws: A finite element, linear programming approach, *Meccanica*, Vol. 4, pp 250–260.
- Melan, E., 1936, Theorie statisch unbestimmter Systeme, *Prelim. Publ. 2nd Congress of International Association of Bridge and Structure Engineering*, Berlin, p 43.
- Moghaddas-Nejad, F. and Small, J.C., 1996, Effect of Geogrid Reinforcement in Model Track Tests on Pavements, *Journal of Transportation Engineering*, Vol. 127, No. 6, pp 468–474.
- Raad, L., Weichert, D. and Najm, W., 1988, Stability of multilayer systems under repeated loads, *Transportation Research Record*, 1207, pp 181–186.
- Raad, L., Weichert, D. and Haider, A., 1989, Analysis of full-depth asphalt concrete pavements using shakedown theory, *Transportation Research Record*, 1227, pp 53–65.
- Sharp, R.W. and Booker, J.R., 1984, Shakedown of Pavements under moving surface loads, *Journal of Transportation Engineering*, ASCE, Vol. 110, No. 1, pp 1–14.
- Shiau, S.H. and Yu, H.S., 2000, Shakedown analysis of flexible pavements, *Developments in Theoretical Geomechanics*, Smith & Cater, Balkema.
- Shiau, S.H., 2001, Numerical Methods for Shakedown Analysis of Pavements under Moving Loads, PhD thesis, University of Newcastle, Australia.
- Werkmeister, S., Dawson, A.R. and Wellner, F., 2001, Permanent deformation behaviour of unbound granular materials and the shakedown-theory, *Transportation Research Record* No. 1757, Transportation Research Board, Washington, D.C., pp 75–81.
- Wong, H.K.W. and Small, J.C., 1994, Effect of Orientation of Approach Slabs on Pavement Deformation, *Journal of Transportation Engineering*, Vol. 120, No. 4, pp 590–602.

*Pavement performance,
evaluation and management*

Damage law exponents for thin surfaced granular pavements

G.Arnold

Transit New Zealand, Wellington, New Zealand

D.Alabaster

Transit New Zealand, Christchurch, New Zealand

B.Steven

University of Canterbury, Christchurch, New Zealand

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ABSTRACT: A three year research study at Transit New Zealand's pavement testing facility, CAPTIF was undertaken to assess the relative effect of increases in wheel loads to unbound granular pavements. CAPTIF is a circular track where the wheel paths of the two vehicles can be separated to assess the relative damaging effect of the pavement and surfacing. Tests were conducted comparing the damaging effect of 40 kN (equivalent 8.2 tonne dual tyred axle) compared with 50 kN (equivalent 10 tonne dual tyred axle) and 60 kN (equivalent 12 tonne dual tyred axle) on the pavement with four different aggregate types, two thicknesses and one subgrade type. For each load comparison a total of 1 million wheel passes were applied. During the testing falling weight deflectometer (FWD), and rut measurements were taken in both wheel paths. It was found that the pavement structural number was a good predictor of relative damage compared to the standard load. Relative damage was defined in terms of damage law exponent akin with the fourth power law used to calculate the design traffic loading in pavement design. A range of damage law exponents was obtained from 1 to 4 that could be related to the pavements structural number. Higher exponent values were obtained for the weaker pavements (i.e. low pavement structural number).

1 INTRODUCTION

1.1 Background

The road transport freight industry in New Zealand understandably wishes to increase its efficiency. One of the ways to do this is through increases in the allowable mass limits

for heavy vehicles. This in turn can result in economic benefits to the whole country provided the impact of the changes in mass limits are accurately known and considered in assigning the new limits and in determining appropriate road user charges (RUC). Heavy vehicles are charged in New Zealand according to use of road space and expected pavement wear. One of the impacts concerning road controlling authorities (RCA) is the effect on increasing mass limits on the life of their pavements or how much more pavement rehabilitation and maintenance will be required.

In response to possibly inevitable increases in mass limits on New Zealand roads a three year research study at Transit New Zealand's pavement testing facility, CAPTIF was undertaken. CAPTIF (Figures 1 and 2) is a circular track where the wheel paths of the two vehicles can be separated to assess the relative damaging effect of the pavement and surfacing. Tests were conducted comparing the damaging effect of 40 kN (equivalent 8.2 tonne dual tyred axle) compared with 50 kN and 60 kN on the pavement with a few different aggregate types and thicknesses. Full results of these tests are reported in de Pont et al (2001 and 2002) and Arnold et al (2001, 2003 and 2004).

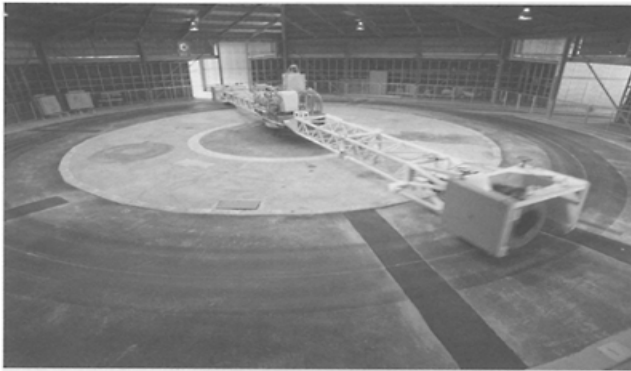


Figure 1. Transit New Zealand's pavement testing facility, CAPTIF.

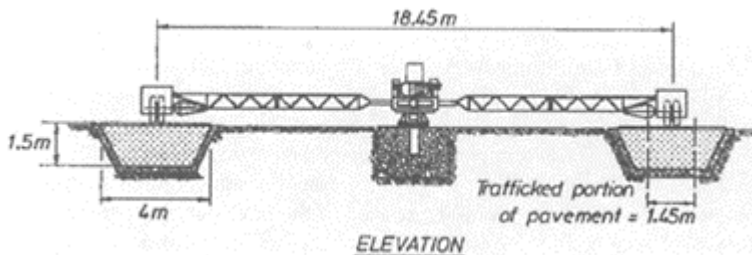


Figure 2. Elevation view of CAPTIF.

1.2 Damage law exponent

Road traffic consists of a range of vehicle types, wheels and loads. A method intrinsic in pavement design and deterioration modelling is to combine all traffic into one type. This one type of traffic is commonly referred to as an Equivalent Standard Axle (ESA). The standard axle is defined as a single axle with dual wheels that carries a load of 8.2 tonnes (40 kN half dual tyred axle as tested at CAPTIF). Equation 1 from the Austroads Pavement Design Guide (Austroads 1992) is used to calculate the number of ESAs for any given traffic distribution.

$$ESAs = \left[\frac{Axle\_load}{Axle\_load\_reference} \right]^n \quad (1)$$

where:

ESAs = number of standard axles needed to cause the same damage as one pass of the actual axle load;

Axle_load = actual axle load in kN;

Axle_load_reference = reference load depending on the axle load group as defined in Table 1;

n = damage law exponent (commonly=4).

Table 1. Reference axle loads (Table 7.1, Austroads 1992).

Axle:	Single	Single	Tandem	Triaxle
Tyres:	Single	Dual	Dual	Dual
Load (kN)	53	80	135	181

The origins of the damage law equation (Equation 1) for combining traffic into one type came from the AASHTO (1962) road test which was conducted in the United States in the late 1950s using roads and vehicles which bear little resemblance to those in use in New Zealand today. AASHTO calculated a damage law exponent of 4 based on a comparison of the number of axle passes to reach the end of the pavement life between the reference axle and the axle load in question. The pavement end of life in the AASHTO tests was defined by reaching a certain pavement serviceability index value which considers factors such as rut depth, roughness and cracking. The damage law equation can be re-written for the purpose of determining the most appropriate damage law exponent as per Equation 2:

$$n = \frac{\log \left(\frac{N_{40kN}}{N_{L\_kN}} \right)}{\log \left(\frac{P_{L\_kN}}{P_{40kN}} \right)} \quad (2)$$

Where:

n = the exponent of the power law;

N_{L_kN} = the load cycles of load *P_{L_kN}* to reach a certain level of wear defined as the pavements end

of life; and

N_{40kN} = the load cycles of load P_{40kN} to reach the same given level of wear defining the pavements end of life as achieved by load $P_{L,kN}$ in $N_{L,kN}$ load cycles.

The tests at CAPTIF determined the number of wheel passes to reach a specified failure condition for the 50 and 60 kN loads compared to the standard dual tyred half axle of 40 kN for the calculation of the damage law exponent. Failure was defined as when the pavement reached a vertical surface deformation (VSD) of 15 mm. Vertical surface deformation is more conveniently measured at CAPTIF with the transverse profilometer as it's the maximum vertical difference between the start reference level of the pavement. Rut depth is determined using a straight edge across the pavement and considers the upward shoving at the edges of the wheel path. However, at CAPTIF the edges moved downwards which distorted the straight edge rut depth as shown in Figure 3. From this Figure it can be seen that the two wheel loads could interfere with each other. It is possible that the lighter wheel path could result in higher VSD than otherwise would be the case if there was no interference with the other heavier wheel path. However, it is noted that the effect does not extend to the outer edges of the wheel path. This influence will be investigated further through pavement modelling that predicts rut depth.

A VSD value of 15 mm at the end of the pavements life was chosen as this corresponds to a time when rutting accelerates rapidly towards failure. Further, when a VSD of 15 mm occurs, in the field water would pond in the wheel tracks, which would accelerate the onset of failure. Thus a VSD value of 15 mm to define the pavements end of life at CAPTIF being a dry environment is considered appropriate. Other VSD values of 10 and 20 mm were trialled where it was found to affect the calculated lives. However, there was little difference between the calculated damage law exponents for the different VSD values as this is a measure of the relative difference in lives.

For each CAPTIF pavement Segment, there are atleast 10 measurements of VSD at 1 m stations for each wheel path. The number of wheel passes to reach a VSD value of 15 mm was determined for each station's measurements. Finally, the life of the pavement Segment was the 10 percentile value or when 10% of the segment has reached the failure criteria. This failure criteria of the whole

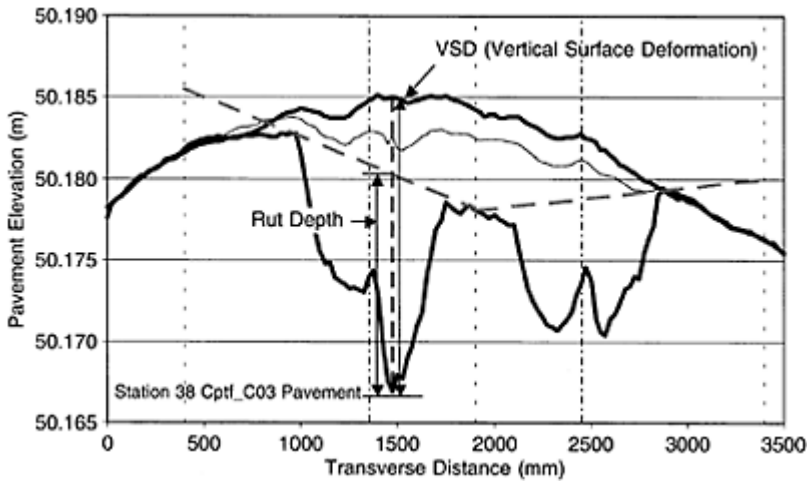


Figure 3. Typical measurement of rut depth and VSD at CAPTIF.

Segment is akin to the failure criteria or more appropriately an investigatory level for New Zealand's highway network taken from high speed rut measurements. The average life was also investigated but it was found that this value did not result in significant differences between damage law exponent for the pavement Segments. Another approach investigated was the calculation of damage law exponent values for each individual 1 m station measurement of VSD. However, the result using individual stations resulted in significant scatter in power law exponent values and upon discussion with Transit New Zealand it was considered more appropriate to keep with the same definition of pavement life as used on state highways (i.e when 10% of the pavement reaches the failure criteria).

2 TEST PAVEMENTS

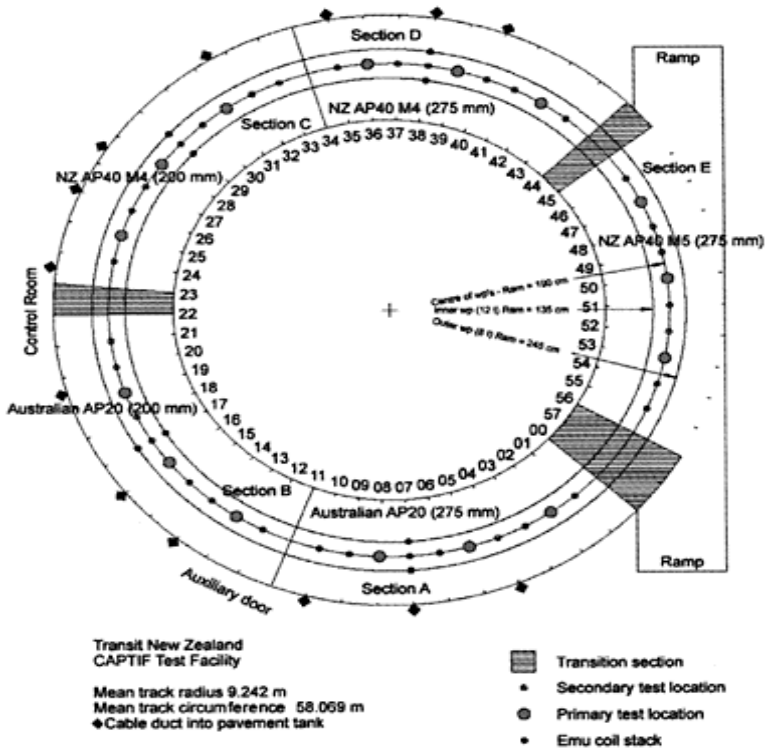
All the pavements tested consisted of three layers: the surface (25mm); aggregate; and subgrade. Thicknesses and materials used of the various pavement types tested are detailed in Table 2. As can be seen all the pavements have the same medium strength subgrade with pavement depth varying from 250 to 320 mm. A total of 5 different aggregates are used, where 3 of these are considered premium quality crushed rock. The other two aggregates considered as sub-standard were a rounded uncrushed river gravel in the Cptf_E03 segment and a aggregate contaminated with 10% by mass of silty clay fines was used in the Cptf_B01 segment. The subgrade depth was 1200 mm and the pavement was enclosed by concrete walls (Figure 2). The Segments with different aggregates were divided around the track as shown in Figure 4 for the 2003 test at CAPTIF.

3 PAVEMENT LIFE

For each station in each CAPTIF pavement segment (Table 2) vertical surface deformation (VSD, Figure 3) was measured at various loading increments up to 1 million loading passes. As discussed in Section 1.2 the value of VSD that governs the end of the pavement life was chosen to be 15 mm. However, a VSD of 15 mm was only achieved on the few stations that failed early. Therefore, the VSD measurements were extrapolated to estimate the number of wheel passes until the VSD was equal to 15 mm. The method of extrapolation was a best fit linear projection to the data from 150 k

Table 2. Summary of test pavements.

ID	Pavement thickness (mm)	Aggregate short descr.	Aggregate description	Subgrade type
Cptf_A03	320	Montrose Class 2	A 20 mm max size rhyolite from Montrose, Vic. Aust.	Silty clay (CBR=1 1%)
Cptf_B03	250			
Cptf_C03	250	AP40 TNZ M/4	A 40 mm max size alluvial greywacke from Canterbury, NZ.	
Cptf_D03	320			
Cptf_E03	320	Rounded AP40 TNZ M/5	A 40 mm max size uncrushed rounded river gravel from Canterbury, NZ.	
Cptf_A01	300	AP40 TNZ M/4	A 40 mm max size alluvial greywacke from Canterbury, NZ.	
Cptf_B01	300	AP40 TNZ M/4+fines	A 40 mm max size alluvial greywacke contaminated with 10% by mass of silty clay fines from Canterbury, NZ.	
Cptf_C01	300	Montrose Class 2	A 20 mm max size rhyolite from Montrose, Vic. Aust.	
Cptf_D01	300	Recycled concrete	Recycled crushed concrete from Auckland building demolition sites.	



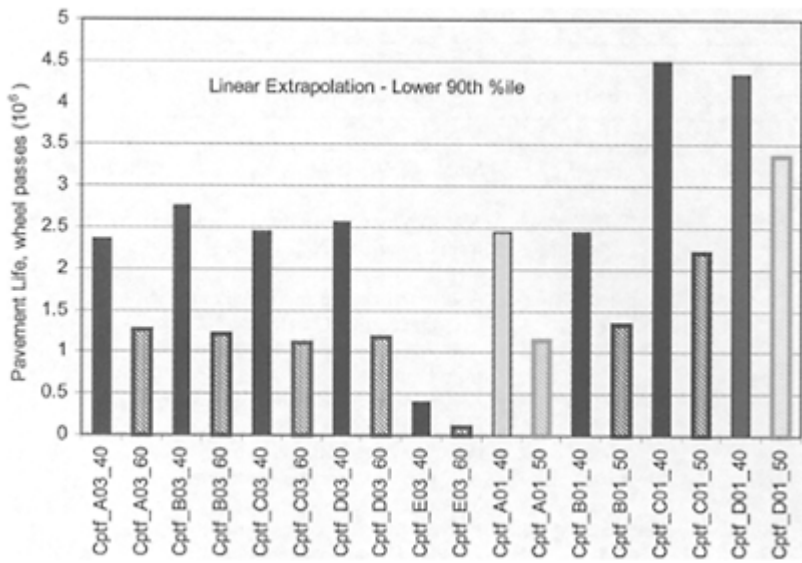


Figure 5. Lower 90th percentile lives found from linear extrapolation of VSD data to 15 mm.

to 1000 k after the initial compaction period. This linear fit was justified in terms of providing the most accurate prediction of life when reviewing sections where a VSD of 15 mm was obtained. A linear projection of rut depth is commonly observed from high speed rut depth surveys with lasers on New Zealand’s sprayed chip seal surfaced unbound granular pavements. Arnold et al (2002), recognise that three different models of rut depth progression are possible but further modelling by Arnold supports a linear rut depth progression due to the high stresses in the granular material due to no covering base material (i.e. a thin surface less than 25 mm).

The number of wheel passes when a VSD of 15 mm occurs was calculated for each wheel path at each 1 m increment. For each pavement segment approximately 9 to 12 pavement lives were obtained for each wheel path/load. It is common to assume the pavement segment has reached the end of it’s life when 10% of the area has failed. Therefore, the life calculated in the lower 90th percentile column was used as the most appropriate estimate of life for the Segment (Figure 5). Figure 5 shows the number of wheel passes obtained for all the half axle wheel loads (i.e. 40, 60 & 50 kN) as denoted in the last two digits of the values on the horizontal axis.

4 DAMAGE LAW EXPONENT

As discussed in Section 1.2 the damage law exponent is used in pavement design and deterioration modelling. The damage law exponent was calculated from lives predicted when 10% of the Segment had a VSD of 15 mm or greater using the linear method of

extrapolation (Figure 5). Results are shown in Table 3 where the Cptf_D01 Segment had the lowest damage law exponent of 1.1. This Segment used recycled crushed concrete, which as shown less sensitive to increases in mass limits compared to other aggregate types.

5 PAVEMENT STRUCTURAL NUMBER (SNP)

Falling Weight Deflectometer (FWD) tests were undertaken at regular intervals during testing. These FWD measurements were analysed to determine the pavement structural number. Pavement

Table 3. Damage law exponent calculated for each pavement segment.

ID (Table 2.1)	Exponent, <i>n</i> (Equation 2)
Cptf_A03	1.5
Cptf_B03	2.0
Cptf_C03	2.0
Cptf_D03	1.9
Cptf_E03	3.2
Cptf_A01	3.4
Cptf_B01	2.7
Cptf_C01	3.2
Cptf_D01	1.1

Structural numbers (SNP) were calculated using a formula as currently recommended by Transit New Zealand (Equation 3). This Equation has the advantage of not needing an interim analysis to back-calculate pavement layer moduli and the pavement depths are not needed. The formula was derived from FWD tests on New Zealand thin-surfaced pavements where deflections were compared to the structural number calculated from pavement layer moduli as per HDM III (Paterson 1987).

SNP=4.47+0.463log_e(d₀)+0.063log_e(d₀-d₉₀₀)^{2.2}-0.031log_e(d₀-d₁₅₀₀)³

(3)

Where:

d₀, d₉₀₀, d₁₅₀₀ = deflections in microns at 0, 900 and 1500 mm offsets, under a 40 kN FWD impact load.

6 DAMAGE LAW EXPONENT PREDICTION

The damage law exponent (Table 3) was plotted against the structural number calculated in the heavier wheel path (60 or 50 kN half axle dual tyred wheel load). The Cptf_C01 pavement segment had many surfacing repairs due to de-bonding which effected the measurements of VSD and thus was removed from the analysis. Further, as the study was for typical pavements in New Zealand the Cptf_D01 was also excluded from the dataset. Cptf_D01 was constructed with recycled crushed concrete which is currently uncommon for state highways. The Cptf_C01 pavement segment had several surfacing repairs which affected the VSD measurements and thus predictions of life/damage exponent for the two wheel paths. Results show a correlation of 0.97 when relating damage law exponent with pavement structural number (Figure 6). This result is valid for the pavements tested at CAPTIF for the particular loading and environmental conditions (i.e. moisture content). Pavement modelling utilising permanent strain test results from Repeated Load Triaxial apparatus and validated from the CAPTIF tests will be used to extend the results to other pavement types.

7 CONCLUSIONS

- Based on the pavement segment lives predicted in the heavy (50 or 60 kN) and light (40 kN) wheel path the damage law exponents calculated ranged from 1.1 to 3.4;
- The lowest damage law exponent of 1.1 was calculated for the Cptf_D01 pavement that was constructed with recycled crushed concrete;
- A damage law exponent of 3.2 was calculated for the pavement constructed with rounded aggregate that had the shortest life (Cptf_E03) which suggests the damage law exponent is related to pavement strength;

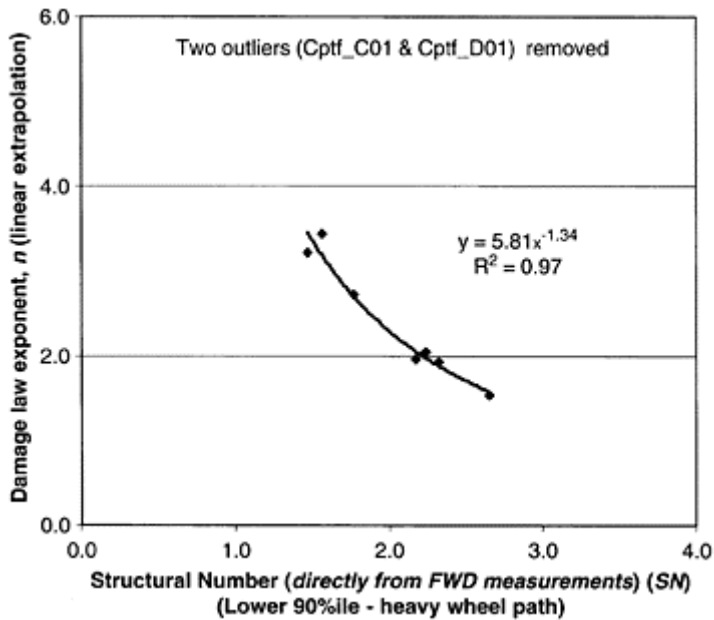


Figure 6. Relationships between lower 90 percentile structural number calculated directly from FWD measurements and damage law exponent, n .

- Relationships between structural number and pavement life are best from FWD measurements in the heavy wheel path;
- Good relationships between SN from moduli and damage law exponent were obtained if two justifiable outliers were removed from the dataset;
- Results are valid for the pavements tested which are thin-surfaced asphalt (<25 mm) over granular materials;
- Pavement modeling and further testing is required to extend the result outside those pavements and materials tested including moisture content.

REFERENCES

- AASHTO. 1962. The AASHTO Road Test. Conference Proceedings: Special Report 73. Highway Research Board, National Academy of Sciences—National Research Council. Washington DC. USA.
- Arnold, G., Dawson, A.R., Hughes, D.A.B., Robinson, D. 2002. Shakedown of Granular Materials. *Proc. 9th Int. Conf. Asphalt Pavements, Copenhagen*.
- Arnold, G., Steven, B., Alabaster, D., Fussell, A. 2004. Effect on Pavement Wear of an Increase in Mass Limits—Concluding Report. *Transfund New Zealand Research – in press*.
- de Pont, J., Steven, B., Alabaster, D., Fussell, A. 2001. Effect on pavement wear of an increase in mass limits for heavy vehicles. *Transfund New Zealand Research Report No. 207*. 55pp.
- de Pont, J., Steven, B., Alabaster, D., Fussell, A. 2002. Effect on pavement wear of an increase in mass limits for heavy vehicles—Stage 2. *Transfund New Zealand Research Report No. 231*. 50pp.
- Ioannnides, A. 1991. Theoretical Implications of the AASHTO 1986 Nondestructive Testing Method 2 for Pavement Evaluation. Transportation Research Record 1307, TRB, Washington D.C.
- Paterson, W.D.O., Watanada, T., Haarral, C.G., Dhareshwar, A.M., Bhandari, A., Tsunkawa, K., 1987. The Highway Design and Maintenance Standards Model. Vol. 1 John Hopkins University Press, Baltimore.

Behaviour of granular materials: field results versus numerical simulations

J.M.C.Neves

IST, CESUR, Technical University of Lisbon, Lisbon, Portugal

A.Gomes Correia

*Department of Civil Engineering, University of Minho, Guimarães,
Portugal*

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ABSTRACT: The purpose of this paper is to demonstrate the influence of granular material behaviour models in flexible pavement design, by comparison with measured response observed during wheel load tests on experimental pavements and with numerical simulations performed by the finite element method. The experimental pavements were instrumented during the construction phase with deformation gauges installed in bituminous, granular and soil foundation layers. The resilient behaviour of unbound granular materials was back analysed by wheel load test results, using different nonlinear elastic models, which were verified and calibrated by repeated load triaxial tests. Comparisons between the field results and numerical simulations have demonstrated the importance of a more accurate behaviour model for unbound granular materials in structural pavement analysis.

1 INTRODUCTION

The correct prediction of stresses and strains in different layers of pavements is essential for flexible pavement design. Most recent laboratory and in situ experimental research has shown that mechanical pavement modelling must require a more advanced approach, taking into account material models and response methods that are more adapted to the real behaviour of road pavements (Gomes Correia, 2001).

The main objective of this paper is to analyze how modelling the behaviour of granular materials influences structural pavement design. The paper is based on the most recent research work carried out at the Department of Civil Engineering and Architecture of the “Instituto Superior Técnico” (IST), in the Technical University of Lisbon, on the topic of laboratory testing, in situ monitoring and behaviour modelling for unbound granular materials.

For this purpose, two full-scale experimental test sections of pavement, located in Lisbon, were instrumented during the construction phase. The structural behaviour of the

pavements was observed during in situ wheel load tests. The experimental results from field tests were compared with those of the pavement modelling performed with the FENLAP2 finite element program, incorporating non-linear elastic constitutive laws for the granular material, which were verified and calibrated in repeated load triaxial tests.

2 MONITORING EXPERIMENTAL PAVEMENTS

2.1 *In situ pavements*

Two experimental test sections—test section CRIL1 and CRIL2—were instrumented during the construction of the IC 17 road (Lisbon Internal Ring Road) in 1996. The pavement structures of

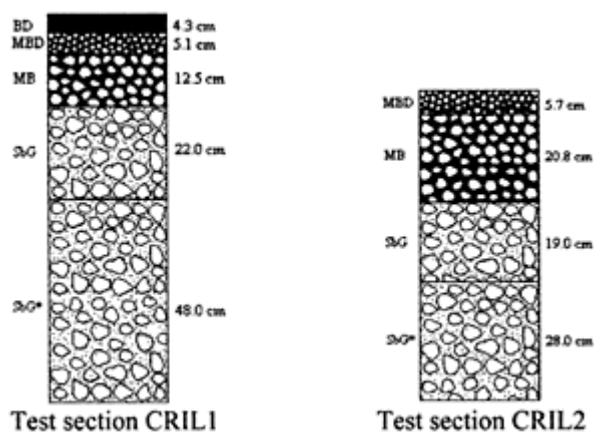


Figure 1. Pavement structures of experimental test sections.

Table 1. Compaction characteristics of the granular material and the subgrade soil.

Test section	Layer	Modified Proctor		Mean values for in situ control	
		$\rho_{d,max}$ (kg/m ³)	w_{opt} (%)	ρ_d (kg/m ³)	w (%)
CRIL1	SbG	2350	5.3	2350	3.4
	SbG*	2350	5.3	2360	2.9
	Soil	1920	13.2	1820	12.4
CRIL2	SbG	2350	5.3	2350	3.5
	SbG*	2350	5.3	2370	2.7
	Soil	1920	13.2	1840	12.6

$\rho_{d,max}$ —maximum dry density; w_{opt} —optimum moisture content;
 ρ_d —dry density; w —moisture content.

these experimental test sections are shown in Figure 1. They consist of the material components described in brief below and in more detail in Neves (2001):

- Bituminous concrete in the wearing course (BD) and in the binder course (MBD), composed of crushed basaltic aggregates (maximum aggregate size 19 mm), sand, filler and bitumen binder.
- Bituminous macadam in the base course (MB), composed of crushed limestone aggregates (maximum aggregate size 25 mm), sand, filler and bitumen binder.
- Unbound granular material in the sub-base course (SbG and SbG*). It consists of a 0/25 crushed limestone aggregate, with 9.8% of fines content and 37% of Los Angeles abrasion.
- Subgrade soil classified as SM—silty sand (ASTM D 2487) and A-7-5 (5) (AASHTO M 145). The Atterberg limits were determined as a liquid limit of 61% and a plasticity index of 28% while a fines content of 37.6% passed through No. 200 ASTM sieve (0.075 mm).

Table 1 presents the compaction characteristics of the granular material and subgrade soil. It shows the values for density and water content from the laboratory modified Proctor test and in situ control, using Troxler nuclear apparatus. The composition of the bituminous materials is indicated in Table 2. The mean values represented were determined by testing cores extracted from the experimental test sections. Bitumen binder is characterised by a penetration of 68 (0.1 mm) and softening point of 48°C.

Table 2. Composition of the bituminous materials.

Test section	Layer	ρ_{mix} (kg/m ³)	Pb (%)	Vv (%)	Vb (%)	P200 (%)
CRIL1	BD	2460	5.3	6.7	12.7	8.1
	MBD	2270	5.0	9.4	11.0	7.9
	MB	2390	4.2	4.5	9.8	6.4
CRIL2	MBD	2280	4.9	8.6	10.4	7.3
	MB	2390	4.1	4.4	8.6	6.4

ρ_{mix} —density; P_b —binder weight content; V_v —void volume content;
 V_b —binder volume content; P_{200} —percent aggregate passing a No. 200 ASTM si sieve.

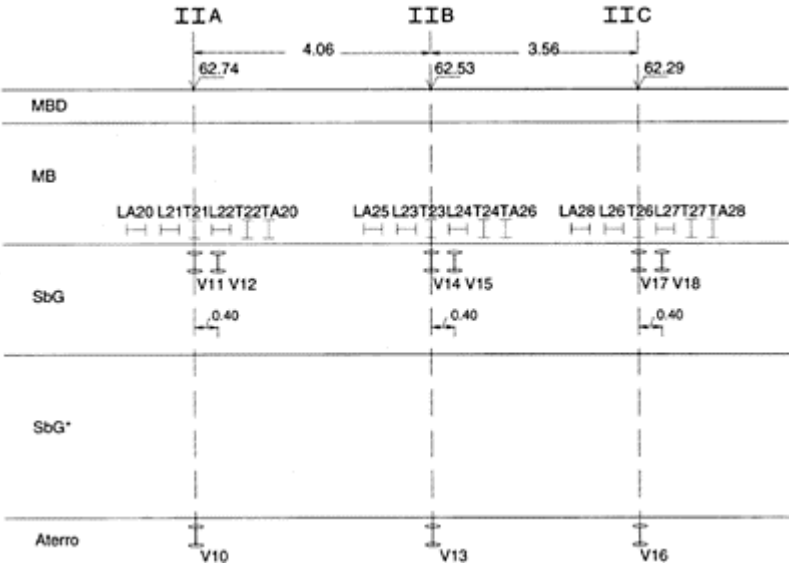


Figure 2. Instrumentation plan of test section CRIL2.

2.2 Instrumentation

The instruments used in this research were manufactured and installed by the “Laboratoire Centrale des Ponts et Chaussées” (LCPC) in Nantes (France). The instruments installed, which are described in greater detail by Ódeon et al. (1996) and Neves (2001), are strain gauges and thermocouple probes:

- Bituminous strain gauges, placed in a horizontal position at the bottom of the bituminous base layer, to measure horizontal strains in a longitudinal and transversal direction.
- Granular material and soil strain gauges, placed in a vertical position at the top of subbase layer and subgrade soil, to measure vertical strains.
- Thermocouple probes, placed in three different vertical positions, to monitor the temperature in the bituminous macadam base layer.

The gauges—model Kyowa and type KFL-30-350-C1-11—are all the same type and were positioned with a similar configuration in the two test sections. As an example, the schematic representation of the instrumentation in test section CRIL2 is shown in Figure 2.

Table 3. Characteristics of test vehicles.

Test section	CRIL1		CRIL2	
Test	I.A	I.B	II.A	II.B

Volvo Vehicle model	FL7	N720	NL 10	N10
Wheel load (kN)	34.3	34.1	36.0	31.2
Single tyre type	Toyo	Baurum	Continental	Michelin
	10.00R19.5	10.00R20	315/80R22.5	12.00R20
Tyre pressure (kPa)	689.0	865.0	826.8	826.8

2.3 Loading tests

The in situ loading test consisted of single wheel loading with the purpose of measuring the strains in the different materials by the instruments installed as described above. Temperature measurements in the bituminous materials and air were also taken. Table 3 presents the main characteristics of the heavy vehicles used in the tests. In the test section CRIL2, test II.A was carried out during the construction phase, after the construction of the bituminous base layer in bituminous macadam. All the others tests were conducted on the final pavement structures represented in Figure 1. Vehicle speed during the tests varied from 1 to 4 km/h. The loads and tyre characteristics indicated in Table 3 refer to the front vehicle axle, constituted by a single wheel at each extremity.

In all the tests, loads were applied over the centre of the gauges and strains were measured simultaneously by a data acquisition system from LCPC (Ódeon et al., 1996). During tests, bituminous macadam and air temperatures were being monitored by the thermocouple probes. The temperature of the others bituminous layers was estimated as described by Neves (2001).

3 GRANULAR MATERIAL MODELS

3.1 Resilient behaviour models

It is well known that granular materials and soils respond to traffic loading with non-linear behaviour. In the case of granular materials, Boyce (1980) proposed a non-linear elastic model suitable for describing granular material behaviour observed in repeated load triaxial tests with variable confining pressure and with measurement of radial strains. Several researchers have studied this model intensively.

In 1994, Jouve and Elhannani modified the model by proposing the following expressions to calculate the resilient volumetric (ϵ_v^r) and shear (ϵ_q^r) strains of isotropic granular materials:

$$\epsilon_v^r = \frac{p^n}{p^{n-1}} \left[\frac{1}{K_a} - \frac{\beta}{K_a} \left(\frac{q}{p} \right)^2 \right]; \quad \epsilon_q^r = \frac{p^n}{p^{n-1}} \frac{1}{3 G_a} \left(\frac{q}{p} \right) \quad (1; 2)$$

Later, Hornych et al. (1998) presented the most recent generalization of the Boyce model to the case of cross-anisotropic granular material (anisotropy between both vertical and horizontal directions). The expressions for this model are:

$$\varepsilon_v^r = \frac{p^*{}^n}{p_a^{n-1}} \left[\frac{\gamma+2}{3 K_a} + \frac{n-1}{18 G_a} (\gamma+2) \left(\frac{q^*}{p^*} \right)^2 + \frac{\gamma-1}{3 G_a} \left(\frac{q^*}{p^*} \right) \right] \quad (3)$$

$$\varepsilon_q^r = \frac{2}{3} \frac{p^*{}^n}{p_a^{n-1}} \left[\frac{\gamma-1}{3 K_a} + \frac{n-1}{18 G_a} (\gamma-1) \left(\frac{q^*}{p^*} \right)^2 + \frac{2\gamma+1}{6 G_a} \left(\frac{q^*}{p^*} \right) \right] \quad (4)$$

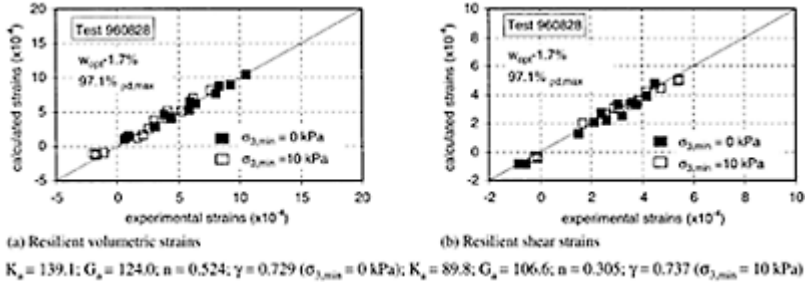


Figure 3. Calculated vs. experimental resilient shear and volumetric strains.

where

$$p^* = \frac{\gamma \sigma_1 + 2 \sigma_2}{3}; \quad q^* = \gamma \sigma_1 - \sigma_3 \quad (5)$$

6)

and γ is the coefficient of anisotropy. Parameters K_a , G_a and n depend on the water content and compacity of the material, and p_a is a constant equal to 100 kPa. Besides the elastic component, this model also incorporates a perfectly plastic part given by the Drucker-Prager criterion defined by the expression: $q = Mp + S$.

3.2 Repeated load triaxial tests

In order to achieve the validation and calibration of Boyce models, laboratory tests were carried out with the use of repeated load triaxial apparatus, originally developed at “Laboratoires des Ponts et Chaussées” (LPC) (Paute et al., 1994) and installed in the Geotechnical Laboratory of IST. The test procedure adopted was that proposed in the initial CEN triaxial standard (CEN prENV 00227413 (1995)) which consists of applying cyclic linear stress paths in p , q space (p and q are the mean normal stress and the deviator stress, respectively), characterised by the variable confining pressure (σ_3) and initial static confining stress $\sigma_{3,min}$. The tests all included an initial conditioning phase of 20,000 cycles following a stress path with a q/p of 2.0 (in order to stabilise the permanent strains and attain resilient behaviour) and a second phase composed of a series of 100 cycles characterised by ratios q/p of 1.5, 2.0 and 2.5, to study the resilient behaviour.

The test specimens had a diameter of 160 mm and a height of 320 mm and were compacted in five layers by a Kango vibrating hammer (model 900 K). The grading

curve of the specimens was composed of a mixture of four fractions, combined in appropriate percentages, obtained from the original sample: 0–4.75 mm, 4.75–9.50 mm, 9.50–19.0 mm and 19.0–31.5 mm. According to the standard test method, tests were conducted for different dry densities and water contents.

The modelling for the resilient behaviour results from repeated load triaxial tests was performed with the models represented by equations (1) to (6). An example of the goodness of fit of experimental volumetric and shear strain results is shown in Figure 3 for the anisotropic Boyce model. As can be seen in general, the right adjustment is attained for all the strains. Identical results were obtained for the isotropic Boyce model. However, a more accurate prediction is achieved when anisotropy is considered.

Analysis of the test results allowed the calibration of the isotropic and anisotropic Boyce model parameters as a function of the density and water content. Figure 4 shows an example of calibration of the anisotropic Boyce model parameters.

In the case of parameters K_a and G_a , Neves (2001) has established standard linear relationships with the following form:

$$F\left(\frac{\rho_d}{\rho_{d,\max}}; w_{\text{opt}} - w\right) = a_1 + a_2 \frac{\rho_d}{\rho_{d,\max}} + a_3 (w_{\text{opt}} - w) \quad [\text{MPa}] \quad (7)$$

where the values relating to dry density ($\rho_d/\rho_{d,\max}$) and water content ($w_{\text{opt}} - w$) are expressed as percentages. For n , β and γ , Neves (2001) suggests considering the mean values.

Table 4 contains values of the coefficients a_1 , a_2 and a_3 for the anisotropic Boyce model, where it can be observed that better agreement of the expression (see R^2 values) is obtained for K_a and for $\sigma_{3,\min}=10$ kPa. Identical conclusion was obtained for the isotropic Boyce model.

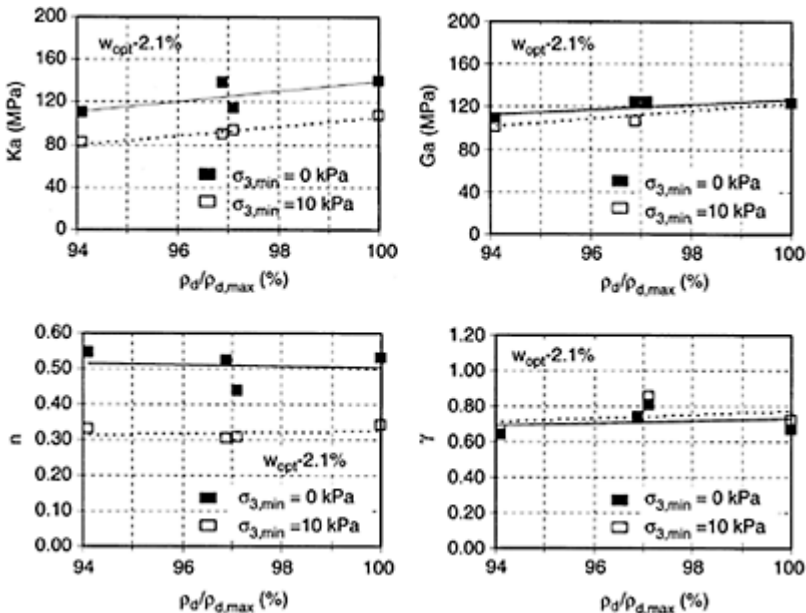


Figure 4. Example of calibration of the anisotropic Boyce model parameters.

Table 4. Coefficients of the expression (7) for the anisotropic Boyce model.

Parameters	$\sigma_{3,\min}$	a_1	a_2	a_3	R^2
K_a	0	-458.3	4.587	81.40	0.62
	10	-437.7	4.105	77.51	0.97
G_a	0	-167.0	2.307	36.94	0.63
	10	-362.0	3.381	86.53	0.72

R^2 —Correlation coefficient.

4 ANALYSIS OF FIELD RESULTS

Numerical modelling of field load tests results was accomplished with the finite element program FENLAP2, which incorporates the non-linear elastic models of the unbound granular materials under study. This program was originally created at Nottingham University and has been modified recently by Neves (2001).

Tables 5 and 6 present the parameters of pavement material models used in numerical simulations. Linear elastic parameters of subgrade soil and granular materials were obtained by back-analysis of Benkelman beam tests. The elastic stiffness of bituminous mixtures was evaluated by laboratory tests performed in the Nottingham Asphalt Tester (NAT). The parameters of Boyce models indicated in Table 6 were inferred from triaxial tests (Neves, 2001). The Drucker-Prager equation considered is $q=1.75 p+165$.

Figure 5 shows the influence lines of measured and calculated strains registered in/due to the passage of vehicle test during test I.A, in the case of test section CRIL2. Besides experimental data scattering, the comparison of both responses may conclude that the anisotropic Boyce model gives the best prediction of experimental results and linear elasticity leads to higher values. In this case, the agreement with the experimental results is less satisfactory. Identical conclusions were obtained during test I.B conducted on test section CRIL1.

Figure 6 shows the influence of the thickness of bituminous and granular layers on pavement modelling: H1 and H2 represent the thickness of bituminous and granular layers, respectively. Strain calculations were done for both test sections with the material properties and structure pavements of load tests I.A and II.A. Experimental strains obtained in all the tests are also presented. From this figure it is possible to conclude that calculated strains depend on the constitutive model of the granular material. This influence is more remarkable in the case of pavements

Table 5. Linear elastic parameters of pavement materials.

Test	Subgrade		SbG*		SbG		MB	
	E (MPa)	ν	E (MPa)	ν	E (MPa)	ν	E (MPa)	ν
I.A	81	0.40	100	0.35	200	0.35	6600	0.40
II.A	140	0.40	80	0.35	200	0.35	9000	0.35

E—Young's moduli; ν —Poisson coefficient.

Table 6. Non-linear elastic parameters of unbound granular material.

Test	Layer	Isotropic model				Anisotropic model			
		K_a (MPa)	G_a (MPa)	n	β	K_a (MPa)	G_a (MPa)	n	
I.A	SbG	126.5	201.3	0.311	0.109	111.9	133.7	0.316	0.753
	SbG*	170.2	282.8	0.311	0.109	150.6	177.0	0.316	0.753
I.B	SbG	126.5	201.3	0.311	0.109	111.9	133.7	0.316	0.753
	SbG*	126.5	201.3	0.311	0.109	111.9	133.7	0.316	0.753
II.A	SbG	117.8	184.9	0.311	0.109	104.1	125.1	0.316	0.753
	SbG*	187.6	315.4	0.311	0.109	166.1	194.3	0.316	0.753
II.B	SbG	126.5	201.3	0.311	0.109	111.9	133.7	0.316	0.753
	SbG*	126.5	201.3	0.311	0.109	111.9	133.7	0.316	0.753

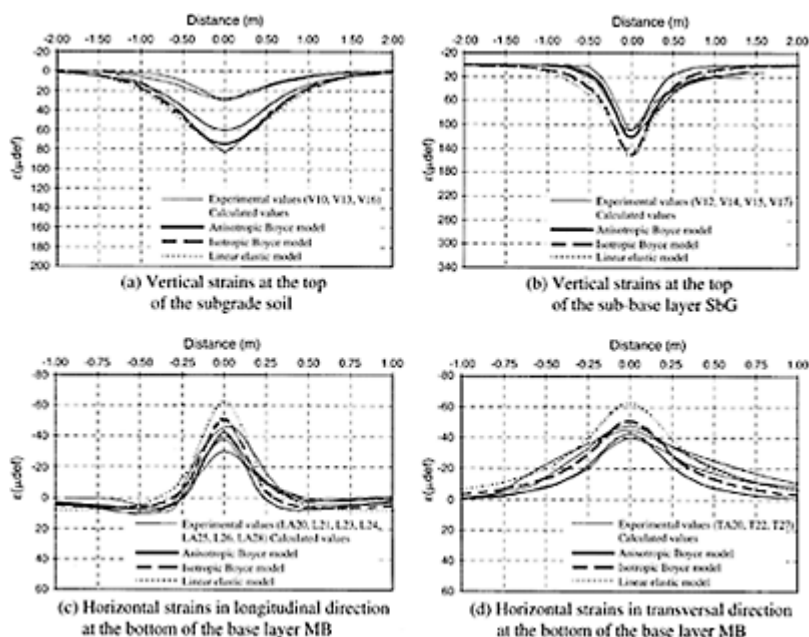


Figure 5. Influence lines of measured and calculated strains.

with thick granular layers and thin bituminous layers. Isotropic versus anisotropic unbound granular modelling is also more sensitive for vertical strain calculations at the top of the subgrade soil.

5 CONCLUSIONS

The research described in this paper shows the importance of a more realistic modelling approach for unbound granular behaviour in structural pavement analysis. Based on the comparison of field results with numerical simulations, the following main conclusions can be drawn:

- Calculated strains are very sensitive to the material model used to describe unbound granular material behaviour.
- Modelling is less satisfactory when the linear elastic behaviour of unbound granular material is used, because numerical simulations have always given the highest calculated strains.
- Non-linear Boyce models, verified and calibrated in repeated load triaxial tests, were quite appropriate in modelling the results of field load tests carried out on two experimental pavements constructed according to Portuguese standards.
- Calculated strains with the anisotropic Boyce model are very close to experimental strains.

- The thickness of layers largely influences structural pavement modelling, as the consideration of non-linearity and anisotropy becomes more important in the case of pavements with thick granular layers and thin bituminous layers.

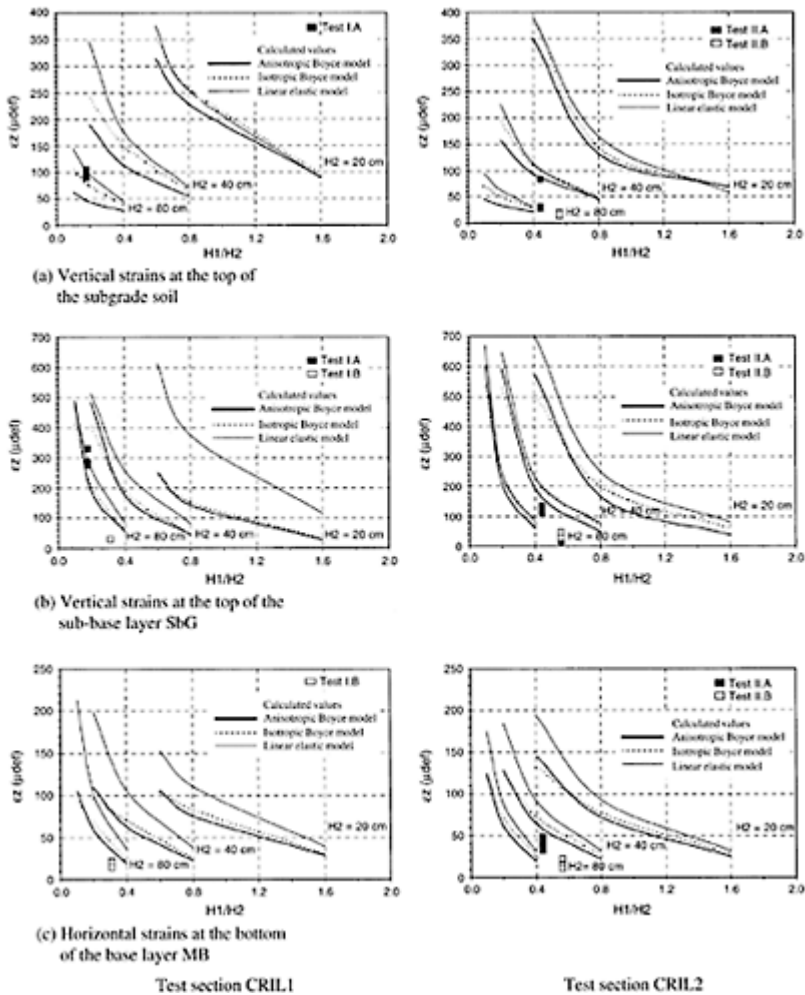


Figure 6. Influence of layer thickness on experimental pavement modelling.

REFERENCES

- Boyce, J.R. 1980. A non-linear model for the elastic behaviour of granular materials under repeated loading. *Proc. Int. Symp. on Soils under Cyclic and Transient Loading*, United Kingdom.
- CEN prENV 00227413. 1995. Unbound and hydraulic bound mixtures for roads—test methods—cyclic load triaxial test. Draft, CEN/TC227/WG4/TG2, Brussels, Belgium.
- Gomes Correia, A. 2001. Soil Mechanics in Routine and Advanced Pavement and Rail Track Rational Design. In Gomes Correia & Brandl (editors). Rotterdam: Balkema.
- Hornych, P., Kazai, A. & Piau, J.M. 1998. Study of the resilient behaviour of unbound granular materials. *Proc. of the 5th International Conference on Bearing Capacity of Roads and Airfields*. Trondheim, Norway: 1277–1287.
- Jouve, P. & Elhannani, M. 1994. Application des modèles non linéaires au calcul des chaussées souples. *Bulletin de Liaison des LPC*, 190:39–55.
- Neves, J.M.C. 2001. Contribution to the Structural Behaviour Modelling of Flexible Road Pavements (in Portuguese). PhD Thesis. Technical University of Lisbon.
- Odéon, H., Kerzrého, J.P., Kobisch, R. & Paute, J.L. 1996. Experiments with three unbound granular materials on the LCPC circular test track. In A. Gomes Correia (ed.). *Flexible Pavements. Proc. of the European Symposium Euroflex*. Lisbon, Portugal. Rotterdam: Balkema.
- Paute, J.L., Marignier, J. & Vidal, B. 1994. Le triaxial a chargements répétés LPC pour l'étude des graves non traitées. *Bulletin de Liaison des LPC*, 190.

Test of the influence from mica and LWA on permanent deformations and calculation of the elastic and permanent response under HVS testing

P.Ekdahl

Ramböll RST, Malmö, Sweden

J.Hansson

Chalmers University of Technology, Gothenburg, Sweden

A.Huvstig & H.Thorén

Swedish National Road Administration, Western Region, Gothenburg, Sweden

Pavements Unbound—Dawson (ed.)

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ABSTRACT: A major distress problem on roads is rutting. Especially for roads with thin asphalt layers this is a typical problem when permanent deformation occurs in the unbound material under loads from heavy vehicles.

Two frequent problems for the SNRA are addressed in this paper. The first is that high mica content in the unbound materials is believed to have a strong negative effect on the plastic deformations. The second problem is that roads with a light weight clay aggregate (LWA) as filling displays frequent problems with rutting.

These two problems were addressed at a full-scale test with the Swedish/Finnish HVS during the summer of 2003. The test was conducted in connection with a large motorway project (E6). Four test sections with varying mica content in the unbound base and four sections with light weight clay aggregate (LWA) were constructed for HVS testing. Elastic and permanent response were measured under the HVS as well as extensive laboratory material testing and FWD loading.

The project was subject to an international workshop in September 2003 where invited researchers compared response models to the test results. The major findings from this workshop are presented in the paper.

1 INTRODUCTION

Road design and deterioration of roads is a difficult and complex subject. One of the reasons being that many of the input values are difficult to measure or predict. One example on this is the variation in loads from heavy vehicles (axle loads and tire pressures) and their distribution in time. Another example is the influence from the climate, such as frost/thaw development and the moisture content inside the road. A third example is the large variations in material properties. These and many other input data make the prediction of road deterioration quite complicated.

In order to judge about the future function of a road, a necessary prerequisite is that the calculations are grounded on a reliable deterioration and response models. The most common way for road design in Sweden, up to year 2000, was to choose a suitable design from the catalogue "ROAD 94" (the former Swedish design code). The design model, which was used as a basis for this catalogue, was grounded on a multi-layer model with linear elastic materials. Such models do not take into account the viscous and plastic behavior of the materials in a road structure. A certain degree of consideration to this is taken with help of some empirical correction factors, and also by choosing modified values for the elastic modulus. These empirical corrections results in a fairly good agreement with earlier experience from the road network in the country. However, the correction factors are depending on the road network and road types, to which they are calibrated and have to adjusted before being used elsewhere.

Modern technique for calculation with 3-dimensional finite element method, and powerful computers, has created possibilities to make a more realistic simulation of the behaviour of a road structure under traffic load and frost/thaw conditions etc. Sweden has recently developed a new mechanistic road design model, based on 3D finite element modelling.

With the purpose of developing models for the calculation of the permanent deformations in unbound materials, a test with the Swedish-Finnish HVS was arranged on a site for a large motorway project during the summer of 2003. The main reason for this was frequent problems with rutting in the unbound layers depending on the mica content in the aggregate, see Chapter 3. Another reason was test the use of light weight clay aggregate (LWA) in road structures.

In order to cooperate with other countries about the calculation of elastic and permanent responses in unbound materials, an international workshop was arranged in Gothenburg, Sweden in September 2003. Researchers from different countries were invited to use the data to calculate the expected material response. The goal for this workshop was to discuss the results from these calculations in comparison with real results from the test. This work should be continued, firstly with a dialogue on the 6th International Symposium on Pavements Unbound in July 2004.

2 GEOLOGICAL BACKGROUND FOR THE TESTS OF UNBOUND MATERIALS

The test site was built in order to test the unbound materials with mica and light weight materials. Road design normally includes frost heave, but at this test site, with a structure on soft and wet clays, this was not necessary. This test was performed at a constant temperature of 10°C.

Half of the test site was built with one certain sub base layer (aggregate size 0–90 mm) and four different base layers (aggregate size 0–32 mm) with different aggregate materials. The aggregate distribution was in accordance with the Swedish standard. The base materials come from crushed rock come from different sites in western Sweden and they have an equal grain size distribution. The different rock materials consisted mainly of quartz, feldspar, amphiboles and biotite. There were no carbonates or harmful fines in these materials. The differences were mainly in mica content. The mica content varied between 6 and 34%.

On the other half of the test site, light weight materials under the base layers were tested. The base layers consisted of base material from test site no 1, with 14% mica content. The light weight material consisted of expanded clay. For these sections the issue to evaluate the necessary thickness of the unbound layers.

3 DESCRIPTION OF THE TEST SITE

The test site consists of eight test sections situated about 100 km north of Gothenburg along road E6 towards Oslo. Each test section was between 12 and 15 meters long. Sections 1–4 were designed with the purpose to determine the effect on pavement response from varying mica content. These four sections were designed with identical layer thicknesses, but with varying mica content in the unbound base. The degree of compaction was similar for all sections.

Section 5–8 had the purpose to test the use of light weight clay aggregate (LWA) with varying thicknesses of unbound base. Section 8 also had a stabilized and reinforced plate with light weight clay aggregate (LLP).

Each test section was equipped with instruments for registration of pavement response and moisture content. In Sections 1–4 gauges (EMU coils) were installed for registration of vertical strains and deformations in the upper and lower half of the unbound base. In the middle of these layers were also gauges (Geokon 3500) installed for registration of vertical stresses.

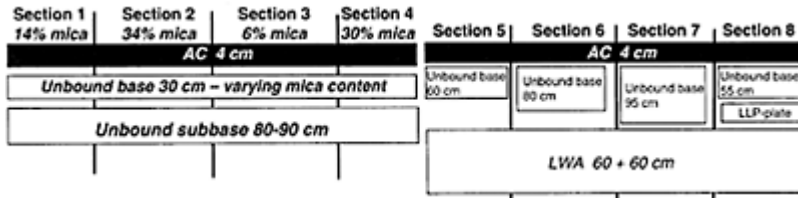


Figure 1. Road design Section 1–8.

Sections 5–7 were installed with gauges for vertical deformations in both the unbound base and in the LWA. Vertical stresses were measured 7.5 cm down in the unbound base, on top of the LWA and in the middle of the LWA layer. Single LVDT gauges were installed for comparison with the EMU coils in the LWA (they proved to display very similar deformations). Section 8 was instrumented similar to Section 5–7, but with the addition of strain gauges mounted on the steel reinforcement of the LLP.

In total 60 EMU coils, 4 LVDT's, 20 soil pressure cells and 8 strains gauges were used in the eight test sections.

4 DESCRIPTION OF THE TESTS

The materials used in the test sections were tested in numerous ways. Material properties such as grading curve, moisture content, mineral content, density, ball mill and Micro Deval were registered. Triaxial test were performed for the unbound base materials.

During the construction phase loading tests, such as static plate loading, portable falling weight and FWD were used to test the structural properties for all section. Furthermore the degree of compaction was registered at all stages. All surface levels were carefully registered during construction in order to get hold of variations in layer thickness.

After construction the Heavy Vehicle Simulator (HVS) from the Swedish National Road Research Institute (VTI) was used for loading tests on each section. 20000 pre-loadings were performed. Thereafter 80000 loading were applied on each section with an 80 kN axle. All HVS tests were performed at 10°C.

At every 20000 loadings the material response was measured for the 80 kN wheel load during four passages. At the same time three cross sectional surface profiles were measured in order to determine the total permanent deformation. Performed measurements can be divided into the following main groups:

- Material properties before and during construction (e.g. grading curves, triaxial tests, moisture content).
- Field tests during construction (e.g. static plate loading, degree of compaction).
- Quality control of levels.
- Structural strength before HVS-loading (FWD).
- Material response during HVS-loading (deformations, stresses and strains).

- Total deformation at surface during and after HVS-loading.
- Ground penetrating radar to find depth to rock.
- Material properties after HVS.

The results from all these tests are gathered in a database which is available from the Swedish National Road Administration for those interested.

Table 1. HVS load specifications.

	Pre-loading	Response loading
Wheel load	30 kN	80 kN
Tire pressure	1.0 MPa	1.0 MPa
Tire type	Dual tire	Dual tire
Lateral displacement	Even +25 cm from centerline	VTI standard (table 2)
No of passages	20 000	app. 80 000

Table 2. Lateral displacement of loading wheel.

Displacement (cm)	–25	–20	–15	–10	–5	0	5	10	15	20	25
Distribution (%)	0,4	1,6	6	12	18	24	18	12	6	1,6	0,4

5 RESULTS FROM THE CALCULATIONS

5.1 Background

Invited researches had the opportunity to predict stresses and strains (elastic and permanent) in a structure equal to the test road. As input data to the modeling, they received results from triaxial tests on the base layer material, plate loading and FWD from performed field tests. The task was to predict rutting on the road surface after 100000 loadings, stress and elastic strain at two levels of the unbound base layer and the permanent deformation in the unbound base layer after 100000 loadings. The predictions were compared with the result from the HVS-test. The modeling and HVS-test were discussed in an international workshop (Validation of design models and test methods for road deterioration) in Ellös, Sweden 11–12 September 2003. This workshop had 15 participants from USA, France, Norway, Finland, Iceland and Sweden.

The participants of the calculation project were Prof. Sigurdur Erlingsson, University of Iceland, Iceland, Dr. Pierre Hornych, Laboratoire Central des Ponts et Chaussées Route de Bouaye (LCPC) and Prof. Charles Schwartz, University of Maryland.

5.2 Predictions of Professor Charles Schwartz

For the stress and elastic strain calculations (structure analysis) the program Kenlayer was used. E-moduli from back calculation of FWD-results were used as in data for asphalt layer, subbase layer and subgrade. For the base layer, a stress dependent resilient modulus E was used (Eq. 1)

$$E = K \left(\frac{3p_{\max}}{p_a} \right)^n \quad (1)$$

where: $p_{\max}$ =sum of principal stresses; p_a =reference pressure equal to 100 kPa; K and n =model parameters received from triaxial tests.

Stress and resilient strain calculations were also done with E-modulus received from back-calculated FWD-results. There was small difference between the results and because of that, calculation results from using back-calculated E-module was used instead. Stress and elastic strain in different part of the structure were then calculated. For the permanent deformation calculation, Eq. 2 was used for the base layer.

$$\frac{\epsilon_p}{\epsilon_r} = k_1 \cdot N^{k_2} \quad (2)$$

where: ϵ_p =plastic strain; ϵ_r =resilient strain; N =number of cycles; k_1 and k_2 =constants estimated from the triaxial test.

The loading levels, used for the Eq. 2, were 620 and 820 kPa. Due to that several loading levels were put on the same specimen, k_1 and k_2 had to be adjusted for the earlier loadings. The permanent deformation in the sub base layer was then calculated by using Eq. 2 and ϵ_r from the structure analysis.

Permanent deformation in the asphalt layer was calculated by using a model, including parameters such elastic strain, temperature, number of load repetitions and regression coefficients, recieved from earlier projects. For the subbase layer and subgrade, models were used, including parameters such as resilient strain, water content, E-moduli and stress levels.

5.3 Predictions of Professor Sigurdur Erlingsson

For the stress and elastic strain calculations (structure analysis), a FEM program was used. E-moduli from back calculation of FWD-results were used as in data for asphalt layer, subbase layer and subgrade. For the base layer, a stress dependent resilient modulus M_r was used (Eq. 3) (k - θ model).

$$M_r = k_1 \cdot \theta^{k_2} \quad (3)$$

where: k_1 and k_2 =regression parameters from the triaxial test; and θ =stress invariant.

For the permanent deformation calculations of the base layer, triaxial test results of permanent deformation were used and Eq. 4 modelled the test results.

$$\varepsilon_p(N) = \varepsilon_0 \cdot e^{-\left(\frac{\rho}{N}\right)^\beta} \quad (4)$$

where: N =number of loadings; ρ and β =regression parameters.

Assuming that the relation between elastic and permanent strain is the same in laboratory and field, Eq. 5 is obtained.

$$\frac{\varepsilon_p^{lab}}{\varepsilon_r^{lab}} = \frac{\varepsilon_p^{field}}{\varepsilon_r^{field}} \quad (5)$$

where: ε_r =elastic (resilient) strain; and ε_p =permanent strain

Eq. 4 and Eq. 5 gives Eq. 6, where the permanent strain in field could be calculated by using calculated results from elastic strain in field and measured elastic strain in laboratory.

$$\varepsilon_p^{field}(N) = \frac{\varepsilon_0}{\varepsilon_r^{lab}} \cdot e^{-\left(\frac{\rho}{N}\right)^\beta} \cdot \varepsilon_r^{field} \quad (6)$$

5.4 Predictions of Dr. Pierre Horny ch

The FEM program CESAR-LCPC was used for the calculation of stress. As in data to the modelling, linear elastic modulus was used, for the asphalt layer and subgrade. These modulus' were given from back-calculation of FWD-test results. It was assumed that the base layer and subbase layer had the same properties and their resilient modulus was given by calibrating the k - θ model (Eq. 7) by using triax test results.

$$E = K \left(\frac{3P_{max}}{P_a} \right)^n \quad (7)$$

where: E =resilient modulus. K , n =model parameters; P_{max} =maximum value of the mean stress $p=(\sigma_1+\sigma_2+\sigma_3)/3$; P_a =reference pressure equal to 100 kPa.

For the permanent deformation calculations an empirical relationship (Eq. 8), describing the variation of permanent axial strains with the number of load cycles, and the maximum applied cyclic stresses. The relationship is the following:

$$\varepsilon_1^p(N) = \varepsilon_1^{p0} [1 - N]^{-B} \cdot \left(\frac{L_{max}}{P_a} \right)^n \cdot \frac{1}{m + \frac{s}{P_{max}} - \frac{q_{max}}{P_{max}}} \quad (8)$$

where: ε_1^p = permanent axial strain; ε_1^{p0} =premenant axial strain; N =number of load cycles; p_{max} and q_{max} =maximum values of the mean normal stress p and deviatoric stress q .

$$L_{max} = \sqrt{p_{max}^2 + q_{max}^2} \quad (9)$$

and P_a =reference pressure equal to 100 kPa; ε_1^{p0} , B and n =model parameters; m and s =parameters of the failure line of the material, of equation $q=m.p+s$; (from experience, $m=2.5$ to 2.6 and $s=0$ kPa).

The proposed permanent deformation relationship is the product of a function of the number of load cycles

$$f(N) = \varepsilon_1^{r0} [1 - N]^{-B} \text{ and a function of the applied stresses } g(p_{max}, q_{max}). \quad (10)$$

The two functions are generally fitted separately; first, the function $f(N)$ is determined using the results obtained for the first stress level (when several stress levels are successively applied in the same test), and then, the function $g(p_{max}, q_{max})$ is fitted to the values of ε_{1p} obtained after a fixed number of load cycles (typically 10^5 to 10^6), for the different stress levels. The function $f(N)$ was adjusted on the 5th stress level of the triaxial test. In a second step, the stress level function $g(p_{max}, q_{max})$ was adjusted on stress levels 5 to 8, using the values of ε_{1p} obtained after 100 000 cycles; for stress levels 7 and 8, because only 10000 cycles were effectively applied, the values were extrapolated to 100 000 cycles, using the $f(N)$ function. By using stresses from the FEM calculations, the permanent strain could be predicted.

5.5 Summary results of calculations

The results of the calculations of stresses, elastic strain and permanent deformation are presented in Table 3 and 4 respectively for one test section together with the measurement results. Results of

Table 3. Calculated and measured stresses in kPa at two levels of the base layer.

Level [mm]	Schwartz	Erlingsson	Hornych	Measurements
115 mm	270	270	386	430
265 mm	150	170	254	260

Table 4. Calculated and measured strains in microstrain at two levels of the base layer.

Level [mm]	Schwartz	Erlingsson	Hornych	Measurements
115 mm	1400	1800	–	3500
265 mm	1200	1700	–	2200

the stresses and elastic strains are shown for two levels in the road structure, 115 mm and 265 mm below the surface of the asphalt layer. The permanent deformation results are displayed in Table 5 for the base layer and as the total rutting on surface of the asphalt layer. Due to misunderstanding of how triaxial results was present, the predicted permanent deformation in the base layer should be 3.3 times higher.

5.6 Comments

All calculations were performed based on the designed layer thickness. However noticeable variations in layer thickness were later registered for all layers. Professor Hornych did not use elastic strain in the prediction of permanent strain and because of that no elastic strain data is present. Depending on the results from this workshop, supplementary triaxial tests will be performed and the analyses will continue in several long-term projects and in cooperation with the participant researchers.

6 EFFECTS FROM MICA AND PERFORMANCE OF LWA

The first analyses have not given any clear answers of how the mica content influence on the permanent deformations. The variation in total permanent deformation is such that other parameters probably is more decisive. Figure 2 displays that a major part of the deformation derives from other layers than the unbound base which complicates the analysis somewhat. However the results indicate that a mica content lower less than the maximum 34% tested does not mean a largely increased risk for permanent deformations. Very low contents of mica may be performing better than the other base materials, but further studies are necessary before any final conclusions may be drawn.

Table 5. Calculated and measured permanent deformation in mm in base layer road surface after 100000 loadings.

	Level	Schwartz	Erlingsson	Hornych	Measurements
Base layer		3.5	1.4	3.2	6.3
Surface		5.7	6.7	5.1	13.9
Corrected base layer		11.6	4.6	10.6	6.3
Corrected surface		13.8	9.9	12.5	13.9

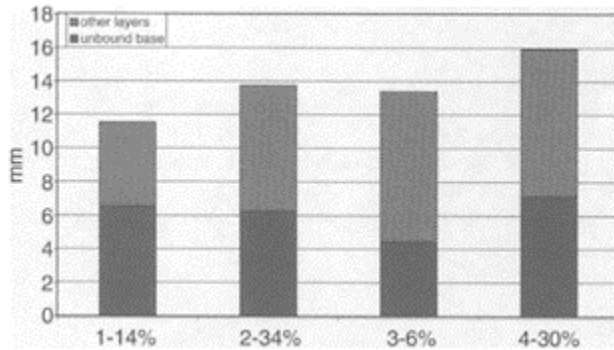


Figure 2. Permanent deformations area 1- after 100 000 HVS loadings.

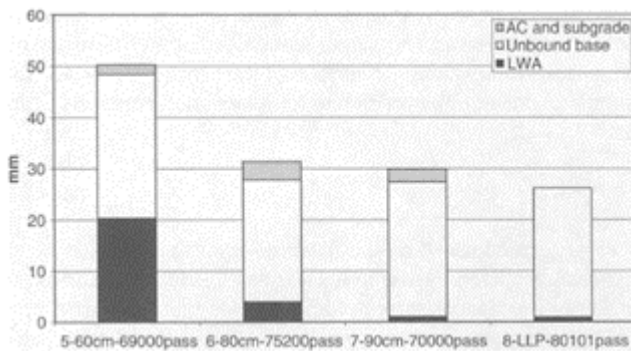


Figure 3. Permanent deformations area 5-8 after app 75 000 HVS loadings.

For the LWA Sections (5-8) there was a clear indication of that certain stress or strain levels must be achieved in order to reduce the deformation in the LWA layer. In these tests a layer thickness of between 80 and 90 cm seemed to results in response levels leading to permanent deformations in a reasonable zone. These layer thicknesses correspond to approximately 50 kPa on top of the LWA. No horizontal responses were measured, however they will be calculated in the further work since they are assumed to be crucial for the performance of LWA. The tests also indicate that a LLP plate greatly reduces the deformations in LWA since it reduces the stresses below the plate.

7 CONCLUSION

The results from the workshop show that the prediction of elastic and permanent deformations in unbound materials can be made with a certain degree of uncertainty.

However, all predictions of the permanent deformations have underestimated the real result. The variation in real layer thicknesses may have affected the study to a noticeable degree.

The effect from varying mica content did not seem to be of such a magnitude in this test that it greatly influenced on the permanent deformations, as long as it is less than about 35%.

The deformations in LWA are noticeable. It is probably necessary to reduce the stress level to about 50 kPa to get a stabile behavior in this material.

The above conclusions are drawn from the preliminary results and with limited analysis efforts. The work now steps into a phase of deeper analysis in a Phd-project and within the work on the Swedish design code.

All measurement results are gathered in a data base free of use for those interested.

REFERENCES

- Hansson (2004)*—Hansson J., “Documentation from international workshop for calculation of response at test site E6”, Gothenburg, September 2003
- Ekdahl et al (2004)*—Ekdahl P., Enocksson C-G., Hansson J., Wiman L-G., “Report from test site E6—description and test results”, SNRA West, 2004, Swedish
- Enocksson (2004)*—Enocksson C-G., “Description of test site”, VTI transport forum, 2004

Influence of spring thaw on pavement rutting

V. Janoo & S. Shoop

*U.S. Army Engineer Research and Development Center, Cold Regions
Research and Engineering
Laboratory, Hanover, New Hampshire, USA*

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ABSTRACT: Pavements in cold regions are subjected annual freeze-thaw cycles. During the spring thaw, pavement structures can become saturated and can deform considerably under normal loading. A full-scale study was conducted in the Frost Effects Research Facility (FERF) to quantify the rate and amount of rutting of four pavement structures during thaw. Two of the sections were full-depth asphalt concrete over subgrade, and the remaining two were thin asphalt concrete over base/subbase layers over subgrade. The test sections were first frozen and then thawed. During the thaw they were loaded using a standard 80-kN axle load. Surface ruts and falling weight deflection (FWD) measurements were taken periodically. The falling weight data were used to compute the Impulse Stiffness Modulus (ISM). The measured rut depth was found to be a function of the number of load applications, the pavement cross section, the thaw depth and the ISM. A multiple regression equation was developed to predict rut depth as a function of the ISM and load applications.

1 INTRODUCTION

Pavements in cold regions are subjected to annual freeze-thaw cycles. During the winter the strength of pavement sections increase as much as ten-fold from ice formation and temporary cementation of the soil particles. However, as the pavement thaws, the ice converts to excess water in the soil structure and creates a weakened state. Normal loads applied during the thaw weakening can lead to severe permanent deformation, reflected as surface rutting.

With the advance of mechanistic pavement design and evaluation methods, deterioration models for predicting cracking and rutting have been pushed to the forefront. Although there are existing models for asphalt concrete rutting and cracking, these models were developed for the hot summer months or for “normal-condition” trafficking. These models do not apply to the deformation of the base and subgrade during thaw.

Full-scale test sections of thawing pavement structures have been used to study the effects of thaw on pavement deterioration, most notable are studies by Saarelainen et al.

(1999) and Zhang and Macdonald (2001). The work reported here serves to add to those datasets and proposes modeling the pavement deterioration during spring thaw in terms of rut depth as a function of the impulse stiffness modulus (ISM).

Several full-scale test sections were built in the Frost Effects Research Facility (FERF) in the late 1980s to quantify the rate and amount of rutting under controlled thawing conditions. The pavement structures were subjected to five freeze-thaw cycles. Loading was applied in the third and fifth thaw cycles. The results from the first and second thaw cycles were reported by Janoo and Berg (1990a, b). Their main conclusions were that the clay subgrade was weakened (with no loading) by 50–60% during the thaw cycle, and that the thaw depth could be predicted from the changes in the FWD deflections. The results for the third to fifth freeze-thaw cycles have never been reported. The data and analytical results for the third thaw cycle, where the test sections were subjected to traffic loading, are presented here.

2 DESCRIPTION OF TEST SECTIONS

Four test sections (TS1, TS2, TS3 and TS4), 7.6 m (25 ft) long, 6 m (20 ft) wide and 2.45 m (8 ft) deep were constructed in the FERG located at the Cold Regions Research and Engineering Laboratory (CRREL) in Hanover, New Hampshire, USA. The FERG is a 2700-m² environmentally controlled building.

Test sections TS1 and TS2 were constructed as full-depth pavements using the Corps of Engineers (COE) mechanistic design process for a Design Index of 3, which is equivalent to 60000 passes of an 80-kN ESAL (Equivalent Single Axle Load) over 20 years. For the AC (Asphalt Concrete) and subgrade layers, design moduli of 2760 and 30 MPa, respectively, were used. For TS1 this turned out to be 152 mm of AC over the subgrade. In TS2 the drainage layer requirement of a minimum of 102 mm of free-draining material was added under the full-depth (152-mm) AC layer.

Test sections TS3 and TS4 were constructed using the COE Reduced Subgrade Strength Method, outlined in the Army Design Manual, *TM 5-882-13* (U.S. Army 1994). The pavement structures were designed for the same Design Index (DI) of 3. This requirement led to a total pavement thickness of 432 mm. TS3 was constructed of 51 mm of AC over 178 mm of base over 203 mm of gravel subbase over the subgrade. TS4 consisted of 51 mm of AC over 254 mm of base over 127 mm of sand subbase over the subgrade. The layer thicknesses for all the test sections are shown in Figure 1.

The test sections were instrumented with thermocouples and resistivity probes to track the temperature and the freezing front. The thermocouples were placed 152 mm apart in the vertical direction in the subgrade and 51 mm apart in the AC, base and subbase layers. The resistivity probes were made with copper rings spaced 51 mm apart on a solid PVC rod. The resistivity of natural soil water changes from 20 to 100k Ω when frozen. Resistivity probes are useful for locating the thaw depth when ground conditions are nearly isothermal. More detailed information on the resistivity probes can be found in Atkins (1979). Additional details on the test sections can be found in Janoo and Berg (1990a, b).

3 DESCRIPTION OF TEST MATERIALS

The subgrade was classified as a CH using the Unified Soil Classification System (USCS) or an A-7-6 using the AASHTO designation. The base used in the test sections was a crushed, bank-run gravel and was classified as GP or A-1-a. The percentage passing the 0.074-mm sieve was less

Test Section 1	Test Section 2	Test Section 3	Test Section 4
152 mm AC	152 mm AC	51 mm AC	51 mm AC
	104 mm Base	178 mm Base	254 mm Base
		203 mm Gravel Subbase	127 mm Sand Subbase
1.42 m Subgrade	1.32 m Subgrade	1.14 m Subgrade	1.14 m Subgrade

Figure 1. Cross sections of test sections.

than 10% for both subbase and base. The subbase materials were classified as SW-SC (labeled gravel subbase) and SW (labeled sand subbase), with an AASHTO designation of A-2-4 for both. The percentages passing the 0.0740 mm sieve were 7 and 8%, respectively. The grain size distributions for the subgrade, base, and subbase soils are shown in Figure 2. Additional properties (specific gravity, liquid limit, plasticity index, maximum density and optimum moisture content) of the clay, base and subbase are presented in Table 1.

The maximum density-moisture relationships were obtained from the COE CE-55 maximum density-optimum moisture tests. This test is very similar to the ASTM D-1557 and AASHTO T-180 Method B compaction tests. The single difference between the CE-55 test and the other tests is that 55 and not 56 blows are used to compact the layers.

The clay subgrade was placed in lifts that averaged 160 mm thick at the lower depths and 250 mm thick closer to the surface. The subbase and base course layers were placed in single lifts. The subbase was compacted with one pass of a 9070-kg vibratory roller. The base was compacted with the same roller until there was no change in density with increased roller passes.

4 TESTING PROGRAM

During the third freeze-thaw cycle, the test sections were frozen at a rate of approximately 25 mm per day using cooling panels on their surfaces. Freezing was continued until the frost depth was

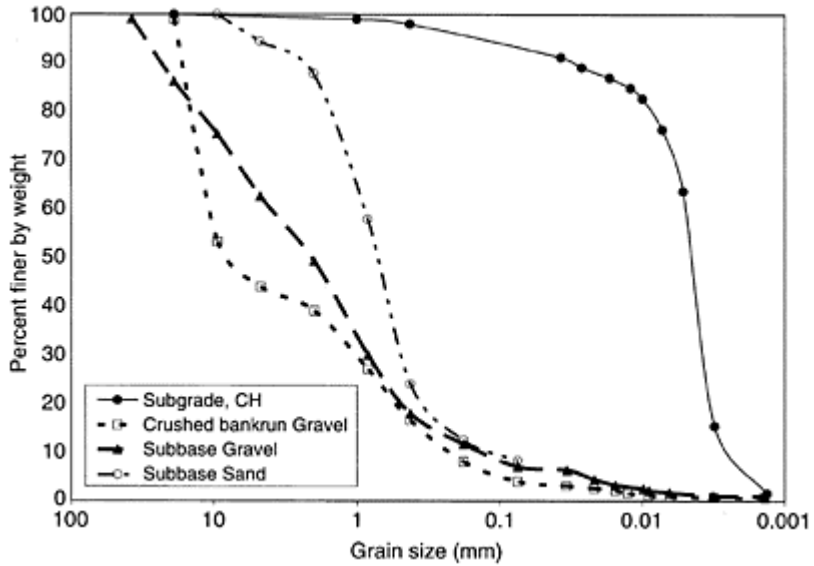


Figure 2. Gradation curves for base, subbase and subgrade.

Table 1. Properties of subgrade, base and subbase materials.

	Soil classification		Specific gravity	Liquid limit	Plasticity index	Max density	Optimum moisture
Material	uses	ASSHTO	(G _s)	(LL)	(PL)	(g/cm ³)	(%)
Subgrade	CH	A-7-6	2.79	64	36	1.70	21.0
Base	GP	A-1-a	2.80	0	0	2.29	5.5
Gravel subbase	SW-SC	A-2-4	2.80	26	9	2.26	6.5
Sand subbase	SW	A-2-4	NA	NA	NA	1.92	13.3

approximately 1.4 m. Then the panels were removed, and the test sections were allowed to thaw. The ambient temperature of the facility was maintained at around 16°C.

During the thaw process, the test sections were subjected to trafficking. Traffic was applied using a rear dual-wheel truck (Fig. 3). The center-to-center distance between the dual tires was 1.85 m (73 in.), and the width of the dual tires was 508 mm (20 in.). The metal pointer on the truck was used as guide for trafficking the test section. The traffic was wandered using the line pattern along the side of the truck as shown in Figure 3. Traffic was started with the outer wheel at 737 mm (29 in.) from the west wall, and wander was incremented in 305-, 152-, 152-, and 305-mm (12-, 6-, 6-, and 12-in.) increments, for a total wander width of 914 mm (36 in.). Traffic loading was back and forth for a total of two load passes. The same sequence of loading was applied to the first, third and fourth wander increments. On the second wander increment, four passes were applied. Therefore, for one complete loading sequence, 12 passes were applied. The traffic pattern produced the distribution shown in Figure 4, which is close to the intended Gaussian distribution. The truck speed was between 5 and 8 km/hr (3 and 5 mph).

Prior to every trafficking sequence, FWD tests were conducted across the test sections. Eight FWD measurements were taken in each test section. At the end of the trafficking process, surface

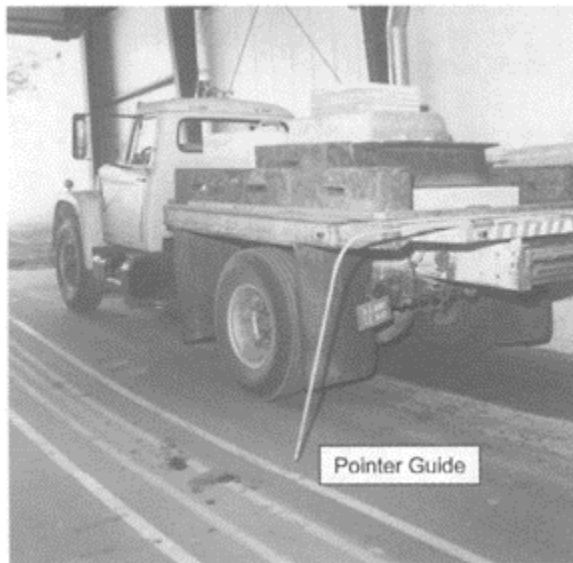


Figure 3. Load truck used for trafficking tests.

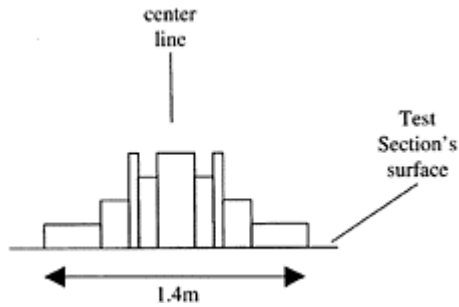


Figure 4. Traffic distribution used during load testing. The vertical scale indicates the relative number of passes at that location on the test section.

rut depths were measured in trafficked and untrafficked areas. Transverse rut depth measurements, spaced 300 mm apart, were taken along two lines in each test section. The lines were located 1.83 m from each end of the 6.1-m-square test section. A set of reference rut depth measurements were taken prior to the third freezing cycle of the test sections.

5 DATA COLLECTION AND ANALYSIS

The rate and magnitude of permanent deformation are a function of the pavement structure, loading and thaw depth.

5.1 *Determination of freeze—thaw depths*

For this paper, frost and thaw depth progression was determined from the subsurface temperature measurements. The transition between frozen and thawed was assumed to occur when the temperature of the subsurface was at 0°C. The frost depths in the test sections were found to range from 787 to 1422 mm from the AC surface. In addition, at the end of the freezing process, core samples were taken to determine the frost depth (Table 2). In TS1 the subgrade was frozen to a depth of 940 mm. At depths from 940–1219 mm, the samples were wet (not frozen) and showed excessive air voids. Similar results were found in TS3, where the clay subgrade was frozen to a depth of 787 mm. At depths from 787–1422 mm, the samples were wet (not fully frozen) and showed excessive air voids.

The thaw depths as a function of the days into thaw are presented in Figure 5. The thaw rate ranged between 37 and 40 mm per day. The sections were completely thawed after 28 days of thaw.

Table 2. Frost penetration depths based on coring.

Test section	Frost depth (mm)
TS1	940–1219 ¹
TS2	1219
TS3	787–1422 ²
TS4	1245

¹Sample in this depth range were partially wet (not frozen solidly) and had large air voids.

²Large air voids were found in the cores at this depth (not totally frozen).

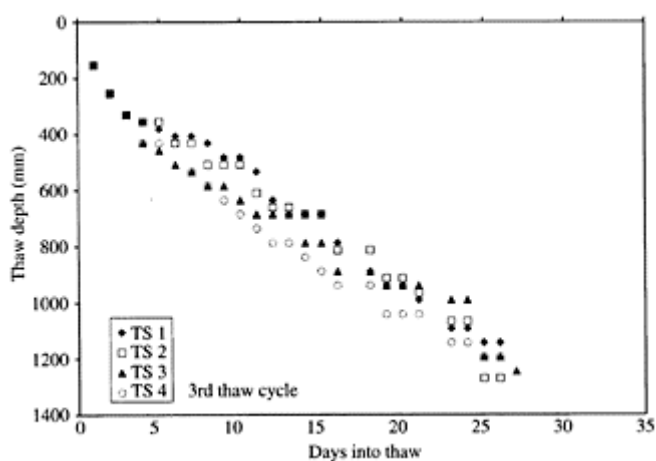


Figure 5. Thaw depths in the test sections as a function of time.

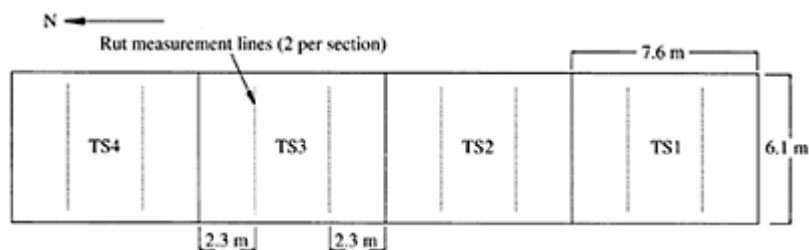


Figure 6. Locations of rut measurements.

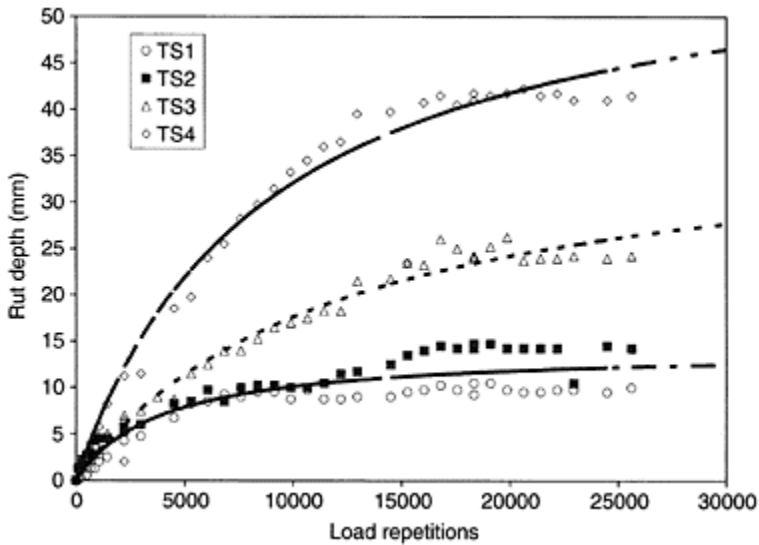


Figure 7. Average rut depth in the test sections during thaw under an 80-kN axle load.

5.2 Influence of load on rut depths

The test sections were trafficked during the thaw process, and transverse rut depths were measured at two locations on each test section, spaced at 2.3 m from each end (Fig. 6). Rut depths were measured using a steel tape referenced to an aluminum beam placed transversely across the test section. The beam was attached to the side walls and was located 1 m above the AC surface.

The load applied on the test windows was 80 kN through the rear axle. The accumulation of the average rut depth as a function of load repetitions at points A and B (Fig. 4) are presented in Figure 7. The results show that for the full-depth AC sections (TS1 and TS2), the maximum rut depth after 25000 load repetitions was less than 15 mm. The initial progression of the rut depth is similar in both test sections up to about 12000 passes, after which there is a small increase in rut depth in TS2. Although this difference is about 3 mm, for all practical purposes, the rut depths can be considered to be the same in both test sections. This suggests that incorporation of the free-draining layer in TS2 provided no additional strength to the pavement structure to prevent rutting (or inversely, the drainage layer had no effect on the rutting of the pavement structure). For the thin-AC sections (TS3 and TS4), the rut depths after 25 000 load repetitions was between 25 and 40 mm for the same load. It is difficult to conclude whether the thickness of the base course (178

Table 3. Coefficients a and b for rut depth predictions.

Test section	TS1 & 2	TS3	TS4
a	14.17	38.83	59.89
b	3981	12056	8667
R^2	0.84	0.98	0.99

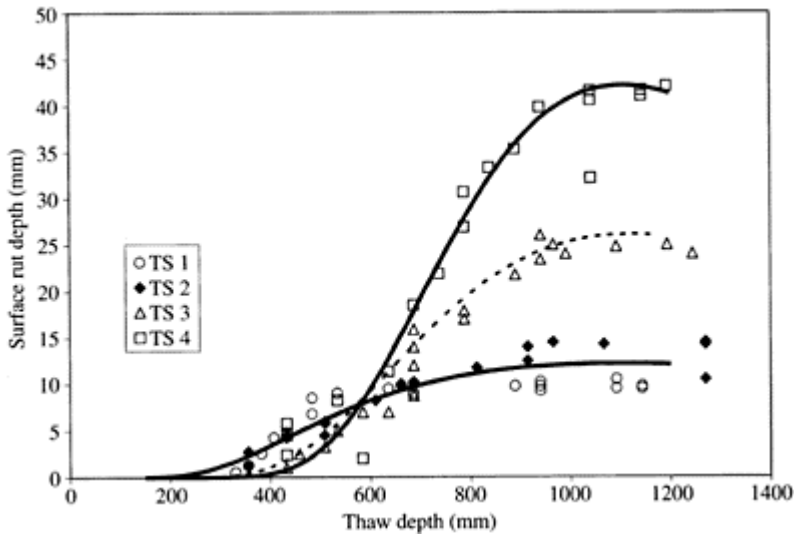


Figure 8. Influence of thaw depth on surface rut depths.

and 254 mm in TS3 and TS4, respectively) or the subbase type and thickness (203 and 127 mm for gravel and sand, respectively) were predominant in determining the rutting potential. As the base course was constructed from crushed aggregate and the sand subbase was difficult to compact, the increased rutting in TS4 is most likely attributable to the sand subbase.

The following best-fit equation (P values close to 0) was used to fit the data in Figure 7; the constants for the equation are presented in Table 3.

$$RD = \frac{aN}{b+N} \quad (1)$$

where RD =rut depth (mm), N =number of load applications, and a, b =constants.

5.3 Influence of thaw depth on rut depth

The influence of thaw depths on the surface ruts in the test sections are shown in Figure 8. As in Figure 7, the thick-AC sections (TS1 and TS2) show less rutting during thaw compared to the thinner-AC sections, even when the thaw is complete. The following exponential function was found to fit the data reasonably well:

$$RD = e^{\left(a + \frac{b}{x} + c \ln(x)\right)}$$

(2)

Table 4. Coefficients a, b and c for Equation 2.

Test section	TS1 & 2	TS3	TS4
a	16.54	39.40	55.01
b	−1922	−5160	−7079
c	−1.755	−4.492	−6.40
R ²	0.82	0.98	0.98

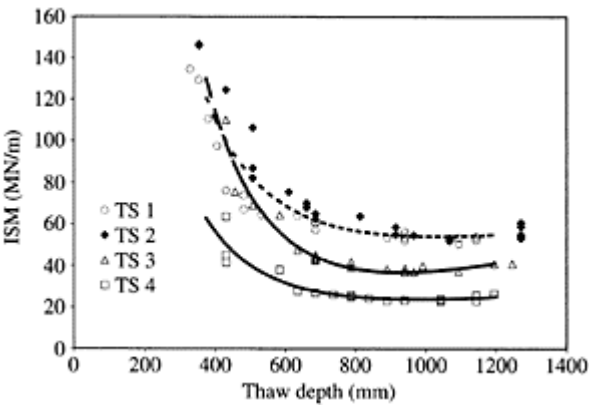


Figure 9. Influence of thaw depth on ISM.

where RD =rut depth (mm), x =thaw depth (mm), a , b , c =constants. The coefficients for Equation 2 are presented in Table 4.

5.4 Influence of Impulse Stiffness Modulus on rut depth

The influence of the pavement stiffness on the rut depth was studied using data from the Falling Weight Deflectometer (FWD). The FWD deflection data were used to calculate the Impulse Stiffness Modulus (ISM). The ISM, which is analogous to the spring constant, is calculated as follows:

$$ISM = \frac{P}{\delta_{\text{center}}} \quad (3)$$

where ISM =impulse stiffness modulus (MN/m), P =applied load (kN), δ_{center} =center deflection (μm).

The ISM is a good indicator of the overall structural pavement performance during thaw (Janoo and Berg, 1990a), as the ISM decreases as thaw progresses into the pavement structure (Fig. 9). Because the ISM is an indicator of the structural capacity of the pavement system, the rut depths measured should also closely relate to the ISM . Figure 10 shows that the ISM had a strong influence on the rut depth for the four test sections. The following model was used to predict the rut depth as a function of the ISM :

$$RD = a \times ISM^{(b \times ISM)} \quad (4)$$

The coefficients a , b and c from the regression analysis are presented in Table 5. The rate of rutting is a function of the constant b , and the magnitude is a function of the constant a . The constants a and b are then a function of the layer properties, geometry and thaw depth.

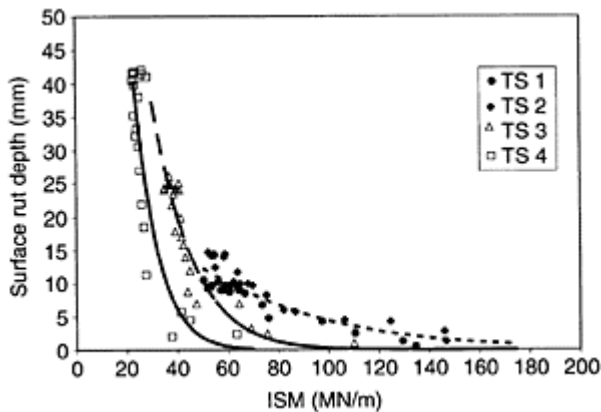


Figure 10. Influence of ISM on surface rut depth.

Table 5. Coefficients a and b for Equation 4.

Test section	a	b	R^2
TS1 & 2	26.4	-0.00388	0.84
TS3	151.4	-0.0137	0.84
TS4	242.0	-0.0294	0.76

Table 6. Regression constants and statistics for Equation 5.

Variable	Value	Standard error	t-ratio	Prob(t)
a	50.42	18.079	2.7889	0.0066
b	-1.321	0.0475	-27.7806	0.0
c	0.4068	0.0319	12.7632	0.0

5.5 Combined effect of thaw depth, ISM and load passes on rut depth

Based on the previous analysis, it can easily be seen that the rut depth during thaw is a function of the structural capacity (ISM), the thaw depth, and the number of load applications. Additional analysis was conducted with all the data from the four test sections. Our hypothesis is that the ISM equates to material properties and layer geometries and also to the influence of thaw depth. Therefore, the rut depth is a function of the ISM and the number of load repetitions (N).

A multiple regression analysis was conducted, with rut depth being the dependent variable and ISM and number of passes as the independent variables. The following equation provided a good fit between rut depth and ISM and N :

$$RD = a \times ISM^b \times N^c \quad (5)$$

where RD =rut depth (mm), ISM =Impulse Stiffness Modulus (MN/m), N =number of load repetitions, a , b , c =constants. The regression constants and statistics on the constants are presented in the Table 6.

This equation is similar in form to that proposed by Zhang and Macdonald (2002); however, we choose a relationship to ISM rather than subgrade strain. Equation 5 will be further evaluated using trafficking data from additional freeze-thaw cycles and test road data. If it is validated, it can be used to develop rutting guidance for trafficking during spring thaw.

6 SUMMARY AND CONCLUSIONS

Test sections were constructed in the FERF to study the impact of loading on thawing pavement structures. Two structures, full-depth and thin-asphalt concrete sections, were constructed over a clay subgrade. Temperature sensors were installed in the test section to track freezing and thawing in the pavement structures.

Falling weight deflection measurements were taken prior to loading of the pavement structures and during thaw. At the end of each loading session, surface rut measurements were taken. The trafficking load applied was 80 kN. The falling weight data were used to compute the Impulse Stiffness Modulus (ISM). The ISM is analogous to a spring constant and accounts for the pavement layer properties and geometry.

The measured rut depth was found to be a function of the number of load applications, the pavement cross section, the thaw depth, and the ISM. A multiple regression equation was developed to predict rut depth as a function of the ISM and load applications. This relationship will be further evaluated using trafficking data from additional freeze-thaw data sets, and if validated, it can be used to develop rutting guidance for pavement deterioration during spring thaw.

REFERENCES

- Atkins, R.T. 1979. Determination of frost penetration by soil resistivity measurements. *CRREL Special Report 79-22*, U.S. Army Cold Regions Research and Engineering Laboratory.
- Janoo, V.C. & Berg, R.L. 1990a. Predicting the behavior of asphalt concrete pavements in seasonal frost areas using nondestructive techniques. *CRREL Report 90-10*, U.S. Army Cold Regions Research and Engineering Laboratory.
- Janoo, V. & Berg, R. 1990b. Thaw weakening of pavement structures in seasonal frost areas. *Transportation Research Record* No. 1286, Transportation Research Board, p 217–233.
- Saarelainen, S., Onninen, H., Kangas, H., & Pihlajamäki, J. 1999. Full-scale accelerated testing of a pavement on thawing, frost-susceptible subgrade (CS8-3). *International Conference on the Accelerated Pavement Testing*, Oct. 18–20, 1999, Reno, Nevada www.tieh.fi/tppt/hvsr
- U.S. Army. 1994. *Pavement Design for Roads, Streets, and Open Storage Areas, Elastic Layered Method*. Technical Manual TM 5-822-13.
- Zhang, W., & Macdonald, R.A. 2002. The effects of freeze-thaw periods on a test pavement in the Danish road testing machine. *9th International Conference on Asphalt Pavements*, Aug. 17–22, 2002, Copenhagen, Denmark. Published by the International Society for Asphalt Pavements, 2002, CD-ROM, 20 http://www.ctt.dtu.dk/group/rtn/ISAP_Paper338_Zhang.pdf.

Application of acceleration measurement method for estimating the stiffness of unbound aggregates in roadbed

M.Kamiura & S.Nakayaka

Hokkai Gakuen University, Sapporo, Japan

Pavements Unbound—Dawson (ed.)

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ABSTRACT: In this paper, we describe the transit elastic wave system for measuring the wave velocities of unbound granular aggregates in base and subgrade. The technique is based on the measurements of surface responses due to an impact at the surface using a portable FWD. The portable system consists of a portable PC, a sampling A/D card (the sampling interval is 0.05 msec), an acceleration system, a portable FWD and Matlab-based software. Measurements of the elastic wave velocities on the unbound granular aggregates base as well as the subgrade have demonstrated its potential in the nondestructive evaluation of unbound granular aggregates structure. The developed portable transient elastic wave system can be utilized to measure the in-situ elastic wave velocities of base and subgrade in pavements and further to evaluate the quality of the base and subgrade structure.

1 INTRODUCTION

This paper is concerned with the base of unbound granular aggregates in pavement. The pavement consists of the surface layer, base, subbase and subgrade. The unbound granular aggregates are the most popular material in a base and subbase. It is required to have adequate stiffness, strength and resistance to deformation by traffic. Recent developments of in-situ testing devices have now made it possible to obtain a direct measurement of stiffness modulus during construction. Falling weight deflectometer (FWD) is performed at each site to characterize the mechanical response of the pavement. Light versions of FWD (hereafter it is called as a portable FWD or pFWD) have zero, 1 or 2 external and movable receivers with acceleration sensors. It was developed to measure stiffness of road bases and underlying layers during the construction of the pavement. It is very helpful to collect field data in terms of layer stiffness during the process of road building. Deflections in the base constituted by unbound granular aggregates have changed with number of drops by FWD or pFWD. It is related to the rate of compaction. Seating Factor was introduced to relate the degree of compaction [1]. As the base layer is compacted by drops of FWD or pFWD, it might differ from the original

stiffness. It is desirable to evaluate stiffness of the other part to avoid the dropping point. Here, we focused on the elastic- wave method to estimate stiffness modulus during construction. Such a research method is progressing in seismology and geology. In this method, it could be estimated the strain level in the stiffness measurements is so small that the original stiffness is only slightly influenced. The short-term goal of this study is to develop an elastic wave method using a falling weight and acceleration sensors of a pFWD. Another, long-term objective is to develop a system utilized to predict the original stiffness of base in pavements.

2 TRANSIENT ELASTIC WAVE

Various methods based on the point-source/point-receiver technique that utilize transient elastic wave response have been proposed in the field of nondestructive evaluation of pavements and soil. The main equipment of pFWD (Fig. 1) comprises a falling mass of 10 kg that loads through a rubber buffer the 100 mm diameter bearing plate, within which is mounted acceleration transducer. The load sampling pulse duration is 0.05 milliseconds. The attachments of the equipment are two external receivers mounted with acceleration transducers.

The impact generated by a pFWD produces the longitudinal wave (P wave), the transverse wave (S wave) and Rayleigh wave (R wave). The vertical components are S wave and R wave. R wave arrival induces a sharp corner and can be identified easily. The vertical component velocity in the transient elastic wave is estimated by the wave transit time (t_L) and distance (L) between the first receiver and second receiver.

In a homogeneous isotropic half space with density ρ , and Poisson ratio ν , the corresponding longitudinal wave speed V_s and the stiffness of base E are as follows:

$$V_s = \frac{L}{t_L} \quad (1)$$

$$G = \rho \cdot V_s^2 \quad (2)$$

$$E = 2 \cdot (1 + \nu) \cdot G \quad (3)$$

In the Spectral-Analysis-of-Surface-Wave (SAWS) method, several successful applications have been reported [2, 3, 4]. The values of frequency in Table 1 were processed from these papers. In this study, the impact loadings by a pFWD produced the transient elastic wave accelerations in the base and a new system using this transient elastic wave was developed and was described in the following section.

Finally, results of the in-situ applications using this system in determining the elastic wave velocity of base are given.

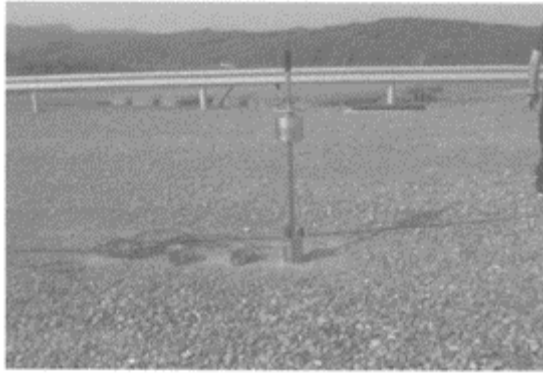


Figure 1. The portable FWD.

Table 1. Example of the range in the frequency and stiffness.

		Frequency(Hz)		Stiffness (MPa)	Reference
Base	Limestone	143		210	[2]
Base	Limestone	92	105	200	[3]
Subgrade	Sandy slit	50	100	Poison ratio 0.45	
				Unit volume weight 20 kN/m ³	[4]

3 PRELIMINARY FIELD WORK TESTS

3.1 Measurement methods

Prior to the main tests, the preliminary field work tests were carried out in a sandy subgrades. The needs of the examinations are as follows:

- It needs to verify a clear displacement jump at the wave-front arrival.
- The receiver of a pFWD are not fixed but placed on the surface of the base. It is necessary to check the measurements compared with those by the sensors fixed to the surface. For the surface fixing, there was every possibility to disturb the base comprised by the unbound aggregate. Therefore, the authors adopted a sandy subgrade for the preliminary field work tests.

The performance of the acceleration sensor is as follows: The capacity is 1000 m/sec² acquisition of temperature compensated digital acceleration data of 0.2 mHz–700 Hz. Surface layers of subgrade consists of sandy soil and clay soil (Fig. 2). In sandy soil, unit volume weight is 19 kN/m³, water content is 15% and K value stiffness is 137 MN/m³.

The receiver A to the receiver B distance changed from 100 mm to 1000 mm at the interval of 100 mm. Figure 3 shows the receiver of pFWD and the receiver with an

acceleration sensor (A or B) on the pin. The receiver of pFWD and the receiver B was placed at 100 mm intervals from 100 mm to 1000 mm. The pin was fixed on the surface of the subgrade. Here, the time difference means transit time between the receiver A and B. The values of time difference of the pin or the stick were compared

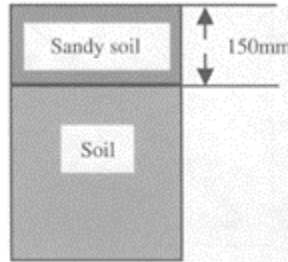


Figure 2. Subgrade layers.

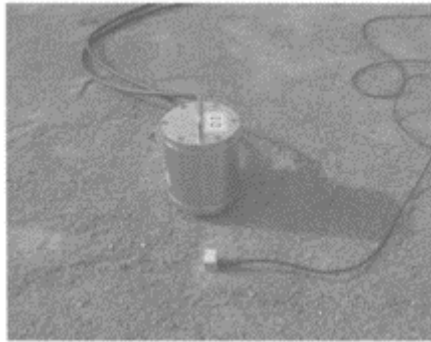


Figure 3. The receiver of pFWD and the acceleration sensor on the pin.

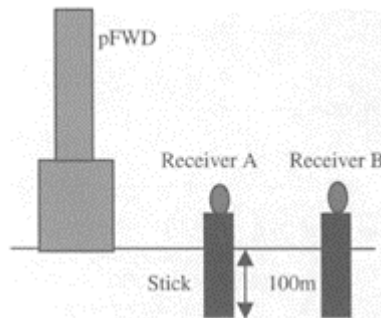


Figure 4. Acceleration receivers (A and B) on the sticks (Depth: 100 mm).

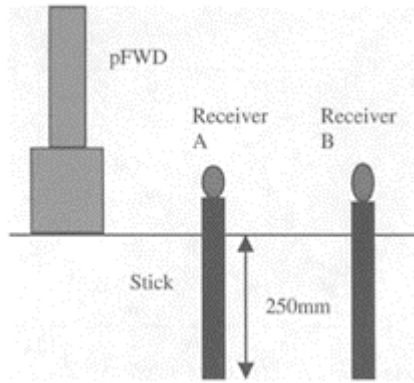


Figure 5. Acceleration receivers (A and B) on the sticks (Depth: 250 mm).

with that of pFWD. Sticks hammered with depths of 100 mm in the subgrade (Fig. 4). In the same way, the sticks hammered with depths of about 250 mm in the subgrade and the lower end of the stick reached the second layer(in Fig. 5). Table 2 shows the quality and shape of these fixing materials.

3.2 Results of the tests

Figures 6, 7 and 8 are the obtained relationships between the time difference and distance. These results show a good correlation between the time difference and the distance. Figures 6 and 7 show the correlation lines are approximately identical in cases of the pins and pFWD or the sticks and pFWD. But, the line of Fig. 8 does not match at all. The authors have an assumption that the main reason for appearance of 2 different cases might be caused by the difference in layer of the transferred elastic wave. In case of Fig. 6 or 7, all of the acceleration sensors might receive the elastic wave transferred in the sandy soil layer. In contrast to these cases, in case of Fig. 8 the sensors of sticks might receive the wave in soil layer. It is estimated an acceleration of the pin or the stick generated by a transit elastic wave might reach at the lower end that was the opposite end of the sensor.

Summarizing these considerations, it could be said the receiver of a pFWD unfixed on the surface of the base has almost the same capability as the sensors glued on the fixing materials that

Table 2. Quality and shape of fixing materials.

	Quality	Shape	
Pin	steel	Diameter: 10mm	Length 7 mm
Stick	steel	Diameter: 10 mm	Length 275 mm

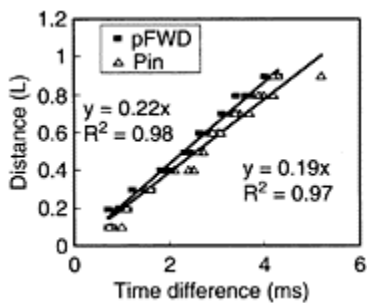


Figure 6. Time difference and distance (Pin, pFWD).

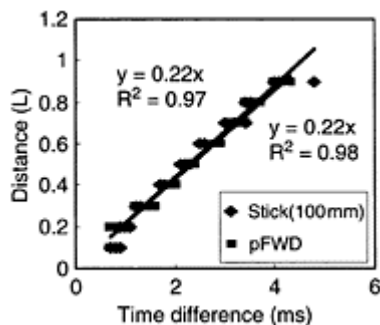


Figure 7. Time difference and distance (Stick: 100 mm, pFWD).

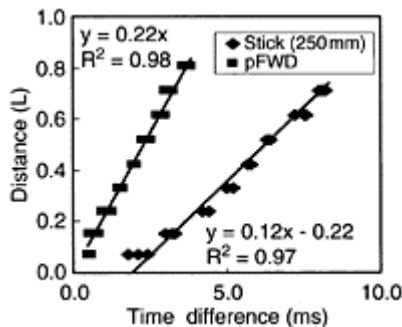


Figure 8. Time difference and distance (Stick: 250 mm, pFWD).

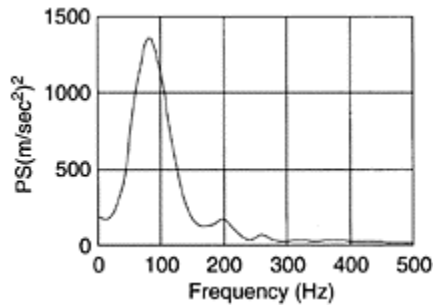


Figure 9. Frequency and power spectrum ($D=300$ mm).

were pricked or knocked into the subgrade. The velocity corresponds to the gradient of pFWD of the line in Fig. 6. Then, using this velocity and assumed that a Poisson's ratio is 0.45, the stiffness of the subgrade was calculated (272 MPa) by the formula (1), (2) and (3). It is the same stiffness value as in case of stick (100 mm) of Fig. 7. The stiffness of the subgrade is estimated to be 203 MPa.

Figure 9 shows an example of the results in Fourier Analysis (FFT). Here, PS in vertical axis means power spectrum and $D=300$ mm means the distance between the center of loading plate and measuring point is 300 mm. In this figure, the frequency value of the peak is nearly 80 Hz that is almost similar to the values of subgrade in the reported example (Table 1).

4 MAIN FIELD WORK TESTS

4.1 Estimation of acceleration wave transit time

A site of an arterial high-standard highway under construction was assessed in Hokkaido Japan (Fig. 1). In the field tests, the layers of base, subbase and subgrade had been constructed (Fig. 10). Figure 11 is a gradation curve in the base layer. Top size of ballast is under 40 mm. The unit volume weight was 23 kN/m^3 . It was not clear in shape on the leading edge of acceleration wave comparing with that in the subgrade of the preliminary field work tests. It was speculated that the vertical wave transferred in the base was more complicated than in the subgrade. Wavelet transformation analysis [5] was introduced to evaluate the time difference of acceleration wave. It is similar to FFT analysis in regard to sample factors according to the amplitude of wave in the temporal frequency. One advantage using wavelet is that a series of frequencies can be set up at each time series. The continuous wavelet transformation is adopted in this study on the basis of the necessity of using all data. It was assumed that the wavelet analysis was suited for acceleration wave using Morlet wavelet [6] that is one of the basic kinds of wavelets and it is very easy to detect the peak of a wave by mechanical routine by Wavelet analysis.

As shown in Fig. 1, a pFWD generated a vertical elastic wave that was detected by the sensor in loading plate of the pFWD, 1st receiver and 2nd receiver. This 1st receiver was

placed and kept at the distance of 300 mm from the center of the loading plate. The 2nd removable receiver was placed at the distance of 200 mm ($L=200$) or 900 mm ($L=900$) from the center of 1st receiver to 2nd receiver.

In Fig. 12, the top figure presents an example of an acceleration wave of 1st receiver. And a result of wavelet analysis is shown at the bottom figure. This acceleration wave was decomposed by wavelet analysis into a series of sin curves. The magnitude of wavelength and amplitude can be calculated at any time by a computer program. For example, the point “P” located in the center of the contour in the figure 12 shows the peak time of the sine curve of which the wavelength is about 100 m. In these figures, they were illustrated in the contour map by the level of brightness. The peaks of the wave were indicated more brightly in the contours. The 2nd receiver was shown in Fig. 13 in the same manner as the 1st receiver. The wave transit time (t_L) in formula (1) could be automatically computed by wavelet analysis.

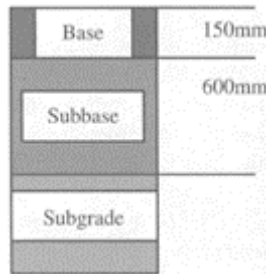


Figure 10. Pavement layers examined in this study.

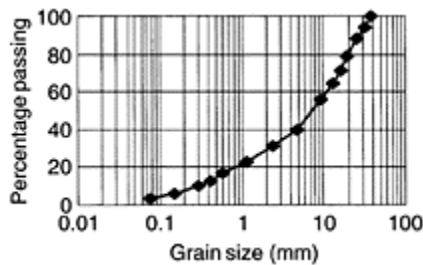


Figure 11. Gradation curve of the base.

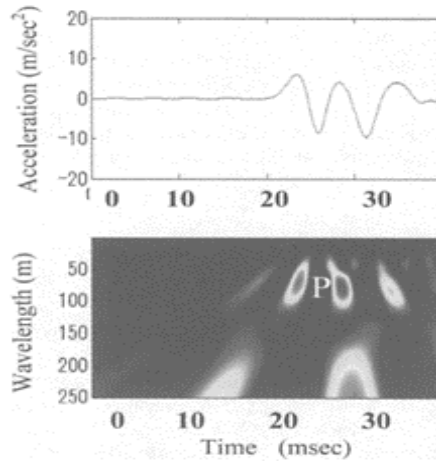


Figure 12. Acceleration and wavelet chart of the 1st receiver.

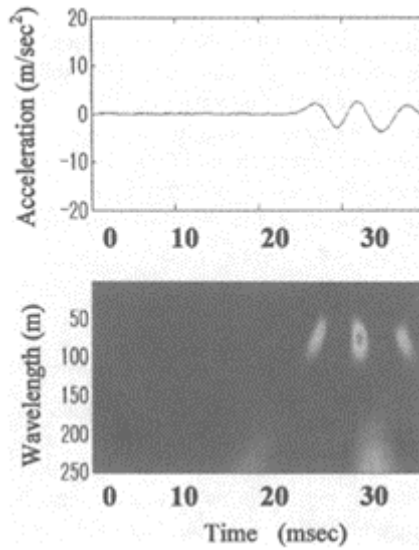


Figure 13. Acceleration and wavelet chart of the 2nd receiver.

4.2 Results of the tests

The values of stiffness in base were evaluated by Formula (1), (2) and (3) by changing the distance from 200 mm to 900 mm from the center of 1st receiver to 2nd receiver. For

compared with the wavelet method, the stiffness of base was also evaluated by Boussinesq method as follows:

$$E = \frac{2 \times (1 - \nu^2)}{\pi \times r \times d} P \quad (4)$$

Where P =Impact loading weight of pFWD, and d =deflection.

Figure 14 presents the results of the stiffness of base evaluated by wavelet and Boussinesq method. Comparing with these results, the average value stiffness ($E=234$ MPa) of wavelet method is a bit smaller than the value ($E=238$ MPa) of Boussinesq method.

5 WAVE ANALYSIS

5.1 FFT analysis

Figure 15 presents FFT results of the vertical acceleration at the center of loading plate and receiver kept at the distance of 300 mm. Observing the figure, the whole trend of plots could be recognized within the range of 0 to 300 Hz. 3 peaks were shown within limits of 300 Hz. One of the peaks was classified in the range under 100 Hz and the others were from 100 Hz to 200 Hz. These were also reported in [1] and [2] shown in Table 1 in the base layers. This fact is different from the FFT results of Fig. 9 in a sandy subgrade.

The authors estimated the Elastic wave became extinct as it transferred on the surface of the base layer. Extinct phenomena were observed by decreasing the amplitude of elastic wave acceleration as the 2nd receiver moved from 200 mm ($L=200$) to 900 mm ($L=900$) (Fig. 16). It was interesting two peaks kept to be present in the form of the apex position in spite of being extinct.

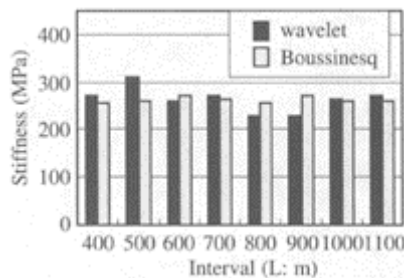


Figure 14. Stiffness of base evaluated by wavelet and Boussinesq method.

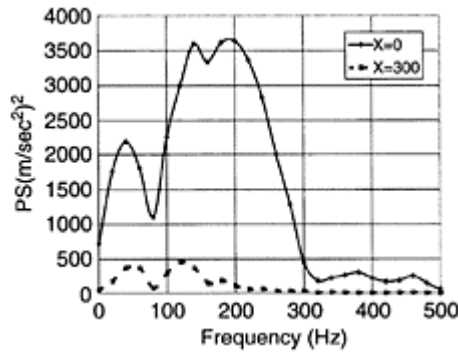


Figure 15 Magnitude spectrum of elastic wave in the base(1).

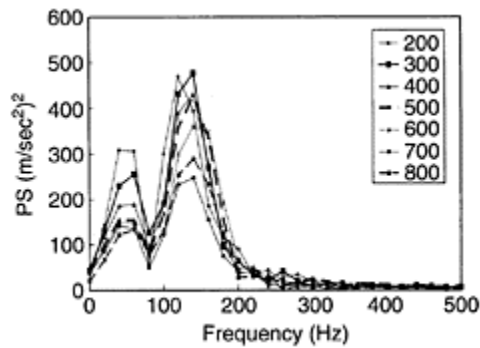


Figure 16. Magnitude spectrum of elastic wave in the base (2).

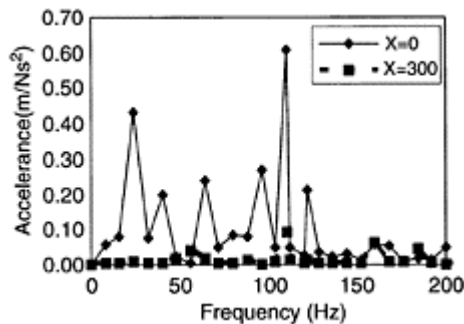


Figure 17. Accelerance of acceleration of the elastic wave and loading weight of pFWD.

5.2 Frequency response function

The authors focused on the peaks in Fig. 15 and Fig. 16 regarding the results as the resonant response. Accelerance that is one of the frequency response functions was introduced to confirm the author's view. It is defined as "acceleration/force". Describing the formula, this function is shown as follows:

$$G(w) = \frac{A(w)}{F(w)} \quad (\text{m/Nsec}^2) \quad (5)$$

where $G(w)$ is accelerance, $A(w)$ is the acceleration of the elastic wave, $F(w)$ is the loading weight, and w is the temporal frequency. As $F(w)$ is the function of loading weight for phase, it is a function of complex variable. To simplify the function of $G(w)$, it is necessary the denominator of formula (5) should be a function comprised of real numbers. It was multiplying $F(w)$ by the conjugate function of $F(w)$ and was simplified as follow:

$$G(w) = \frac{\overline{F(w)}}{F(w)} \times \frac{A(w)}{F(w)} = \frac{W_{fa}(w)}{W_{ff}(w)} \quad (6)$$

where $W_{fa}(w)$ is the cross spectrum function of $F(w)$ and/or $A(w)$, and $W_{ff}(w)$ is the power spectrum function of $F(w)$.

Figure 17 presents an accelerance of acceleration of the elastic wave and loading weight of pFWD at the center of loading plate and the receiver kept at the distance of 300 mm. This figure might reveal the following new two findings:

One finding has to do with the acceleration at the center of loading plate. It is that the peaks in graph contain within the range of frequencies below 130 Hz that accounts for a part of lower frequency range of peaks in the Fig. 15.

The other has to do with the acceleration of the receiver kept at the distance of 300 mm. It is that the graph presents very small wave peaks compared with the loading plate that may be speculated that the attenuation led to lower the accelerance.

6 SUMMARY AND CONCLUSIONS

The stiffness of unbound aggregates in the base of pavement was estimated measuring the acceleration of the elastic wave by the receivers installed in a portable FWD. The main findings can be summarized as follows:

- 1) Although the receivers of a portable FWD were unfixed on the surface, they had almost the same capability as the acceleration sensors glued on the pins or sticks that were pricked or knocked into the subgrade.
- 2) The range of frequency in the acceleration by the impact loading of a pFWD was from 0 to 300 Hz in Fourier analysis. The peaks in power spectrum were 2 groups: one is less than 100 Hz and the other is from 100 Hz to 200 Hz. These were almost the same classification as the reported researches.
- 3) The wave transit time (t_t) could be automatically computed by wavelet analysis.
- 4) The peaks in power spectrum were regarded as the resonant response. This findings were confirmed by the analysis introducing the index of accelerance.

REFERENCE

- [1] Christ van Gorp, Jacob Groenendijk. 2000. Experience with various types of foundation tests. *Proc. 5th. Unbound Aggregates in Road Construction*. 239–246.
- [2] Soheil Nazarian and Kenneth. H.Stokoe. 1986. Use of surface waves in pavement Evaluation. *Transportation Research Record 1070 TRB National Research Council*. 132–144. Washington D.C.
- [3] Soheil Nazarian and Kenneth. H.Stokoe, Robert.C.Briggs and Richard Rogers. 1988. Effect of reflected waves in SASW testing of Pavements. *Transportation Research Record 1196 TRB National Research Council* 133–150. Washington D.C.
- [4] Deren Yuan and Soheil Nazarian. 1993. Automated surface wave method inversion technique. *Journal of geotechnical engineering*, vol. 119 No. 7.1112–1126.
- [5] Susumu Sakakibara. 1995. *Wavelet Beginner's guide*. 2–12. Tokyo Denki University
- [6] Masaki kamiura. 2003. An effective method for the turnout maintenance using wavelet analysis. *World Congress on railway research*. 9–12.

Performance testing of unbound materials within the pavement foundation

B.Rahimzadeh, M.Jones, B.Hakim & N.Thom

Scott Wilson Pavement Engineering, Nottingham, UK

Pavements Unbound—Dawson (ed.)

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ABSTRACT: Pavement design requires knowledge of the performance of the foundation layers. The specific condition at a given site can be considered in the design; such as the use of locally available material, recycled or secondary aggregate in the pavement foundation, provided the specified end-product performance requirements are achieved.

Performance testing is required in order to ensure that the design assumptions for the material properties are met during construction. Both the capping and sub-base layers need to attain adequate strength and stiffness. Therefore there is a need to undertake complimentary compliance testing that would include target values based on the behaviour of the materials.

This paper describes the research into the development of performance testing for foundation layers based on case studies performed by Scott Wilson Pavement Engineering (SWPE). The performance testing was carried out by Falling Weight Deflectometer (FWD), German Dynamic Plate (GDP) and Prima 100. The results using these test methods were compared and correlated.

The results indicated that the relationships between FWD, GDP and Prima were material type and thickness dependent. The Prima usually gave broadly similar results to the FWD, but was significantly more variable from point to point. The GDP almost always gave a lower stiffness than the other devices, but to varying degrees.

1 INTRODUCTION

The road foundation layers, which consist of the capping (where necessary) and sub-base layers that overly the natural soil subgrade, perform several functions both during construction and when the road is in service. In particular they act as load-spreading layers to reduce to acceptable levels the stresses transmitted to the subgrade, (often as temporary haul roads during construction), and as construction bases on which the overlying pavement layers can be adequately laid and compacted. The critical loading

conditions usually occur during construction where the materials are directly trafficked, and hence where the applied stresses are greatest. The materials that are used for the capping and sub-base layers need to have both adequate stiffness and strength to perform satisfactorily.

The usual current UK specification for road foundations (MCHW Vol. 1 2004) is based on a recipe approach, whereby selected materials are laid and compacted with specified plant in a specified manner to achieve a minimum level of performance. The pavement foundation designs are based primarily on the use of the California Bearing Ratio (CBR) to characterise the subgrade, capping and sub-base materials. Here CBR is used as a measure of both material strength and stiffness. Although CBR has been correlated with pavement performance in many countries over many years and provides a trusted empirical indicator of material behaviour, the use of CBR as a performance parameter is widely acknowledged as being not wholly satisfactory (Brown 1996). Furthermore, the need for a fundamental engineering property (such as stiffness) to describe the unbound material has become important for use in analytical/mechanistic design methods.

Advances in the in-situ testing of pavement foundation materials now allow the performance parameter of stiffness (and, more indirectly, strength and resistance to permanent deformation) to be measured on a routine basis during construction. This in turn enables a performance-based specification for road foundation layers to be introduced, hence facilitating the use of secondary aggregates, marginal materials and stabilised ground.

The stiffness modulus of a pavement foundation is a measure of the quality of support which is provided to the overlaying asphalt or concrete layers. Recent developments of in-situ testing devices have now made it possible to obtain a direct measure of the stiffness modulus during construction. Use of such devices for compliance testing is becoming a real possibility and ultimately may be expected to supersede the use of the California Bearing Ratio (CBR) test, considered by many countries as being not wholly satisfactory.

Considerable research has been undertaken over the past few years to develop in-situ testing devices that quickly measure the stiffness of the subgrade and the pavement layers during construction (Fleming et al. 2000). These devices measure a composite stiffness under a transient load pulse, which is applied to the ground by dropping a weight onto a bearing plate via a rubber buffer. The deflection of the ground is measured and combined with the applied load, which is either measured or is assumed to be constant (by means of a constant drop height), to calculate the stiffness using conventional Boussinesq static analysis. Such devices include Dynamic Plate Tests (e.g. Falling Weight Deflectometer (FWD), German Dynamic Plate (GDP) and Prima 100).

This paper describes an investigation in to the differences between in-situ testing devices (FWD, GDP and Prima 100) at live construction sites and comparison of results for stiffness measured using these apparatus.

2 EXPERIMENTAL DETAILS

2.1 *Trial sites*

The testing results of five trial sites established by Scott Wilson Pavement Engineering (SWPE) have been used for this experiment (Thom 2003). The sites were selected in order to cover as wide a range as possible of foundation materials as well as site and contractual conditions as outlined below:

- A2-M2 Cobham to Junction 4 Widening. Trials performed on capping comprising stabilised clay with flints as well as crushed concrete
- Realignment of Taxiway Alpha, Jersey Airport. Full contractual implementation of the proposed Performance Specification, resulting in significant savings to the Client
- Widening of Church Way, Doncaster. Trial to demonstrate the applicability of the Performance Specification within an urban environment where construction proceeded in a piecemeal fashion
- A27 Polegate Bypass. Trial performed on a lime and cement stabilised Weald Clay sub-base
- A43 Towcester to M40 Dualling. Trial to assess the site-won Oolitic limestone capping.

For all these projects, Dynamic plate testing (FWD, Prima and GDP) was conducted. Tests were conducted on the surface of the subgrade and capping following all surface preparation work prior to construction of the subsequent pavement layer.

The tests were first performed with the FWD, since the towing vehicle is fitted with a distance-measuring device, and the equipment can provide several repeat measurements with different drop heights (applied loading) to investigate load related effects. Following each FWD test, the location of the load plate was circled with spray paint and given a station reference number (this referencing is evident in Figure 1). This allowed the “portable” GDP and Prima equipment tests to be performed at exactly the same locations, in order to assess whether a correlation with the FWD equipment could be obtained. The FWD is well established testing device, which is widely used as a bench mark.



Figure 1. Test at sub-formation level comprises FWD, Prima and GDP testing at the same location as FWD.

2.2 Test equipment

The Dynatest manufactured 8001 Falling Weight Deflectometer (FWD)—Stripped of all additional weights the FWD provides a minimum falling weight of around 50 kg which, when raised to a drop height of around 70 mm delivers a measured (by a load cell) pulse load of 100 kPa via two round-topped rubber buffers. In addition, the static assembly mass (providing a degree of static pre-load through the load plate) is around 50 kg. The 150 mm radius load platen was fitted (as standard) with a rubber mat in an attempt to provide an improved contact with the test surface. Deflection is measured by a geophone (which measures velocity, which is then integrated to obtain vertical displacement i.e. deflection) located above a 10mm diameter tip at the centre of the load plate.

The Carl Bro manufactured Prima has a falling weight of 10 kg which, when raised to a drop height of around 900 mm delivers a measured (by a load cell) pulse load of 100 kPa via four rounded conical buffers. In addition, the static assembly mass is around 16 kg. The 150 mm radius load platen is not fitted (as standard) with a rubber mat. In the modified form used, deflection is measured by a geophone located above a 30 mm diameter plunger at the centre of the load plate.

The German manufactured German Dynamic Plate (GDP) has a falling weight of 10 kg which, when raised to a drop height of around 750 mm delivers an assumed (by prior calibration) pulse load of 100 kPa via a spring. In addition, the static assembly mass is around 20 kg. The 150 mm radius load platen is not fitted (as standard) with a rubber mat. Deflection is measured by an accelerometer (which requires double integration to obtain the deflection) built into the 300 mm diameter load plate.

3 RESULTS AND DISCUSSION

Dynamic Plate Tests were carried out using three pieces of equipment, the Falling Weight Deflectometer, Prima and German Dynamic Plate. The surface modulus/stiffness results (E_{vd}),

Table 1. Summary of stiffness correlations—
Factors to be applied to FWD stiffness to equate to
PRIMA/GDP stiffness.

Test location	Site 1: Tusmore				Site 2: Baynards Green			
	PRIMA		GDP		PRIMA		GDP	
	Run 1	Run 2	Run 1	Run 2	Run 3	Run 4	Run 3	Run 4
Top of sub-base	–	–	–	–	1.16	–	0.92	–
Top of capping (Formation)	1.11	1.14	0.44	0.42	0.80	1.02	0.62	0.63
Top of subgrade (Sub-formation)	0.89	0.87	0.36	0.35	0.67	0.69	0.48	0.43

based on vertical deflection, were calculated using the following equation:

$$E_{vd}=PRF\times(1-\nu)^2\times r\times p/d$$

(1)

- where, E_{vd} =Surface Modulus (MN/m² or MPa)
- PRF =Plate Rigidity Factor (=2.0 for Prima, and $\pi/2$ for FWD & GDP)
- n= Poisson’s Ratio (set at=0.35 for all three pieces of equipment)
- r= Load Plate Radius (=150 mm)
- p= Contact Pressure (normalised to 0.1 MN/m² or 0.1 MPa)
- d= Vertical Deflection (mm).

A summary of correlations between the FWD stiffnesses and both the Prima and GDP (equipment under trial) stiffnesses for two sites between A43 Towcester and M40 Dualling is presented in Table 1.

It is evident from Table 1 that the factors to be applied to the FWD stiffnesses, to equate to the PRIMA/GDP stiffnesses, are consistent between the adjacent Runs but differ between the two sites and the two pieces of equipment.

Testing results of the different devices on five construction sites and on a number of subgrade, capping and sub-base layers were compared. The dynamic plate test plays a central role in this experiment and in general it has been found to be both user-friendly and contract-friendly. However, the issue of the different results obtained from the three different devices used in this project (FWD, Prima and GDP) is a serious one, and needs further discussion.

Firstly, in several instances the results obtained showed that:

- the Prima was subject to a high degree of scatter on some sites
- the GDP measured much lower than the other two devices on most sites.

The FWD was confirmed as the most “robust” of the three on which to base the Specification limit, and it was decided to plot the data from the other two devices against that from the FWD. [Note: Procedures already exist for annual calibration of FWD if the equipment is to be used on Highways Agency projects]. The data is shown in Figure 2 for all locations where both the FWD and at least one other device were used. Also shown in Figure 2 are two suggested correlation lines, one for the Prima and the other for the GDP:

$$E(\text{Prima}) = E(\text{FWD}) \times 1.273 \quad (2)$$

$$E(\text{GDP}) = E(\text{FWD}) / [1 + E(\text{FWD}) / 150] \quad (3)$$

Next, the degree of inherent scatter expected was considered. This had been evaluated at most of the sites, based on either repeat testing or on several tests performed within a small area. Typical standard deviations ranged from 10% to 30% of the mean, with relatively high values on coarse capping and higher values for the Prima than the other devices. Figure 3 has been generated as a random set of data for comparison with Figure 2. The same mean correlations as found for Figure 2 have been assumed to apply, but a random scatter has been applied to a given set of “real” stiffnesses, with standard deviations of 20% of the mean for the FWD and the GDP, but 30% for the Prima, approximately

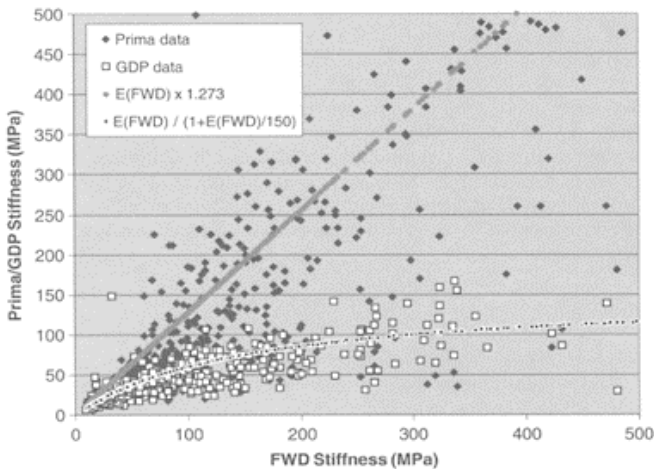


Figure 2. Comparison of actual stiffnesses from Dynamic Plate Tests.

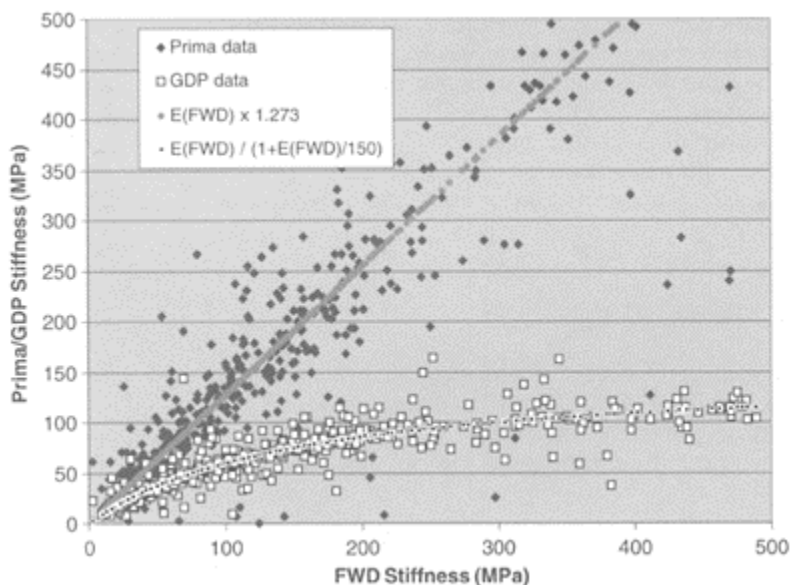


Figure 3. Comparison of generated stiffnesses based on measured scatter.

matching the experience to date. Clearly, Figures 2 and 3 are visually comparable. The conclusion is that inherent variability may be responsible for virtually all the lack of correlation observed.

The other issue is the actual correlation applying. The factor between the FWD and the Prima, 1.273, is simply due to a different assumption regarding plate stiffness; the Prima analysis assumes a rigid plate (plate rigidity factor of 2), whereas the FWD analysis assumes a flexible plate (plate rigidity factor of $\pi/2$). The correlation between deflections is approximately 1:1.

The correlation between the FWD (or the Prima) and the GDP is much less satisfactory. This is undoubtedly due to the GDP measuring only acceleration, with an assumed conversion to stiffness. Because of the flatness of the correlation at higher stiffness, the GDP is clearly not suited to surfaces with stiffness greater than about 100 MPa. The formula suggested in Figures 2 and 3 is considered appropriate at FWD measured stiffness values up to 100 MPa (GDP measurements up to 60 MPa), since it actually lies slightly higher than the mean of the data. If a straight line relationship is assumed, then a ratio of 1.5 is approximately correct up to 100 MPa, but use of the curve shown is considered to be both simple enough and also “safer”.

With the slightly safer correlation given for the GDP, and the Prima analysis corrected to the flexible platen assumption, either device is considered suitable for use in the performance testing (the GDP only on the basis of an agreed correlation for each particular construction), although FWD testing would be recommended wherever possible in order to obtain more data with which to improve the proposed correlations.

4 CONCLUSIONS

- The relationships between FWD, GDP and Prima were found to be material type and thickness dependent. The FWD is still regarded as being the most appropriate device for setting the standard, not only because the loading is most representative of real traffic, but it can also be used for assessment of all pavement layers as construction proceeds.
- The Prima usually gave broadly similar results to the FWD, but was significantly more variable from point to point.
- The GDP almost always gave a lower stiffness than the other devices, but to varying degrees.
- The GDP in particular was found to be very user-friendly.
- The proposed simple statistical treatment of the data was found to work satisfactorily.
- Either the FWD or the Prima (current version, modified to measure ground deflection through a hole in the plate) can be used for measurement of stiffness, as long as the same plate rigidity factor is assumed ($\pi/2$ for a flexible plate). If the Prima default setting (rigid plate, rigidity factor of 2) is assumed, then a correction factor must be applied, such that $E(\text{FWD})=E(\text{Prima})/1.273$.
- The GDP can be used for measurement of stiffness up to 60 MPa (approximately 100 MPa with the FWD); as long as a correlation has been obtained between the devices for each specific construction type.

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REFERENCES

- Brown, S.F. (1996) "Soil Mechanics in Pavement Engineering" 36th Rankine Lecture of the British Geotechnical Society. *Geotechnique*, Vol. 46 No. 3, pp 383–426.
- Fleming, P.R., Rogers, C.D.F., Thom, N.H., Armitage, R.J. & Frost, M.W. (October 2000) "Performance Based Specification for Road Foundation Materials" *Institute of Quarrying Millennium Conference*, Bristol.
- Fleming, P.R. & Rogers, C.D.F. (May 1995) "Assessment of Pavement Foundations during Construction" *Transport, Proc. Of the Institution of Civil Engineers*, 111 (2), pp 105–115.
- Highway Agency (1994) "Structural Assessment Methods", *Design manual for roads and bridges*, Vol. 7, Pavement design and maintenance, HD29/94.
- MCHW (2004) "Manual of Contract Documents for Highway Works" *Specification for Highway Works* Vol. 1. Series 600 and 800 HMSO, London, UK.
- Thom, N. (2003) "Implementation of a Performance Specification for Capping and Subgrade" Scott Wilson Pavement Engineering, Final Report submitted to Highway Agency.

Neural network-based structural models for rapid analysis of flexible pavements with unbound aggregate layers

H.Ceylan & A.Guclu

Iowa State University, Ames, Iowa, USA

E.Tutumluer & M.R.Thompson

University of Illinois, Urbana, Illinois, USA

F.Gomez-Ramirez

EPSA-LABCO, Santo Domingo, Dominican Republic

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ABSTRACT: This paper describes the use of artificial neural networks (ANNs) as pavement structural analysis tools for the rapid and accurate prediction of critical responses and deflection profiles of flexible pavements subjected to typical highway loadings. The ILLI-PAVE finite element program, extensively tested and validated for over three decades, was used as an advanced structural model for solving critical responses of flexible pavement responses with unbound aggregate layers. Unlike the linear elastic layered theory commonly used in pavement layer backcalculation, nonlinear unbound aggregate base (UAB) and subgrade soil response models were used in the ILLI-PAVE program to account for the typical stiffening behavior of UABs and the fine-grained subgrade soil moduli decreasing with increasing stress states. ANN models then trained with the results from the ILLI-PAVE solutions have been found to be viable alternatives. The pavement deflection profiles could be only predicted with the proper characterization of nonlinear stress-dependent UABs and subgrade soils in the trained ANN models. The trained ANN models were also capable of rapidly predicting critical pavement responses with low average errors of those obtained directly from the ILLI-PAVE analyses.

1 INTRODUCTION

Elastic layered programs (ELPs) used in asphalt pavement analysis assume linear elasticity. Pavement geomaterials do not, however, follow a linear type stress-strain

behavior under repeated traffic loading. Rather, nonlinear stress sensitive response of unbound aggregate materials and fine-grained subgrade soils (herein referred to as geomaterials) has been well established (Brown and Pappin, 1981; Thompson and Elliott, 1985; Garg et al., 1998). Unbound aggregates exhibit stress hardening or stiffening whereas fine-grained soils show stress softening type behavior. When these geomaterials are used as pavement layers, the layer stiffnesses, i.e., moduli are no longer constant but functions of the applied stress state. Pavement structural analysis programs that take into account nonlinear geomaterial characterization, such as the ILLI-PAVE finite element program (Raad and Figueroa, 1980), need to be employed to more realistically predict pavement response needed for mechanistic based pavement design.

Recent research at the University of Illinois has focused on the development of Artificial Neural Network (ANN) based forward and backcalculation type flexible pavement analysis models to predict critical pavement responses and layer moduli, respectively. In the field, pavement deflection profiles are obtained from Falling Weight Deflectometer (FWD) measurements, which require the use of backcalculation type structural analysis to determine pavement layer stiffnesses and as a result estimate pavement remaining life. For this purpose, the ILLI-PAVE finite element program was utilized to generate a solution database for developing ANN-based structural models to accurately predict pavement deflection basins, and pavement layer moduli from realistic pavement surface deflection profiles or synthetic FWD data. Such use of ANN models is described in this paper.

2 NONLINEAR GEOMATERIAL CHARACTERIZATION

Under the repeated application of moving traffic loads, most of the pavement deformations are recoverable and thus considered elastic. It has been customary to use resilient modulus (M_R) for the elastic stiffness of the pavement materials defined as the repeatedly applied wheel load stress divided by the recoverable strain. Repeated load triaxial tests are commonly employed to evaluate the resilient properties of unbound aggregate materials and cohesive subgrade soils. Therefore, emphasis should be given in structural pavement analysis to realistic nonlinear material modeling in the base/subbase and subgrade layers primarily based on repeated load triaxial test results (AASHTO T307-99, European CEN Std EN 13286-7).

Simple resilient modulus models are often suitable for finite element programming and practical design use, such as:

$$\textbf{K-}\theta \textbf{ Model (Hicks and Monismith, 1971): } M_R = K(\theta/p_o)^n \quad (1)$$

$$\textbf{Universal Model (Uzan et al., 1992): } M_R = K_1 (\theta/p_o)^{K_2} (\tau_{oct}/p_o)^{K_3} \quad (2)$$

where $\theta = \sigma_1 + \sigma_2 + \sigma_3 = \sigma_1 + 2\sigma_3 =$ bulk stress, $\tau_{oct} =$ octahedral shear stress $= \sqrt{2}/3 * \sigma_d$ (where $\sigma_d = \sigma_1 - \sigma_3 =$ deviator stress in triaxial conditions), p_o is the unit reference pressure (1 kPa or 1 psi) used in the models to make the stresses non-dimensional, and K , n , and K_1 to K_3 are multiple regression constants obtained from repeated load triaxial test data on granular materials. The simpler K- θ model often adequately captures the overall stress

dependency (bulk stress effects) of unbound aggregate behavior under compression type field loading conditions. The universal (Uzan et al., 1992) model considers additionally the effects of shear stresses and handles very well the modulus increase (unbound aggregates) or decrease (fine-grained soils) with increasing stress states even for extension type field loading conditions.

The resilient modulus of fine-grained subgrade soils is also dependent upon the stress state. Typically, soil modulus decreases in proportion to the increasing stress levels thus exhibiting stress-softening type behavior. As a result, the most important parameter affecting the resilient modulus becomes the vertical deviator stress on top of the subgrade due to the applied wheel load. The bilinear or arithmetic model (Thompson and Elliot, 1985) is a commonly used resilient modulus model for subgrade soils expressed by the modulus-deviator stress relationship given in Figure 1.

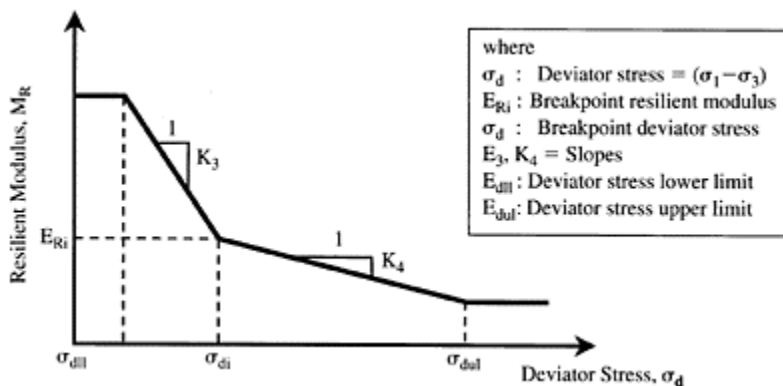


Figure 1. Stress dependency of fine-grained soils characterized by bilinear model.

As indicated by Thompson and Elliot (1985), the value of the resilient modulus at the breakpoint in the bilinear curve, E_{Ri} , (see Figure 1) can be used to classify fine-grained soils as being soft, medium or stiff.

3 PAVEMENT ANALYSIS USING ILLI-PAVE FINITE ELEMENT PROGRAM

Developed at the University of Illinois (Raad and Figueroa, 1980), ILLI-PAVE is an axisymmetric finite element (FE) program commonly used in the structural analysis of flexible pavements. The nonlinear, stress dependent resilient modulus geomaterial models summarized in the previous section are already incorporated into ILLI-PAVE. Numerous research studies have validated that the ILLI-PAVE model provides a realistic pavement structural response prediction for highway and airfield pavements (Thompson and Elliot, 1985; Thompson, 1992; Garg et al., 1998). Recent research at the Federal

Aviation Administration's Center of Excellence established at the University of Illinois also supported the development of a new, updated version of the program, now known as the ILLI-PAVE 2000 (Gomez-Ramirez et al., 2002).

ILLI-PAVE 2000 FE program was used in this study as the main validated nonlinear structural model for analyzing conventional flexible pavements with UABs. The goal was to establish a database of ILLI-PAVE response solutions that would eventually constitute the training and testing data sets for developing ANN-based structural models for the rapid forward and backcalculation analyses. For this purpose, a convergence study was performed to determine the domain size extent for the FE mesh discretization. A radial boundary placed at 25 times the contact area radius was sufficient to obtain convergence of deflections.

The top surface asphalt course was characterized as a linear elastic material with Young's Modulus, E_{AC} , and Poisson ratio, ν . Due to its simplicity and ease in model parameter evaluation, the K- θ model (Hicks and Monismith, 1971) was used as the nonlinear characterization model for the unbound aggregate layer. Based on the work of Rada and Witczak (1981) with a comprehensive granular material database, "K" and "n" model parameters can be correlated to characterize the nonlinear stress dependent behavior with only 1 model parameter using the following equation (Rada and Witczak, 1981):

$$\log_{10}(K) = 4.657 - 1.807 \cdot n \quad R^2 = 0.68; \text{SEE} = 0.22 \quad (3)$$

Accordingly, good quality granular materials, such as crushed stone, show higher K and lower n values, whereas the opposite applies for lower quality aggregates. Following the study by Rada and Witczak (1981), the K-values used typically ranged from 20.7 MPa (3 ksi) to 82.7 MPa (12 ksi) and the corresponding n-values were obtained from Equation 3.

Fine-grained soils were considered as "no-friction" but cohesion only materials and modeled using the bilinear or arithmetic model (see Figure 1) for modulus characterization. The breakpoint deviator stress, E_{Ri} , was the main input for subgrade soils. The K_3 and K_4 slopes shown in Figure 1 were taken as constants, 1,100 and 200, respectively, corresponding to medium soils given by Thompson and Elliott (1985). According to a comprehensive Illinois subgrade soil study by Thompson and Robnett (1979), the breakpoint deviator stress, σ_{di} , was taken as 41.4 kPa (6 psi) and 13.8 kPa (2 psi) was used for the lower limit deviator stress, σ_{dli} . The soil's unconfined compressive strength, Q_u , or cohesion was used to determine the upper limit deviator stress, σ_{dul} , (see Figure 1) as a function of the breakpoint deviator stress, E_{Ri} , using the following relationship (Thompson and Robnett, 1979):

$$\sigma_{dul}(\text{psi}) = 2 \times \text{cohesion}(\text{psi}) = Q_u(\text{psi}) = \frac{E_{Ri}(\text{ksi}) - 0.86}{0.307} \quad (4)$$

Therefore, asphalt concrete modulus, E_{AC} , granular base K- θ model parameter K, and the subgrade soil break point deviator stress, E_{Ri} , in the bilinear model were used as the layer stiffness

Table 1. Pavement geometry and material property/model inputs for ILLI-PAVE solutions.

Material type	Layer thickness	Material model	Layer modulus inputs	Poisson's ratio
Asphalt	h_{AC} =76 to 381 mm	Linear	E_{AC} =690 to 1 3,800 MPa	ν =0.35
concrete	(3 to 15 in.)	elastic	(100 to 2,000 ksi)	
Unbound	h_{GB} =102	Nonlinear	$M_R=K\theta^n$	ν =0.35 for
aggregate	to 559 mm	K- θ model	"K"= 20.7 to 82.7 MPa	$K\geq 34.5$ MPa (5 ksi)
base	(4 to 22 in.)		(3 to 12 ksi) "n" from Equation 3	ν =0.40 for $K>34.5$ MPa (5 ksi)
Fine-grained	7,620 mm (300 in.)	Nonlinear	$M_R=f(E_{Ri})$;	ν =0.45
subgrade	minus total pavement thickness	bilinear model	see Figure 1 E_{Ri} =6.9 to 96.5 MPa (1 to 14 ksi)	

inputs for all the different conventional flexible pavement ILLI-PAVE runs. The 40-kN (9-kip) wheel load was applied as a uniform pressure of 552 kPa (80 psi) over a circular area of radius 152 mm (6 in.). The thickness and moduli ranges used are also summarized in Table 1.

4 ARTIFICIAL NEURAL NETWORKS (ANNS) AS PAVEMENT ANALYSIS TOOLS

Backpropagation type artificial neural network models were trained in this study with the results from the ILLI-PAVE 2000 FE model and were used as rapid analysis design tools for predicting stresses and strains in flexible pavements. Backpropagation ANNs are very powerful and versatile networks that can be taught a mapping from one data space to another using a representative set of patterns/examples to be learned. The term "backpropagation network" actually refers to a multilayered, feed-forward neural network trained using an error backpropagation algorithm. The learning process performed by this algorithm is called "backpropagation learning" which is mainly an "error minimization technique" (see Haykin, 1999).

Artificial neural networks are valuable computational tools that are increasingly being used to solve resource-intensive complex problems as an alternative to using more traditional techniques. Meier et al. (1997) trained backpropagation type ANNs as surrogates for ELP analysis in a computer program for backcalculating pavement layer moduli and realized a 42 times increase in processing speed. In a recent successful application at the University of Illinois, Ceylan (2002) employed ANNs in the analysis of concrete pavement systems and developed ANN-based design tools that incorporated

state-of-the-art FE solutions into routine practical design in ways several orders of magnitude faster than those sophisticated finite element programs. Most recently, the research project team working on the development of the new mechanistic based AASHTO pavement design (NCHRP1–37A) in the U.S. have also recognized ANNs as nontraditional, yet very powerful computing techniques and took advantage of ANN models in preparing a mechanistic concrete pavement analysis package.

A total of 24,093 ILLI-PAVE FE runs were conducted by randomly choosing the pavement layer thicknesses and input variables within the given ranges in Table 1 to generate a knowledge database for ANN trainings. The total analysis depth of the pavement system was taken as 7,620 mm (300 in.). The subgrade thicknesses were calculated by subtracting the thicknesses of the AC and the base from the total analysis depth. The outputs recorded were the pavement surface deflection basin and the critical pavement responses, radial strain at the bottom of the AC layer (ϵ_{AC}), vertical strain on top of the subgrade (ϵ_{SG}), and the deviator stress on top of the subgrades layer (σ_D). To maintain a high level of accuracy in the results from all FE analyses, very similar ILLI-PAVE meshes were employed to have 266 to 494 elements with a total of 20 nodes used in the horizontal direction and 15 to 27 nodes used in the vertical direction. Ceylan (2002) recently highlighted the need to choose such consistent meshes for generating accurate FE solutions and as a result, successfully training ANN structural analysis models.

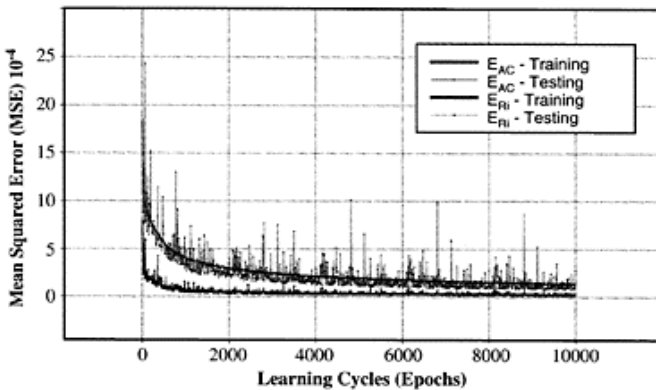


Figure 2. Training progress of ANN backcalculation model BCM-1.

Backpropagation type neural networks were used to develop three ANN structural models with different network architectures for predicting the pavement layer moduli (E_{AC} , KG_B , and E_{Ri}) and critical pavement responses (ϵ_{AC} , ϵ_{SG} , and σ_D) using the FWD deflection data. The FWD surface deflections (D_0 , D_8 , D_{12} , D_{18} , D_{24} , D_{36} , D_{48} , D_{60} , and D_{72}) are often collected at several different locations, at the drop location (0) and at radial offsets of 203-mm (8-in.), 254-mm (12-in.), 457-mm (18-in.), 610-mm (24-in.), 914-mm (36-in.), 1219-mm (48-in.), 1524-mm (60-in.), and 1829-mm (72-in.). For the modeling

work, surface deflections at these FWD sensor radial offsets were obtained from the ILLI-PAVE solutions and used as synthetic data to train ANNs.

The first backcalculation model, BCM-1, was designed to predict E_{AC} of the AC layer and the E_{Ri} value of the subgrade using only four pavement surface deflections, D_0 , D_{12} , D_{24} , and D_{36} , and two layer thicknesses, h_{AC} , h_{GB} . The ANN BCM-1 model therefore had 6 input parameters and 2 outputs, E_{AC} and E_{Ri} . A training data file was formed using the 24,093 ILLI-PAVE runs. One thousand of these runs were set aside for use as an independent testing set to conduct proper training and validate the performance of the trained ANN BCM-1 model. A neural network architecture with two hidden layers was exclusively chosen in accordance with the satisfactory results obtained previously with such networks considering their ability to better facilitate the nonlinear functional mapping (Ceylan, 2002).

Several network architectures with two hidden layers were trained. Overall, the training and testing mean squared errors (MSEs) decreased as the networks grew in size with increasing number of neurons in the hidden layers. The testing MSEs for the two output variables were, in general, slightly lower than the training ones. The error levels for both the training and testing sets matched closely when the number of hidden nodes approached 60 as in the case of 6–60–60–2 network architecture (6 input, 60 and 60 hidden, and 2 output nodes, respectively).

Figure 2 shows the training and testing MSE progress curves for the 6–60–60–2 network for 10,000 learning cycles or training epochs. The 6–60–60–2 architecture was chosen as the best architecture for the ANN BCM-1 model based on its lowest training and testing MSEs in the order of 1×10^{-4} (corresponding to a root mean squared error of 0.3%) for both output variables, E_{AC} and E_{Ri} . The almost constant MSEs obtained for the last 5,000 epochs (see Figure 2) indicate adequate training for this network. The testing curve is not as smooth as the training curve since MSEs are based on the averages of only 1,000 data in the independent testing test and 23,094 data in the training set, but the overlaying of the two curves shows that the network learned the functional mapping rather than memorizing the training set.

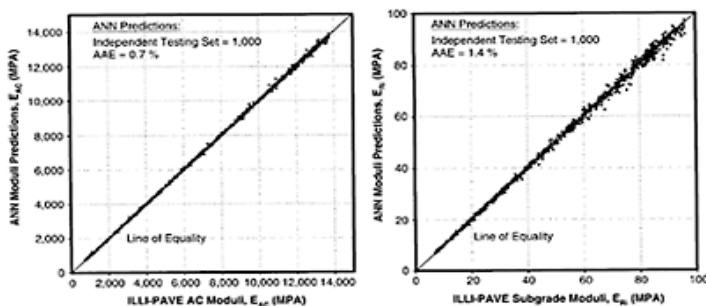


Figure 3. Prediction performance of the 6–60–60–2 BCM-1 network for 10,000 learning cycles.

by the 1,000 independent testing patterns. The AAE for the AC layer moduli was a low Figure 3 depicts the prediction ability of the 6–60–60–2 network at the 10,000th

training epoch. Average absolute errors (AAEs) were calculated as sum of the individual absolute errors divided 0.7% while the AAE for the subgrade breakpoint moduli E_{Ri} was only 1.4%. As shown in Figure 3, all 1,000 ANN predictions fell on the line of equality for the 2 pavement layer moduli thus indicating a proper training and excellent performance of the ANN BCM-1 model.

The development of a second backcalculation model ANN BCM-2 was deemed necessary for accurately predicting the K parameter of the $K\theta^n$ granular base model. The E_{AC} and E_{Ri} , already computed from the ANN BCM-1 model, were used as additional input variables in the BCM-2 model. The BCM-2 network architecture had 12 input variables (h_{AC} , h_{GB} , D_0 , D_8 , D_{12} , D_{24} , D_{36} , D_{48} , D_{60} , D_{72} , E_{AC} , and E_{Ri}) and a single output of K parameter. The trained ANN BCM-2 also had two hidden layers with 60 hidden nodes in each layer and successfully predicted the K values with a low AAE value of 3.4% after 10,000 learning cycles.

Next, using ILLI-PAVE solutions, a third backcalculation model, ANN BCM-3, was developed with the intention of directly predicting the critical pavement responses, ϵ_{AC} , ϵ_{SG} , and σ_D , from deflection basins. This approach eliminates the need of first predicting the pavement layer moduli and then using a forward calculation structural analysis model to compute the critical pavement responses. The directness of this approach can save time and effort in analyzing structural adequacy of field pavement sections from FWD data. Once validated with field data, the ANN model can predict ϵ_{AC} for AC fatigue condition evaluation in the field.

The ANN BCM-3 network architecture had 6 input variables (similar inputs as in the ANN BCM-1 model), two hidden layers with 60 hidden nodes in each layer, and 3 critical pavement responses, ϵ_{AC} , ϵ_{SG} , and σ_D , in the output layer. The AAE values from the ANN BCM-3 predictions were 0.5% and 1.8% for the asphalt radial strains (ϵ_{AC}) and the vertical compressive subgrade strains (ϵ_{SG}), respectively. The AAE value for the predicted subgrade deviator stresses (σ_D) was also 1.4%. Such low errors indicated the proper training and excellent prediction performance of the ANN BCM-3 backcalculation model trained for 10,000 learning cycles.

5 PERFORMANCES OF ANN PAVEMENT BACKCALCULATION MODELS

Six conventional flexible pavement sections were selected to further evaluate the performances of the ANN backcalculation models, BCM-1, BCM-2, and BCM-3. All pavement sections had a 102-mm (4-in.) AC underlain by a 305-mm (12-in.) granular base layer and applied with the 40-kN (9-kip) wheel load and 552 kPa (80 psi) uniform tire pressure. The AC layer moduli were kept constant at 1,379 MPa (200 ksi) with a constant Poisson's ratio of 0.35.

Table 2. Summary analysis results for the six pavements with unbound aggregate bases.

	Pavement section number					
	1	2	3	4	5	6

ILLI-PAVE FE calculations						
E_{AC} (MPa)—input	1,379	1,379	1,379	1,379	1,379	1,379
K (MPa)—input	27.6	27.6	27.6	62.1	62.1	62.1
E_{Ri} (MPa)—input	20.7	41.4	62.1	20.7	41.4	62.1
ϵ_{AC} ($\mu\epsilon$)—output (tension “+”)	426	411	400	355	346	340
ϵ_{SG} ($\mu\epsilon$)—output (compression “-”)	-1,058	-925	-792	-929	-798	-698
σ_D (kPa)—output (compression “-”)	-38	-42	-49	-34	-39	-43
ANN backcalculation model predictions						
E_{AC} (MPa)—(ANN BCM-1)	1,333	1,328	1,373	1,387	1,428	1,389
K (MPa)—(ANN BCM-2)	28.1	28.6	27.3	64.5	65.2	64.9
E_{Ri} (MPa)—(ANN BCM-1)	20.7	40.6	61.7	21.1	42.8	63.7
ϵ_{AC} ($\mu\epsilon$)—(ANN BCM-3)	425	408	400	352	344	339
ϵ_{SG} ($\mu\epsilon$)—(ANN BCM-3)	-1,009	-923	-763	-925	-797	-697
σ_D (kPa)—(ANN BCM-3)	-38	-40	-48	-34	-39	-43
BAKFEE ELP based results						
E_{AC} (MPa)—backcalc. output	1,498	1,579	1,497	1,553	1,551	1,555
E_{GB} (MPa)—backcalc. output	129.3	127.9	148.0	158.2	167.5	174.6
E_{SG} (MPa)—backcalc. output	60.4	86.7	103.7	66.9	91.5	115.7
ϵ_{AC} ($\mu\epsilon$)—forward calc. output	520	505	473	444	425	411
ϵ_{SG} ($\mu\epsilon$)—forward calc. output	-861	-688	-608	-748	-614	-525
σ_D (kPa)—forward calc. output	-52	-59	-63	-50	-56	-60
BAKFEE forward calculations with avg. nonlinear layer moduli						
E_{AC} (MPa)—avg. from ILLI-PAVE	1,379	1,379	1,379	1,379	1,379	1,379
E_{GB} (MPa)—avg. from ILLI-PAVE	193.4	201.0	207.5	223.4	229.2	233.7
E_{SG} (MPa)—avg. from ILLI-PAVE	40.4	57.7	76.6	42.1	59.3	77.2
ϵ_{AC} ($\mu\epsilon$)—forward calc. output	420	407	396	366	357	351
ϵ_{SG} ($\mu\epsilon$)—forward calc. output	-935	-788	-688	-857	-728	-635
σ_D (kPa)—forward calc. output	-38	-45	-52	-36	-43	-49

The pavements were first analyzed with the ILLI-PAVE finite element program. Two aggregate base K values of 27.6 and 62.1 MPa (4 and 9 ksi) and three subgrade E_{Ri} values of 20.7, 41.4, and 62.1 MPa (3, 6, and 9 ksi) were considered for a total factorial of 6 pavement sections analyzed. The rest of the nonlinear model parameters in the base and subgrade layers were assigned in accordance with the properties shown in Table 1. The pavement surface deflections obtained by ILLI-PAVE at 0, 203, 305, 610, 914, 1,219,

1,524, and 1,829 mm (0, 8, 12, 24, 36, 48, 60, and 72 in.) were treated as field measured FWD deflections and used for validating the ANN backcalculation model performances. These deflections were also used in an ELP based backcalculation program, BAKFAA, developed by the Federal Aviation Administration (FAA), to backcalculate the layer moduli (<http://www.airtech.tc.faa.gov/naptf/download/>).

Table 2 presents the inputs and output results of the ILLI-PAVE analyses given on the top section of Table 2. Also given next in Table 2 are the ANN backcalculation model predictions for the AC layer modulus E_{AC} , nonlinear parameters K and E_{Ri} , and the critical pavement responses ϵ_{AC} , ϵ_{SG} , and σ_D . Note that all ANN models produced very close results to the ILLI-PAVE input values and critical pavement responses. The AAE values were 2.1% for the six E_{AC} predictions and 1.7% for the six E_{Ri} predictions from the ANN BCM-1 model, 3.4% for the six K predictions from the ANN BCM-2 model, and 0.5%, 1.5%, and 1.3% for each of the six predicted critical pavement responses for ϵ_{AC} , ϵ_{SG} , and σ_D , respectively, from the ANN BCM-3 model. Clearly, the developed ANN models were quite successful in accurately mapping the nonlinear analysis ability of the ILLI-PAVE FE program into their connection weights and node biases. The AAE values for the predicted critical pavement responses were even lower than the ones obtained for the backcalculated layer properties, which suggests that it would be feasible to estimate, e.g., pavement fatigue life directly from the field measured FWD deflection data.

The ELP based BAKFAA program was used next to backcalculate the average elastic layer moduli from pavement surface deflections. Table 2 also summarizes the BAKFAA backcalculation and forward calculation results. The AAE values were rather high 12%, 170%, and 78% for each of the six predicted moduli for E_{AC} , E_{GB} , and E_{SG} , respectively. These layer moduli were then used as inputs for the forward calculation option of the BAKFAA program to compute critical pavement responses. The AAE values were 22%, 23%, and 39% for the average ϵ_{AC} , ϵ_{SG} , and σ_D , respectively, when compared to the actual ILLI-PAVE pavement responses.

Also presented at the bottom section of Table 2 are the results of additional BAKFAA forward calculation analyses. For all the 6 pavement sections, the subgrade and base layer moduli E_{SG} and E_{GB} were computed by averaging the actual ILLI-PAVE computed nonlinear (pavement centerline) modulus distributions with depth in the subgrade and base layers. These averaged moduli values, tabulated at the bottom section of Table 2, were then used as inputs in the forward calculation BAKFAA analyses to predict the critical pavement responses. This time, the AAE values were calculated as 2%, 11%, and 8% for the average ϵ_{AC} , ϵ_{SG} , and σ_D , respectively, much lower than the previous AAE values of 22%, 23%, and 39% from the ELP based backcalculation. Once again, the nonlinear pavement geomaterial behavior must be properly accounted for to accurately backcalculate layer moduli and predict critical pavement responses.

6 SUMMARY & CONCLUSIONS

Three artificial neural network (ANN) backcalculation models were developed using approximately 24 thousand nonlinear ILLI-PAVE finite element (FE) solutions. Unlike the linear elastic layered theory commonly used in pavement layer backcalculation, realistic nonlinear unbound aggregate base (UAB) and subgrade soil modulus models

were used in the ILLI-PAVE program to account for the typical stiffening behavior of UABs and the fine-grained subgrade soil moduli decreasing with increasing stress states. The ANN models developed successfully predicted the layer moduli and critical pavement responses computed by the ILLI-PAVE FE solutions and were much superior to the liner-elastic-layered forward and backcalculation analyses due to the nonlinear material characterization employed. Such ANN structural analysis models can provide pavement engineers and designers with sophisticated FE solutions, without the need for a high degree of expertise in the input and output of the problem, to rapidly analyze a large number of pavement deflection basins needed for routine pavement evaluation.

REFERENCES

- AASHTO T307-99. 2000. Determining the resilient modulus of soils and aggregate materials. In *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 20th Edition, AASHTO, Washington D.C.
- Brown, S.F. & Pappin, J.W. 1981. Analysis of pavements with granular bases. In *Transportation Research Record 810*, TRB, National Research Council, Washington, D.C., 17-23.
- Ceylan, H. 2002. *Analysis and design of concrete pavement systems using artificial neural networks*. Ph.D. Dissertation, University of Illinois at Urbana-Champaign, December.
- European CEN Std EN 13286-7. 2003. Unbound and hydraulically bound mixtures—Test methods—Part 7: Cyclic load triaxial test for unbound mixtures. European Standard, EC for Standardization.
- Garg, N., Tutumluer, E. & Thompson, M.R. 1998. Structural modeling concepts for the design of airport pavements for heavy aircraft. In *Proceedings, 5th International Conference on the Bearing Capacity of Roads and Airfields*, Trondheim, Norway.
- Gomez-Ramirez, F., Thompson, M.R. & Bejarano, M. 2002. ILLI-PAVE based flexible pavement design concepts for multiple wheel heavy gear load aircraft. In *Proceedings, 9th International Conference on Asphalt Pavements*, Copenhagen, Denmark.
- Haykin, S. 1999. *Neural networks: A comprehensive foundation*. Prentice-Hall, Inc., NJ, USA.
- Hicks, R.G. & Monismith, C.L. 1971. Factors influencing the resilient properties of granular materials. In *Transportation Research Record 345*, TRB, National Research Council, Washington, D.C., 15-31.
- Meier, R.W., Alexander, D.R. & Freeman, R. 1997. Using artificial neural networks as a forward approach to backcalculation. In *Transportation Research Record 1570*, TRB, National Research Council, Washington, D.C., 126-133.
- Raad, L. & Figueroa, J.L. 1980. Load response of transportation support systems. *Transportation Engineering Journal*, ASCE, 106(TE1).
- Rada, G. & Witczak, M.W. 1981. Comprehensive evaluation of laboratory resilient moduli results for granular material. In *Transportation Research Record 810*, TRB, National Research Council, Washington, D.C., 23-33.
- Thompson, M.R., 1992. ILLI-PAVE based conventional flexible pavement design procedure. In *Proceedings, 7th International Conference on Asphalt Pavements*, Nottingham, U.K.
- Thompson, M.R. & Elliott, R.P. 1985. ILLI PAVE based response algorithms for design of conventional flexible pavements. In *Transportation Research Record 1043*, TRB, National Research Council, Washington, D.C., 50-57.
- Thompson, M.R. & Robnett, Q.L. 1979. Resilient properties of subgrade soils. *Transportation Engineering Journal*, ASCE, 105(TE1).

Uzan, J., Witczak, M.W., Scullion, T. & Lytton, R.L. 1992. Development and validation of realistic pavement response models. In *Proceedings, 7th International Conference on Asphalt Pavements*, Nottingham, UK, Vol. 1, 334–350.

Measurement of road performance and impact on transportation operations with the *Opti-Grade* system

S.Mercier, M.Brown & Y.Provencher

*Forest Engineering Research Institute of Canada (FERIC), Pointe-Claire,
QC, Canada*

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ABSTRACT: Increasing costs have forced the forest industry to rationalize its road maintenance operations. Though the industry needs to control its maintenance costs, it must also avoid any negative impacts on trucking activities, which currently account for close to 50% of the industry's wood-procurement costs. To help the forest industry solve this problem, FERIC developed a modified approach to grading that focuses maintenance on the road segments that need grading most, thereby making the most efficient use of graders, lowering grading costs, and improving the performance of the roads.

Today, the system is used on a variety of operations to manage road maintenance, to facilitate research projects aimed at optimizing maintenance operations, to provide a decision-support tool for managing road rehabilitation, and to provide a means of evaluating how the quality of the running surface affects trucking costs.

FERIC worked with a company in western Canada (Alpac) to complete the first phase of a study to assess the impact of road roughness on haul costs. The study's main objective was to find the right balance between minimizing road maintenance costs and maximizing the efficiency of the transportation system. The potential benefits for the company were large, since according to Alpac, each reduction of 1 minute in cycle times translated into a saving of more than \$75 000 for their fleet. This paper presents a summary of the potential offered by *Opti-Grade* as well as the results of our ongoing research.

1 INTRODUCTION

FERIC is a non-profit institute that provides research and development services to the Canadian forest industry and to the federal and provincial governments. Research areas included in the scope of FERIC include harvesting, transportation, reforestation, and road construction and maintenance. Recent trends in the Canadian forest industry having a significant impact on FERIC's research focus is the continuous increase in haul distances and the trend towards partial cutting and dispersed cut blocks will require road networks to be extended even further and maintained longer in coming years. This situation imposes considerable pressure on the road maintenance team, who must control costs and ensure the safety of users of the roads despite the need to allocate dwindling resources over an increasingly large road network (Provencher, Méthot 1994).

Often representing more than 50% of the total procurement costs, roads and transportation are a significant portion of any forestry company's budget (Brown 2002). But these costs cannot be eliminated, since an effective road network is a crucial factor in the efficiency of any transportation system. The performance of the trucks that use that network (i.e., fuel consumption, cycle times, truck maintenance costs, and safety) are directly linked to the quality of the road network. To accurately estimate the costs associated with road maintenance and rehabilitation, managers must also understand the costs sustained by users of the road network and how these users are affected by road conditions. Unfortunately, the forest industry has only limited information on the actual cost of its trucking operations and no information on how these costs relate to road roughness. In its role as a provider of research results to the Canadian forest industry, FERIC undertook a study to examine the impact of road conditions on transportation costs in an effort to help the industry find the most economical balance between the need to minimize maintenance costs and the need to provide roads that sustain an efficient transportation system. This paper looks at some of the trends and results identified in our preliminary data analysis and describes how we plan to continue this research.

2 THE *OPTI-GRADE* SYSTEM

The *Opti-Grade* hardware consists of a datalogger that incorporates a global positioning system (GPS) receiver and a roughness sensor based on accelerometer technology (Mercier, Brown 2002). The equipment is mounted on a vehicle that uses the active road network regularly; in the forest industry, the ideal candidate is typically a haul truck. This approach lets the vehicle collect measurements during its regular travel without interfering with or interrupting normal operations. Thus, data collection imposes no additional ongoing costs.

The *Opti-Grade* software reads the data collected by the system's hardware and stores it in a database for each road network being managed. This network is defined by a digitized basemap that the software uses as the basis for its data analysis. This basemap lets *Opti-Grade* match the GPS data to known landmarks along the road. These landmarks can then be used as reference points that help managers to communicate

instructions to grader operators, as well as in the development of various reports on road conditions. *Opti-Grade* uses the most-recent stored information to produce grading schedules (Figure 1) in response to user-defined criteria, as well as reports on travel speeds and road roughness (Figure 2). (Data from previous trips by the vehicle carrying the *Opti-Grade* hardware are retained in the database and can be used to develop historical summaries of road conditions.)

The *Opti-Grade* software provides additional performance reports that can be used to evaluate the road and its users. These reports include summaries of the average roughness of the road and

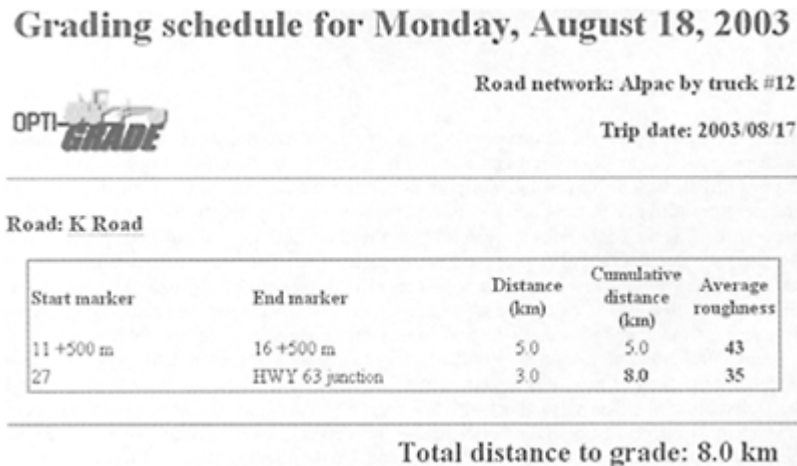


Figure 1. A typical grading schedule produced by *Opti-Grade*.

the average travel speed of the vehicle that carries the *Opti-Grade* hardware for each section of road and for any given day or time period. These reports help managers to evaluate the condition of the road because managers can determine what roughness levels appear to affect driving habits, including travel speeds. This knowledge lets managers make informed decisions about where to set the threshold values (for roughness) that will trigger grading.

The analysis process can also help road managers identify sections of the road that are causing traffic slowdowns unrelated to roughness, including problems related to road geometry or dust levels.

Based on these benefits and others, many Canadian forestry companies have begun implementing *Opti-Grade* in their operations to improve their management of road maintenance and their monitoring of the road's users. Although *Opti-Grade*'s use for daily or routine road maintenance offers a large payback, it also potentially offers more value as a first step towards developing a comprehensive road management system for Canadian forestry operations. Because *Opti-Grade* has now been widely accepted by the Canadian forest industry, FERIC is continuing to develop the system, and has begun to develop new ways to use the *Opti-Grade* data for road rehabilitation management.



Trip roughness report

Truck #12

Start marker: 5 (K Road)
End marker: 15 (K Road)

Start date: 2003/08/15
End date: 2003/08/15

Road: K Road

Marker	Average roughness loaded	Average roughness empty	Average roughness
5	11	12	11
6	11	10	10
7	11	9	10
8	12	9	10
9	12	9	11
10	13	11	12
END PAVE	58	36	46
11	78	25	54
12	86	27	58
13	60	24	44
14	44	23	33
15	50	31	36
Average:	37	19	28
Standard deviation:	29	10	19
Minimum:	11	9	10
Maximum:	86	36	58
Cycle time: 4h 59m 53s			
Idle time: 0h 59m 31s			

Figure 2. A typical *Opti-Grade* trip report.

3 CURRENT RESEARCH WITH *OPTI-GRADE*: EVALUATING TRANSPORTATION COSTS VS. ROAD MAINTENANCE COSTS

With increased use of *Opti-Grade* within the forest industry, road managers began to question the roughness level at which they should be maintaining their roads: Could they achieve the best payback by spending more on road maintenance and providing the best conditions possible for transportation, or would some lower level of maintenance provide a lower combined cost? Given the lack of supporting information on which to make this decision, some companies chose to use the new knowledge provided by *Opti-Grade* to reduce their road maintenance costs by focusing road work on those areas that needed it most; as a result, they maintained the road at an acceptable level while decreasing grader time. Others chose to maintain road maintenance efforts at current levels while improving the road's condition by focusing maintenance on interventions that would reduce haul costs. Both methods showed positive impacts, though companies weren't convinced that

either approach provided the maximum possible benefit. As a result, FERIC received a mandate from its members to determine the most economical balance between road maintenance costs and transportation costs. The approach involved determining what impact various road roughness levels and road maintenance standards would have on the operating costs sustained by the road's users.

3.1 Study design

To collect data on the impact of various road roughness levels on haul costs, we entered into collaboration with a forestry operation that was actively using *Opti-Grade* so we could track information on truck performance (e.g., fuel consumption, travel speeds, engine use). Since road maintenance needs are very different in winter (frozen roads with few roughness problems but with grading to preserve driving safety) and summer (grading primarily to reduce roughness), we limited our data collection to summer operations. For the analysis of operating costs as a function of road roughness, we included all data collected for our collaborator's fleet of trucks during normal work cycles. In addition to this first level of the analysis, we identified the most heavily used 20-km section of the company's road network and arranged to maintain this section at different roughness levels for 4- to 6-week periods throughout the road maintenance season.

3.2 Development of roughness

We designed the original trial to include five levels of road maintenance, but bad weather interrupted trucking operations and we were only able to monitor four roughness levels, each for a period of only 3 weeks. We plan to adjust our future maintenance criteria based on the preliminary results of the present study, and will continue data collection in the upcoming grading season. The evolution of the four roughness levels (from very high to very low) are presented in chronological order in Figure 3.

Table 1 provides a brief description of the four maintenance levels. These levels were established by analyzing the roughness data collected by the *Opti-Grade* system in the year before our study. In order to replicate these levels, we produced daily grading schedules and transmitted them to road maintenance personnel so they could adjust their grading operations accordingly.

3.3 Test location

Alberta-Pacific Forest Industries inc. (Alpac) has made a significant commitment to monitoring road maintenance and truck performance in its operations, and is one of the only companies in Canada that is currently using both the *Opti-Grade* system and onboard computers to monitor their vehicles. This made the company an ideal partner for our study. The mill site is located about 200 km northeast of Edmonton (Alberta), but the entire road network managed by the company covers approximately 1500 km.



Figure 3. Evolution of road maintenance levels under four different grading regimes.

Table 1. Maintenance levels used in the study.

Maintenance level	Description
Very low	Users complain frequently. Under normal operating conditions, less than 5% of the road would require grading.
Low	Users complain more often than average. Under normal operating conditions, 5 to 15% of the road would require grading.
High	Users rarely complain. Under normal operating conditions, 15 to 30% of the road would require grading.
Very high	No user complaints are received. Under normal operating conditions, more than 30% of the road would require grading.

3.4 Traffic levels

The traffic on the section of road in our study was a combination of haul trucks and passenger vehicles. Approximately 280 vehicles per day use this road, of which 180 are haul trucks. All the trucks in the fleet are equipped with central tire inflation (CTI) systems and haul eight-axle B-train trailers with a typical gross vehicle weight of 62500 kg.

4 PRELIMINARY RESULTS AND TRENDS

At the time this paper was written, we were still compiling the huge amount of data that had been collected for the vehicles. Nonetheless, our initial results suggest that free-flow vehicle travel speeds are only influenced by roughness at relatively high roughness levels. Figure 4 compares initial study roughness vs. speed trends to those presented by Paterson (Paterson, 1987). On gravel roads, no significant impact on travel speeds was

apparent until roughness was very high. This suggests that road conditions must be very poor before they will slow drivers on the unpaved road sections.

The change in average speed is noticeable when comparing summer to winter operations (Figure 5); travel speeds increase by around 3 km/h in winter, largely as a result of decreased variability in road conditions once roads have frozen. Once the road's surface and base layers are frozen they perform more like a rigid road than an unbound pavement. As a result, a higher proportion of the road network in summer operations would be in very poor condition (to the right of the Plateau portion of the graph in Figure 4) for a longer period than in winter operations.

When going from paved roads to unpaved roads (Figure 5), vehicle travel speeds decreased by roughly 10 km/h in summer operations and 5 km/h in winter operations. This observation also supports the assumption that frozen roads perform more like rigid roads (e.g., paved public roads), though the unpaved roads were nonetheless in poorer condition than the average paved section. We also observed that driver perception is critical in determining the impact of road conditions on travel speeds. In particular, we noted that a roughness level that would normally have no impact on travel speed can lead drivers to slow down if the rough stretch of road is preceded by a section of relatively smooth road. The initial shock of these large changes in roughness appears to motivate drivers to slow down initially but return to the normal cruising speed shortly thereafter even if the road continues to be rough.

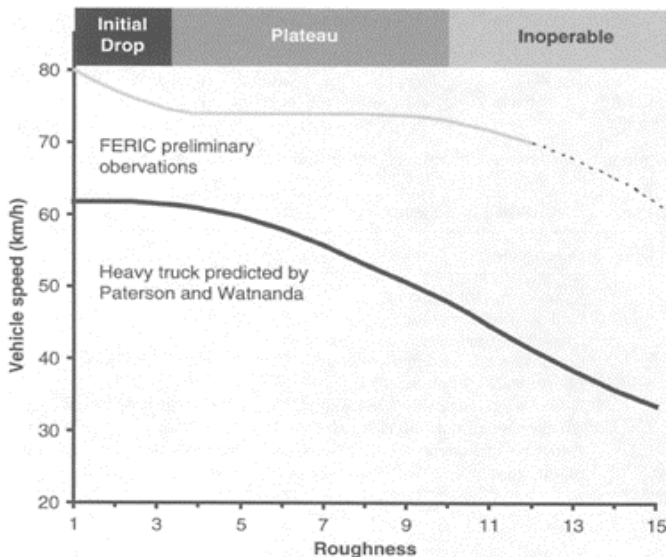


Figure 4. Initial data on the relationship between vehicle travel speed and road roughness.

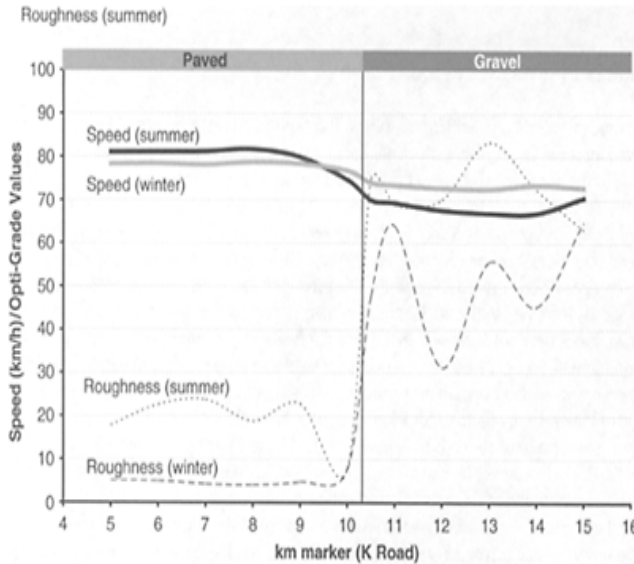


Figure 5. Change in travel speeds in summer and winter when transitioning between paved and unpaved roads.

5 FUTURE TEST PLANS

Based on the encouraging initial trends and results from our study, we plan to continue the trials with Alpac in the upcoming (2004) summer haul season. Given that the current data trends show a significant speed change over a relatively small range of roughness at the lower levels future trials will target this low roughness range. The goal will be to confirm the initial findings and to determine if gravel forestry roads can be maintained at these lower roughness levels where travel speed can be gained. Once the final data has been analyzed, we will propose a “best balance” (lowest overall cost) between transportation and road maintenance costs.

In addition, we will perform additional analyses to determine the paybacks from and value of various alternatives to grading, such as the use of thin pavements. Once the full set of data is available for analysis, we will examine other vehicle operating costs that can be tracked effectively, including fuel use, and will determine whether they are a function of road conditions. At the time this report was being written, analysis of fuel consumption rates had not yet begun, though we expect the relationship to be more linear than that of the relationship between travel speed and road roughness that has already been observed in our data. All these results will help us demonstrate additional value from using *Opti-Grade* as a decision-support tool that will help FERIC’s member companies further optimize their transportations costs.

6 CONCLUSION

Though preliminary the result to this point present some very interesting trends that can significantly impact how forest companies manage their roads. On the gravel portions of the road network no significant impact on travel speed was observed until the road reaches very high roughness levels, well beyond the normally accepted conditions. Significant gains in user efficiency seem to be limited to surfaced roads that are typically at roughness levels that are not maintainable with unsurfaced roads, meaning forest companies may be better served by exploring the application of surface treatments on key road segments rather than increased grading budgets over the whole road network. The data collected also shows a travel speed increase in winter over summer on the gravel roads and that frozen gravel roads in winter had performance closer to the paved roads in the network. This is attributed to the fact that the road is frozen with a much more stable base and surface that does not have the same roughness deterioration as unfrozen roads. For a forest company this may mean that road budgets may be better used for extensive stabilization of key road segments rather than increased grading over the whole road network. The future study plans will allow us to confirm these trends, evaluate at what cost for grading a gravel road can be maintained in a condition that will allow for increased travel speed and provide best road management practices and decision processes to *Opti-Grade* users.

REFERENCES

- Brown, M. 2002. Managing the rehabilitation of forest roads using historical daily roughness data collected by the *Opti-Grade* system. Memoire presented at the Ecole de Technologie Supérieure, Montreal, Quebec. Université du Quebec, Montreal, Que. 92 p.
- Mercier, S.; Brown, M. 2002. The *Opti-Grade* grading-management system. Forest Engineering Research Institute of Canada (FERIC), Pointe-Claire, Que. Advantage 3(17). 4 p.
- Paterson, W. 1987. Road Deterioration and Maintenance Effects: Models for Planning and Management. The Highway Design and maintenance standards series, World Bank, Transportation Department, Washington, D.C., 416 p.
- Provencher, Y.; Méthot, L. 1994. Controlling road surface conditions by management of grading. Forest Engineering Research Institute of Canada (FERIC), Pointe-Claire, Que. Technical Report TR-110. 9 p.

Adaptation of a grading management system for unsealed road networks in New Zealand

R.A.Douglas

*Forest Engineering, School of Forestry, University of Canterbury,
Christchurch, New Zealand*

S.A.Mitchell & B.D.Pidwerbesky

Fulton Hogan Ltd., Christchurch, New Zealand

Pavements Unbound—Dawson (ed.)

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ABSTRACT: A case history on the adaptation of a road grading management system, Optigrade, is presented. The system, developed by the Forest Engineering Research Institute of Canada (FERIC), facilitates maintenance management by identifying those particular segments of a road needing grading. The system records road roughness, measured by an accelerometer mounted on a haul truck or utility vehicle, together with the vehicle's position, at 5-second intervals. The roughness—location data is subsequently analysed with the system's software to produce reports and maps of roughness and speed along the road, together with grading schedules indicating which segments of the road should be graded or otherwise maintained based on road roughness. The system was originally developed for long private forest haul roads with just a few tributaries arranged in dendritic fashion but the authors have adapted it for use on large (circa 1200 km) grids of public roads. This application created many new challenges, which the paper describes. Solutions to the problems are presented.

1 INTRODUCTION

New Zealand presents unique demographic and technical conditions to the pavement engineer. The total population is 4 million, and is concentrated in Auckland, with over a million people. The South Island has approximately 800,000 people, with 300,000 of that in one city, Christchurch. The total network of roads amounts to nearly 100,000 km, of which nearly 90% is classed as "local roads" (i.e. not State Highway) (Transfund 2003a). While on average 50% of the local road network in the country is sealed, almost all with chip seal, the percentage is as low as 16% in some districts. Local roads are usually low

volume roads (LVR): about 75% of the national network carries fewer than 500 vehicles per day, and 45%, fewer than 100 vehicles per day.

Given that the top three industries contributing to net export receipts are agriculture, tourism, and forestry, the picture is one of two extremes. Either roads are of low standard and low volume, unsealed or chip-sealed, serving sparsely populated countryside and supporting the heavy vehicles associated with primary industries, or, in the case of two or three urban centres, roads have high standard pavements and carry the high traffic volumes associated with heavily populated areas. This paper is concerned with the former conditions, presenting a case history on the management of the maintenance of these roads.

2 LVR PAVEMENT MANAGEMENT IN NEW ZEALAND

In New Zealand, the allocation of public funds to passenger transport and local roads has been given to a Crown entity known as Transfund New Zealand (Transfund 2003b), which has a board

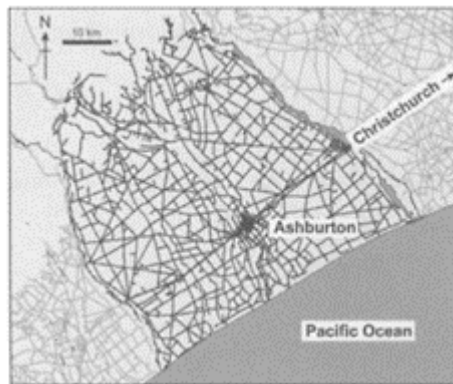


Figure 1. Ashburton road network, South Island, New Zealand.

directly accountable to the Minister of Transport. Since the 1990s, all design, supervision, construction and maintenance work on public roads has been outsourced, as required by an Act of Parliament.

More recently, the introduction of long-term performance-based maintenance contracts has encouraged contractors to significantly enhance their asset management capability. This requires contractors to change from a predominantly production oriented philosophy to an understanding of what an asset requires, and when and by what means treatment should be implemented in order to achieve full service delivery. District reading authorities expect the road asset condition to be maintained to agreed levels for service at least cost. Longer term contracts—five to ten years compared with the 3-year contracts of the early- to mid 1990s—together with Transfund New Zealand permitting

“bulk” funding, enables contractors to apply the most cost effective mix of maintenance treatments over a reasonable time span.

Road maintenance contracts are let on a district basis. Local road maintenance contracts embrace networks of up to 4700 km. One of the largest is in the Ashburton District of the South Island (Figure 1), with a total length of 2470 km, of which 1300 km is unsealed.

Fulton Hogan maintains the Ashburton unsealed road network under contract with the district council. In searching for a management tool which presented proven technology in a complete package of software and hardware, it selected the Optigrade system developed by the Forest Engineering Research Institute of Canada, over the one or two other similar systems available.

3 A SYSTEM FOR MAINTENANCE MANAGEMENT

The “classical” pavement management systems (PMS) approach has it that pavements express four “outputs” (Haas et al. 1994): safety (usually in terms of skid resistance), structural capacity, distress, and riding comfort (road roughness). Managers monitor the changes in each of these four measures, and schedule maintenance or rehabilitation treatments when one or more of the outputs hits some pre-determined threshold. When adapting PMS to low volume roads, the economics of the situation and other considerations call for certain changes and a streamlining of the approach (Douglas 1999, Douglas and McCormack 1995 & 1997). The hardware for structural capacity testing is usually far too expensive in this context and conventional skid resistance testing does not apply to the loose surfaces of unsealed roads. Manual distress testing can be done, but it is usually considered too labour intensive and not cost efficient. On the other hand, monitoring roughness is convenient and inexpensive. In addition, roughness can be seen as an indicator of other problems, as for instance it is linked to some distresses.

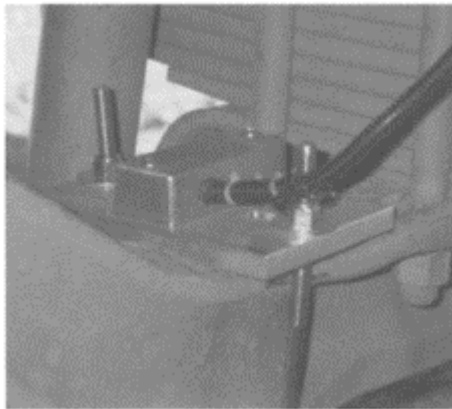


Figure 2. Accelerometer mounted on front axle, (photo: S.Mercier, FERIC).

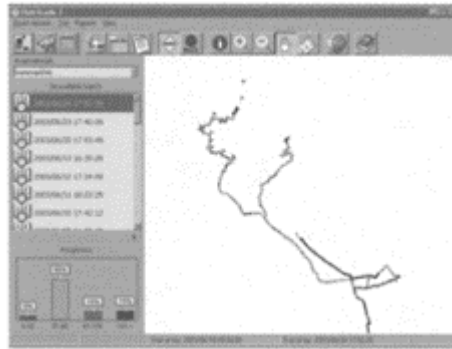


Figure 3. Colour coded roughness map.

The essence of the Optigrade road maintenance management system is that road roughness is used as a measure of road performance (Provencher and Méthot 1994, Provencher 1997, FERIC 2003). A roughness threshold is set as a trigger for road maintenance, particularly grading. Roughness is measured routinely using an accelerometer mounted on the axle of a haul truck using the road (Figure 2). Alternately, some road owners mount the accelerometer on a smaller utility vehicle dedicated to that purpose. The accelerometer measures the vertical excitation of the suspension element at a frequency of 100 Hz, and the peak value in each 5-second interval is recorded. Simultaneously, the position obtained by an on-board GPS receiver is recorded. Records of date, time, position, roughness and instantaneous vehicle speed are written to a flash card in a data logger on board the vehicle, to be downloaded and analysed later. The most recently released version of the system facilitates automatic remote downloading through a radio interface (FERIC 2003).

The system's software is designed to produce maps and reports. Colour coded maps of roughness and speed along the road can be displayed (Figure 3) and reports detailing road roughness or vehicle speed adjacent to user-defined way points can be produced. The data can be analysed with respect to a user-defined roughness threshold to produce grading reports (Figure 4), showing which segments of the road are due for grading. The key to the approach is that only those segments of the road which really require grading, based on road roughness, are scheduled for grading, thus increasing the efficiency of the maintenance effort.

4 ADAPTATION OF OPTIGRADE

It is noteworthy that Optigrade was developed for long haul roads, with just a handful of tributary roads, arranged in dendritic fashion. A base map must be created, so that the system can reference the vehicle position data against known locations, and so that grading reports can be created with starting and ending points referenced to the base map way points. Figure 5 shows a typical base map for this context.

The situation presented by a network of public roads—essentially a grid (Figure 1)—is vastly different from the situation that was assumed in Optigrade’s development (Figure 5). If all the benefits to be obtained from the use of the system were to be available in the new context, the method of referencing road segments would have to be modified.

In the forestry context, it is advantageous to mount the hardware for the system on a typical haul truck which travels the road network. In the public road context, there is no analogue. At one stage, mounting the hardware on a rural mail delivery van was contemplated, but it was quickly realised

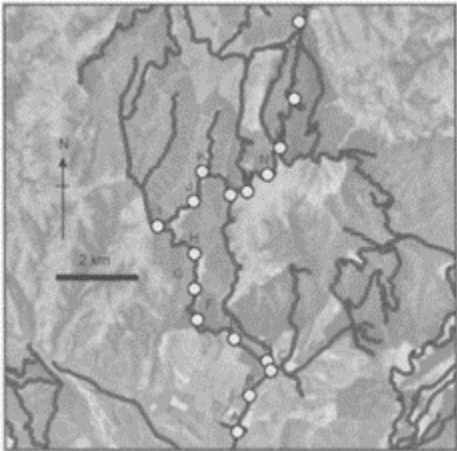


Figure 5. Base map for forest road.

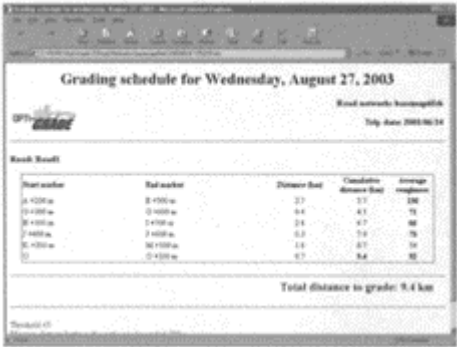


Figure 4. Grading report.

that such a vehicle likely covers nearly as much distance on the shoulder of the road as it does on the road itself, so it could hardly be used to obtain a representative roughness for the road.

The following paragraphs discuss the solutions to these and other challenges that came up, as well as the implications of the solutions, using the results of a trial conducted in the southern corner of the Ashburton district road network (Figure 1).

5 DISCUSSION

5.1 *Base maps and way points*

Clearly, in order to reference the greater number of segments in a grid network, a significantly greater number of base map way points would have to be collected. The recommended frequency is one point every kilometre, with a point at the start and end of each road segment. For a typical network such as Ashburton's, at least a thousand points would be needed. Early on, to save what was perceived to be excessive field time, it was thought the way point locations could be easily extracted from a GIS map of the network. However, it was found that a great deal of useful field information would be lost if such a remote method of creating the base map was used. As an example, it would be useful to have a way point at a heavily used factory driveway connecting to the road network, but it would be unlikely that such a point would be set, using the remote, GIS-based approach. In addition, the GIS-based approach proved not to be significantly quicker than data collection in the field with GPS.

The practice adopted was to drive the full network and record the start and end way points for each segment, together with way points for additional features of significance (e.g. bridges, large culverts, driveways, etc.). In a trial, it took about a day to collect and subsequently process the data for the subset of the Ashburton network (190 km, 180 road segments, 217 way points).

On the analysis side of the problem, there still was the difficulty of isolating, extracting, and analysing the roughness data for individual road segments within the road network. The original Optigrade software did not facilitate this, given that it was written with single roads in mind. Location was simply a matter of referring to distance offsets.

The short term solution FERIC provided was to modify the display of the base map. The identification number assigned internally to each 5-second data collection point (i.e. position, roughness and speed) assumed much greater importance. The i.d. number for a point on the output map now pops up when the cursor is held over the point (Figure 6). Via the i.d. number (4356 in Fig. 6), the analyst can determine the start and end roughness records within the overall data set that are

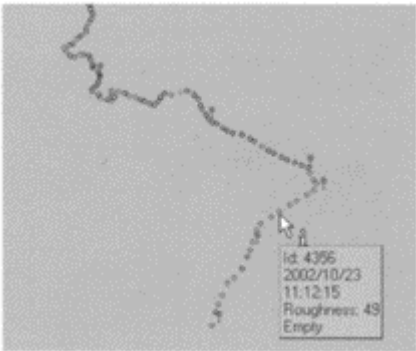


Figure 6. Identification of an individual data point on an Optigrade base map.

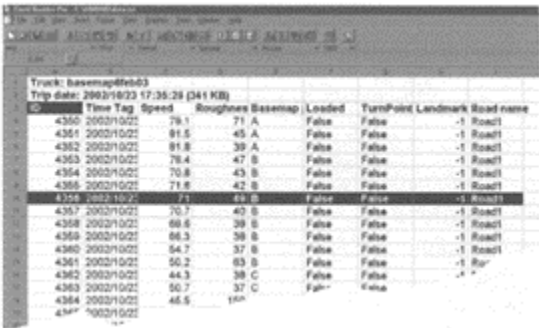


Figure 7. Optigrade data file, open in a spreadsheet, with data for the point with i.d. 4356 highlighted (see Fig. 6). Point 4356 could be the starting point for a particular road segment.

applicable to a particular road segment. Those records can be isolated within the overall data set (Figure 7), and graphs of roughness vs. distance along the particular segment produced, along with summary statistics, using conventional spreadsheet software.

In addition, a newer version of the software and hardware now permits the field engineer to flag particular features of interest during roughness data collection, for example, a single severe pothole which would affect the interpretation of the results. A record of each exceptional feature can be written into the data file, to be noticed when analysis is carried out later in the office.

In the longer term, it is hoped that the Optigrade software can be rewritten to isolate the data for specific road segments directly and graph the results without resort to separate spreadsheet software.

5.2 Data collection protocols

A decision had to be made regarding the speed to be maintained by the monitoring vehicle as the data was collected. Inputs to the decision included safety, data collection precision, the possibility of a roughness—speed relationship affecting repeatability, and field production.

It is best if the monitoring vehicle maintains a constant speed. From the safety point of view, that speed obviously should not exceed what is safe for the road geometry presented to the vehicle, and it should not be significantly different from the speed of any surrounding traffic. Faster speeds mean reduced data precision, as the vehicle covers a greater distance during the interval between data points. It was known that there is at least a weak relationship between recorded roughness and speed, so a data logging speed representative of the general traffic's speed was desirable, and it was desirable to select a speed that could be easily repeated on subsequent visits to the site. Finally, the faster the data logging speed, the more road that could be logged in a shift or a day.

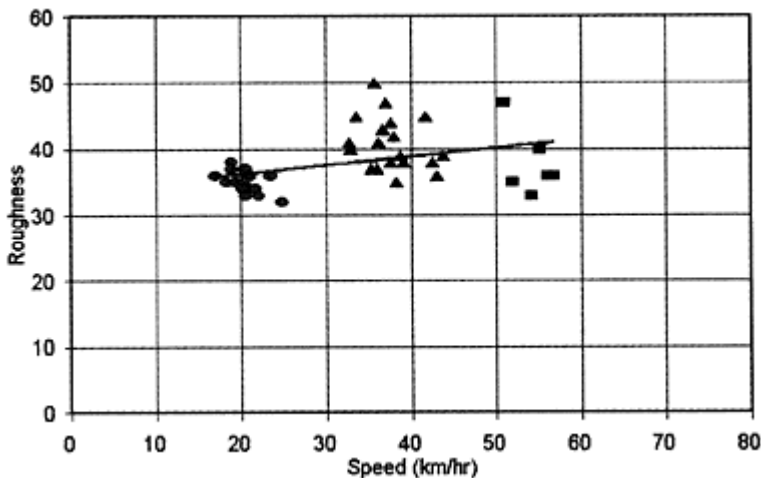


Figure 8. Roughness—speed relationship, trial of a specific heavy logging truck on a particular unsealed road.

A speed of 80 km/hr was selected to satisfy these requirements. Except in a few locations on the Ashburton network, that speed was safe for the field engineer and for the surrounding traffic. At 80 km/hr, with a 5-second recording interval, the data logging interval is 111 m, not considered excessive. And at that speed, it would theoretically take

just over 16 hours to log the full unsealed network of 1300 km, plus additional time travelling between more distant segments, and time for breaks, which seemed reasonable.

5.3 *Monitoring vehicle*

The platform for the assessment of public roads is the field engineer's utility vehicle. Using a lighter vehicle brings a new set of variables into the picture, compared to the circumstances originally envisaged when Optigrade was developed. The vehicle weight and suspension characteristics clearly have an effect on the recorded roughness, which is actually the response of the vehicle's suspension to the road profile. The standard calibration routines for the system account for initial differences in monitoring vehicles (utility vehicles vs. logging trucks), but it is imperative that the characteristics of the vehicle subsequently not change, or that calibration is repeated if they do change. Such changes arise in alterations in weight of the vehicle (carrying a heavier load in the utility vehicle for unrelated field work that day, for instance), tyre inflation pressure, or deterioration of the shock absorbers. Thus frequent re-calibration of the system is necessary, along the lines of what is needed for conventional roughness equipment such as that for IRI or NAASRA testing.

5.4 *Roughness—speed relationships and implications*

It would be desirable to find that the system's roughness output is independent of vehicle speed. However, a weak roughness—speed relationship was noticed in previous work with heavy haul trucks. The linear regression coefficient between roughness and speed for a particular trial was 0.132 Optigrade roughness units per km/hr (Figure 8), for the speed range 20 to 55 km/hr.

It was anticipated that the relationship would be stronger for a utility vehicle, so a second set of trials was conducted (Figure 9). The regression coefficients for the two trials were 0.31 and 0.42 Optigrade roughness units per km/hr over the range 35 to 100 km/hr, indicating that the utility vehicle's suspension was livelier, responding with more sensitivity to the road roughness.

That being the case, it became a necessity to take the speed of data logging into account deliberately. If the speed of the monitoring vehicle deviates significantly from the target speed of

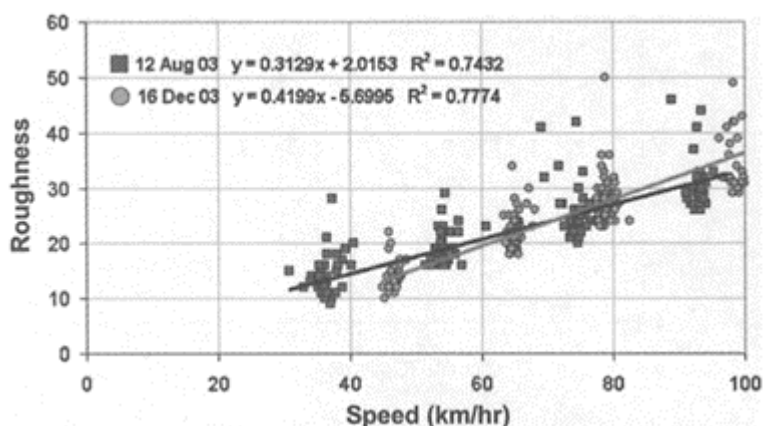


Figure 9. Roughness—speed relationships for a specific utility vehicle on a particular unsealed road.

Table 1. Summary statistics, Ashburton network subset.

Statistic	15/08/2003	16/12/2003
Average speed (km/hr)	68.4	63.6
Average roughness (Optigrade roughness units)	68	69
Maximum roughness (Optigrade roughness units)	195	165
Standard deviation (Optigrade roughness units)	25	21

Table 2. Distribution of roughness across Ashburton network subset.

Roughness (Optigrade roughness units)	Percentage of network	
	15/08/2003	16/12/2003
0–40	13	7
41–65	33	33
66–85	33	42
85 +	21	18

80 km/hr, the regression coefficient is used to correct the recorded roughness back to that for the standard speed of 80 km/hr.

5.5 Use of the system in maintenance management contracts

As its use for public local road maintenance management has developed, the Optigrade system is being employed more as a roughness measuring tool than a grading scheduling tool. Its ability to produce roughness results across the network is being exploited. Typical results for the subset of the Ashburton network are shown in Tables 1 and 2, for two dates of testing approximately four months apart.

A key concept in the management of the maintenance of an unsealed road network to understand is that at any given time, there will be a distribution of roughness values across the network. This distribution can be altered with various treatments applied to individual segments of the network.

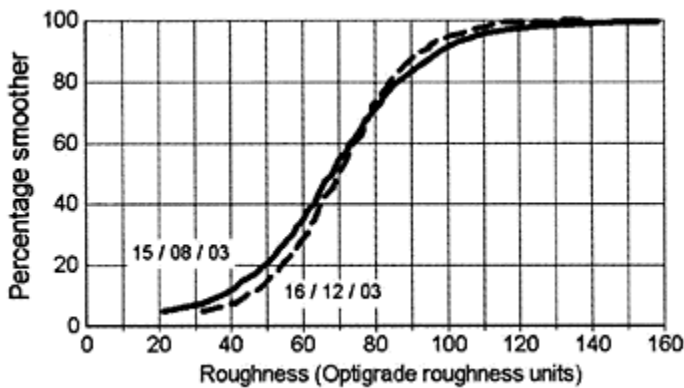


Figure 10. Asset condition profile for the subset of the Ashburton unsealed road network.

When the percentage of the network at a given level of roughness is plotted against the roughness value, an “asset condition profile” can be produced (Engelke 2003). Profiles for the two sets of data are plotted in Figure 10. The differences in the curves depict the change in overall network roughness more clearly than do just the statistics such as average network roughness and standard deviation. In Figure 10, it can be noted that the network became slightly more uniform between August and December.

In the long run, it is envisaged that district councils and contractors will agree on envelopes for the asset condition profiles, akin to grain size distribution envelopes in aggregate testing. The “rough edge” of the envelope will represent the maximum tolerable roughness for the network (or road segment, for that matter), whereas the “smooth edge” of the envelope will represent the distribution of roughness beyond which further improvement is not considered economic.

As this new approach to maintenance management is implemented, there will need to be serious discussion about what the envelope values should be, how the envelopes should vary from one network to another taking surrounding land use into consideration,

perhaps if and how the envelopes should vary over time as the economics change and/or user expectations evolve, and how uniform the asset condition profiles should be across the network and for individual road segments. The concept as applied to the management of unsealed roads and low volume roads is in its very early stages.

6 CONCLUSIONS

Based on the work outlined in this paper, the following may be concluded:

- Optigrade is a proven, commercial system designed to facilitate the management of the maintenance of long, uncomplicated dendritic networks of haul roads with few tributaries.
- The system has been successfully modified to cater to the management of public low volume roads in grid networks.
- The system has been extended beyond being a grading management tool, to become a road condition monitoring tool.
- The concept of the asset condition profile is a valuable one in the management of sealed and unsealed low volume roads. It warrants much greater development.

An example of the benefits of outsourcing road maintenance to private contractors is shown by the fact that it was the contractor that identified the need for a low cost roughness measurement tool to provide an objective operational performance indicator for rural district roads, initiated the development described in this paper, and implemented its use.

ACKNOWLEDGEMENTS

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REFERENCES

- Douglas, R.A. 1999. *Forest roads, resource access roads—delivery, the transportation of raw natural resource products from roadside to mill*. Christchurch, N.Z.: Forest Engineering, NZ School of Forestry. 202 pp.
- Douglas, R.A., and McCormack, R. 1995. Pavement management systems appropriate to forest haul road networks. *Proceedings XX World Congress, International Union of Forestry Research Organisations (IUFRO)*, Tampere, Finland. August 6–12. 4 pp.
- Douglas, R.A., and McCormack, R. 1997. Pavement management systems applied to forest roads. *Proceedings, 1997 Annual International Meeting, American Society of Agricultural Engineers (ASAE)*. Paper No. 975031. 8 pp.
- Engelke, T. 2003. Long term performance based road maintenance contracts in Western Australia. *Proceedings Bay Roads Exposed Conference*, Rotorua, N.Z., April 27–29, 2003. 27 pp.

- FERIC. 2003. *Optimizing your grading operations: the Opti-Grade and Opti-Grade RF systems*. <http://www.feric.ca/en/ed/html/opti-grade.htm>. Web site visited 13/02/2004.
- Haas, R., Hudson, W.R., and Zaniewski, J. 1994. *Modern pavement management*. Malabar, Florida: Krieger Publishing Company. 583 pp. ISBN 0-89464-588-9.
- Provencher, Y. 1997. A pavement management system for forestry road networks. *Proceedings of the International Symposium: Thin Pavements, Surface Treatments, and Unbound Roads*, University of New Brunswick, Fredericton, New Brunswick Canada, June 24-25, 1997. pp. 265-272. ISBN 1-55131-038-4.
- Provencher, Y, and Méthot, L. 1994. Controlling the state of the road surface through grading. *Technical Report TR-110*. Pointe-Claire, Quebec: Forest Engineering Research Institute of Canada (FERIC). 9 pp.
- Transfund NZ. 2003(a). *Roading statistics 2002/03* <http://www.transfund.govt.nz/pubs/RoadStats2003.pdf>. Web site visited 13/02/2004.
- Transfund NZ. 2003(b). *Working with our partners to make the greatest possible progress towards New Zealand's transport goals*. <http://www.transfund.govt.nz/intro.html>. Web site visited 13/02/2004.

Design of thin and unsealed pavements

Deformation behaviour of granular pavements

G.Arnold

Transit New Zealand, Wellington, New Zealand

A.Dawson

*Nottingham Centre for Pavement Engineering, University of Nottingham,
Nottingham, UK*

D.Hughes & D.Robinson

School of Civil Engineering, Queens University Belfast, Belfast, UK

Pavements Unbound—Dawson (ed.)

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ABSTRACT: The Repeat Load Tri-axial (RLT) apparatus can approximate the loading experienced by a material element in a pavement. RLT tests for at least 50,000 load cycles gives an indication of a materials resistance to permanent strain (deformation/rutting) at the stress level tested. A series of RLT permanent strain tests were conducted on two Northern Ireland unbound granular materials (UGMs) and four New Zealand materials used in pavement tests. The results at each testing stress of accumulated permanent strain versus load cycles were categorised into 3 types of behaviour ranges (A, B and C). Range A is where the incremental increase of permanent strain for each load cycle is decreasing (i.e. stable behaviour). Collapse of the RLT specimen or unstable behaviour (rate of permanent strain is increasing) is categorised as range C and range B is behaviour between the two extremes A and C. For pavement design purposes the Range A behaviour case is ideal. The ABAQUS finite element package was used to apply the Range A behaviour criteria. The granular material was assigned a yield line that represented the boundary between Range A and B behaviour. Various pavements of different asphalt and granular depths were analysed. Results were contours of permanent strain showing regions in the pavement that had yielded. The total amount of yielding was quantified as a total permanent surface deformation. It was found that asphalt cover thicknesses of around 200 mm (the actual thickness depended on the granular material type) showed minimal amounts of yielding and thus Range A or stable behaviour in the granular material would be predicted. Other results of interest from this analysis showed how the regions in the pavement that exhibited the highest amount of permanent strain shifted depending on the asphalt and granular thickness. For thin asphalt cover

(<25 mm) and thick granular layers (>600 mm) the maximum amount of permanent strain occurs in the granular material at a depth of 150 mm while the subgrade exhibited nil permanent strain.

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 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 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 in the ABAQUS finite element package. This design method utilises results from RLT permanent strain tests to modify the Drucker-Prager yield criteria to reflect a stress boundary between different rutting behaviours defined as shakedown behaviour ranges (e.g. reducing or constant rate of rutting with increasing load cycles).

2 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 are illustrated in Figure 1 and 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

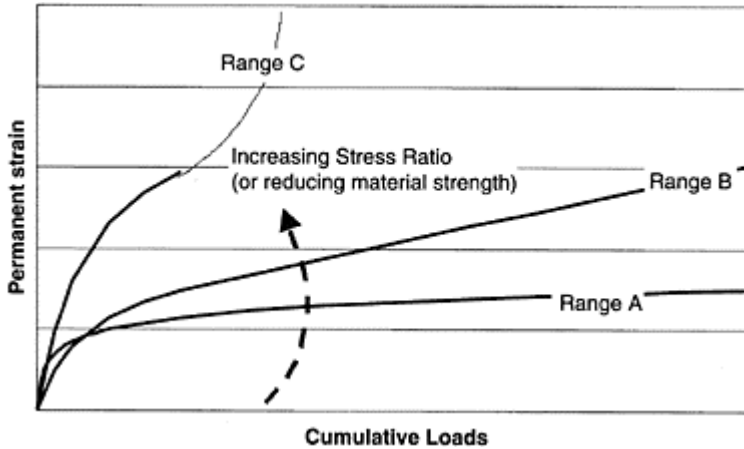


Figure 1. Shakedown range behaviours for permanent strain versus cumulative loading.

that cause the various shakedown ranges for use in defining stress boundaries between the various behaviour types. 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.

3 RLT PERMANENT STRAIN TESTS

A series of Repeated Load Tri-axial (RLT) permanent strain tests were conducted on six unbound granular materials (UGMs) and one silty-clay subgrade soil as detailed in Table 1. The aggregates chosen for testing were those used in a Northern Ireland road trial and in full scale accelerated pavement tests at CAPTIF located in Christchurch New Zealand. This allowed validation of any predictions of rutting behaviour to actual field trial results.

The aim of the RLT tests 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. It is usual to use a new specimen per permanent strain RLT test at a particular stress level because of the effects of stress history. However, to save time, multi-stage permanent strain tests, where several different stress conditions on the same specimen are conducted were employed. For the same sample, maximum p (mean normal stress) was kept constant while maximum 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 at 3 values of maximum p of 75, 150 and 250 kPa. Multi-stage testing was considered appropriate when the aim is to determine the type of permanent strain behaviour (i.e. range A, B or C) for each stress level. It is considered that stress history is

Table 1. Materials tested in the Repeat Load Tri-axial apparatus.

Material name	Description
NI Good	Premium quality crushed rock—graded aggregate with a maximum particle size of 40 mm from Bandridge, Northern Ireland, UK.
NI Poor	Low quality crushed quarry waste rock—graded aggregate (red in colour) with a maximum particle size of 40 mm from Banbridge, Northern Ireland, UK.
CAPTIF 1	Premium quality crushed rock—graded aggregate with a maximum particle size of 40 mm from Christchurch, New Zealand.
CAPTIF 2	Same as CAPTIF 1 but contaminated with 10% by mass of silty clay fines.
CAPTIF 3	Australian class 2 premium crushed rock—graded aggregate with a maximum particle size of 20 mm from Montrose, Victoria, Australia.
CAPTIF 4	Premium quality crushed rock—graded aggregate with a maximum particle size of 20 mm from Christchurch, New Zealand.
CAPTIF Subgrade	Silty clay soil used as the subgrade for tests at CAPTIF from Christchurch, New Zealand.

more likely to affect the magnitude of permanent strain rather than the type of behaviour, however this point will require further research.

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 2. To determine which stress level caused the various shakedown ranges (A, B or C) cumulative permanent strain versus permanent strain rate plots were produced (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).

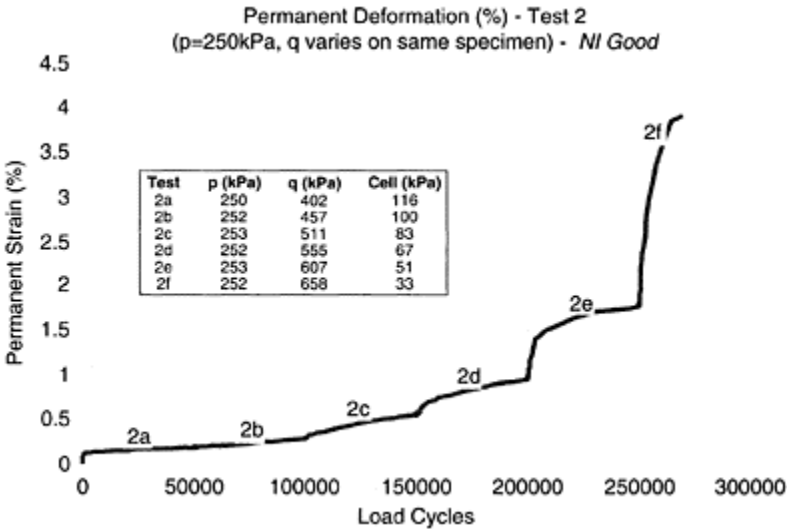


Figure 2. Typical RLT permanent strain test result for NI Good UGM (NB: stress levels shown are maximum values).

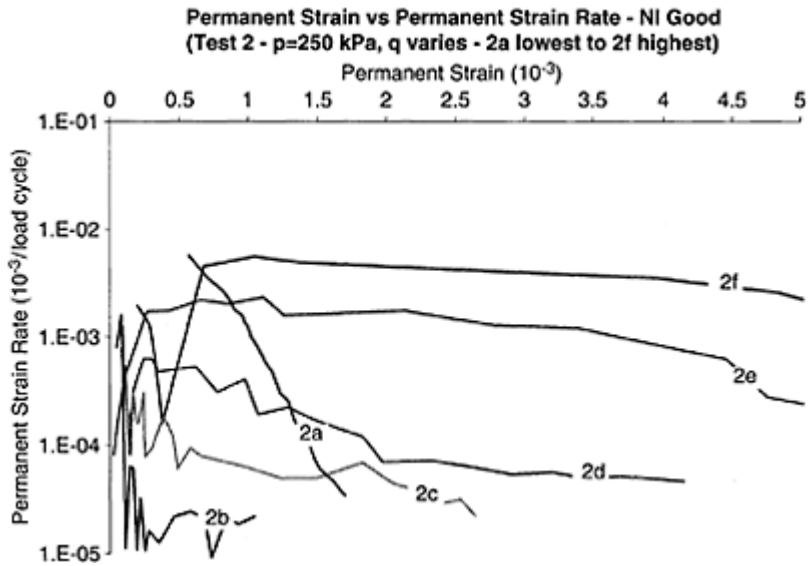


Figure 3. Cumulative permanent strain versus permanent strain rate for NI Good UGM test 2 (Figure 2).

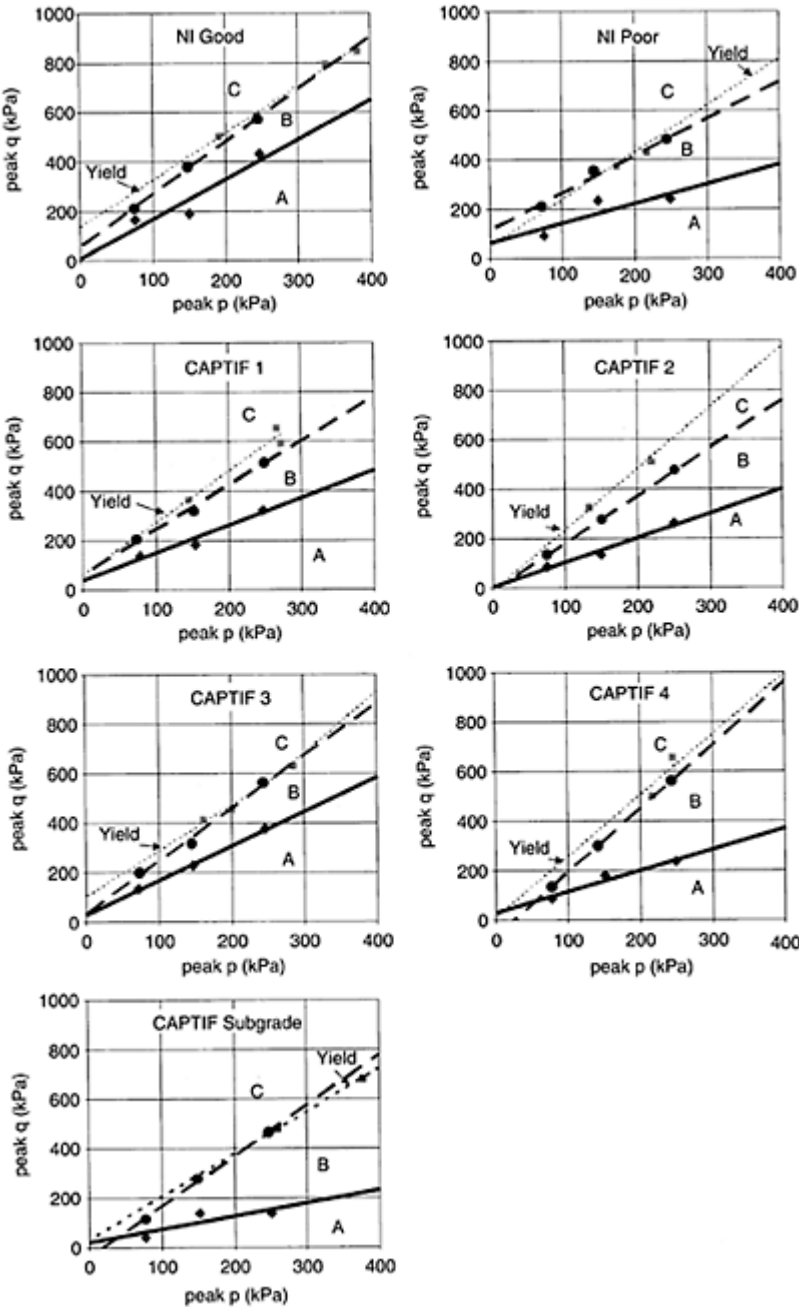


Figure 4. Shakedown range boundaries.

Figure 3 shows the cumulative permanent strain plot versus permanent strain rate for the test results shown in Figure 2.

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 3 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 2, 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 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 Figure 4 for all the materials tested. From the results it can be seen that the boundary between shakedown ranges A and B is significantly below the yield line. 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.

4 DESIGN

Two-dimensional axisymmetric finite element pavement models were developed for the Northern Ireland and CAPTIF pavement trials (Table 2). Loading was a 40 kN half axle dual wheel loads approximated using a single circular tyre print with a pressure of 750 kPa. The model used for the unbound granular and subgrade materials was a non-linear porous elastic model combined with a linear Drucker-Prager yield criteria. Constants for the non-linear elastic model were determined from the measured resilient strains in the repeated load triaxial tests. The Drucker-Prager yield criteria was defined from the intercept and slope of the shakedown Range A-B boundary line in p - q stress space (Figure 4) in order to allow a pseudo-static analysis. The Drucker Prager yield criteria simulates soil/granular material behaviour under load as either purely elastic or perfectly plastic.

Results of the pavement analysis were permanent strain contours showing regions in the pavement that had yielded as shown in Figure 5. The three pavements with asphalt greater than 90 mm (NI Good, NI Poor and CAPTIF 4) show most of the plastic deformation occurs on top of the UGM, for the other pavements with thin asphalt covers the maximum plastic deformation occurs at mid-depth of the UGM or the bottom UGM layer near the subgrade. Plastic strains at each grid point were exported into a spreadsheet to calculate the total permanent surface deformation. In general,

Table 2. Pavement cross-sections.

Layer	Asphalt	Granular thickness (mm)	Subgrade
Pavement cross-section:	1	36 289	1200
	2	31 282	
	3	25 282	

	4	90	200	
	5	100	650	750
	6	100	650	
		Material		
Pavement cross-section:	1	AC	CAPTIF 1	CAPTIF Subgrade
	2		CAPTIF 2	
	3		CAPTIF 3	
	4		CAPTIF 4	
	5		NI Good	Solid Rock
	6		NI Poor	

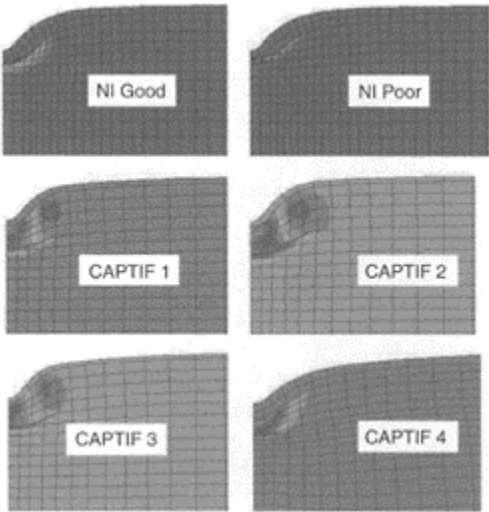


Figure 5. Regions of vertical plastic deformation for shakedown range A/B boundary analysis (NB: scales are not the same and the red circles in CAPTIF 1, 2 and 3 are tensile or upwards).

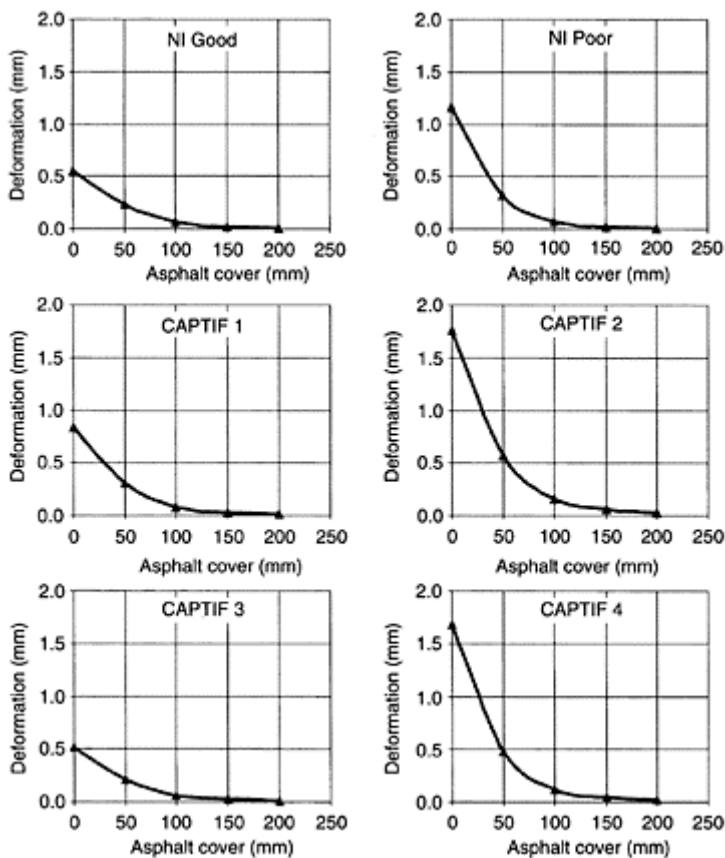


Figure 6. Surface deformation from plastic finite element analysis with linear shakedown range A/B boundary yield line for a range of asphalt cover thicknesses.

Table 3. Minimum asphalt cover required to ensure shakedown range A behaviour.

UGM (Table 1)	Min. AC cover (mm) for range A.
NI Good	138
NI Poor	142
CAPTIF 1	150
CAPTIF 2	200
CAPTIF 3	138

the total permanent vertical surface deformation was relatable to the number of points in the pavement where stable shakedown Range A behaviour was not occurring. Figure 6 shows the effects of increasing asphalt cover thickness on total permanent vertical surface deformation. It is concluded that 200 mm of asphalt cover is sufficient to ensure shakedown range A behaviour in the aggregate due the minimal amount of permanent vertical surface deformation (<0.03 mm). If it is considered that the maximum allowable permanent vertical surface deformation for shakedown range A is 0.03 mm then Table 3 details the asphalt cover requirements for the UGMs.

5 CONCLUSIONS

RLT permanent strain test results for two Northern Ireland and four New Zealand unbound granular materials UGMs 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 boundary between shakedown 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 between stable (acceptable) behaviour and unstable behaviour. This stress boundary was then used in ABAQUS Finite Element Package as a Drucker-Prager yield criteria for the UGM to predict whether or not shakedown/range A behaviour occurs for the field trial pavements and for a range of asphalt cover thicknesses. 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 as a yield criteria in a finite element model for pavement design.

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REFERENCES

- AUSTROADS, 1992. Pavement Design—A Guide to the Structural Design of Road Pavement, Austroads, Sydney, Australia.
- Dawson, A.R. and Wellner, F. 1999. Plastic behaviour of granular materials, Final Report ARC Project 933, University of Nottingham Reference PRG99014, April 1999.
- HMSO, 1994. Design manual for roads and bridges, Vol 7, HD 25/94, Part 2, Foundations.

- Little, Peter H., (1993) The design of unsurfaced roads using geosynthetics, Dept. of Civil Engineering, University of Nottingham.
- Pidwerbesky, B. Fundamental Behaviour of Unbound Granular Pavements Subjected to Various Loading Conditions and Accelerated Trafficking. PhD Thesis, University of Canterbury, Christchurch, New Zealand, 1996.
- Sharp, R. and Booker, J., Shakedown of pavements under moving surface loads, pp. 1–14, ASCE Journal of Transportation Engineering, No 1, 1984.
- Thom, N. and Brown, S., 1989, The mechanical properties of unbound aggregates from various sources. Proceedings of the Third International Symposium on Unbound Aggregates in Roads, UNBAR 3, Nottingham, United Kingdom, 11–13 April 1989.
- TRL, 1993. A guide to the structural design of bitumen-surfaced roads in tropical and sub-tropical countries, RN31, Draft 4th Edition.
- Wardle, L., 1980. Program CIRCLY, a computer program for the analysis of multiple complex circular loads on layered anisotropic media.
- Werkmeister, S. Dawson, A. Wellner, F, 2001. Permanent deformation behaviour of granular materials and the shakedown concept. Transportation Research Board, 80th Annual Meeting, Washington D.C. January 7–11, 2001.

A simplified method of prediction of permanent deformations of unbound pavement layers

A.El abd & P.Hornych

Laboratoire Central des Ponts et Chaussée (LCPC), Nantes, France

D.Breyse & A.Denis

Centre de Développement des Géosciences Appliquées, Université de Bordeaux 1, Talence, France

C.Chazallon

Laboratoire de Modélisation Mécanique des Matériaux et Structures de Génie Civil,

Université de Limoges, France

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ABSTRACT: This paper presents a simplified method for modelling of permanent deformations of unbound granular layers. This method is based on three steps: first, the material is characterised using repeated load triaxial tests, with different stress levels. Then, the test results are analysed using a model of prediction of permanent deformations as a function of maximum applied cyclic stresses and number of load cycles. Two different models can be selected at this stage: an empirical relationship and an elastoplastic model. Finally, a finite element analysis with the program CESAR-LCPC, is used to determine the stress distribution in the pavement. The stresses calculated at different points in the structure allow the permanent vertical strains at each point to be determined, by applying one of the selected permanent deformation models. Finally the permanent strains are integrated along the vertical direction to obtain the vertical displacements in the structure (i.e. rutting of the layer). An example of application of the method is presented.

1 INTRODUCTION

Most present pavement design methods are based on linear elastic calculations. Such methods give good results for rigid pavements, with bituminous or cement treated base and subbase layers. They are much less satisfactory for flexible, low traffic pavements,

with unbound granular bases. The response of such pavements is strongly non-linear, dependent on the level of load, and their main mode of distress is rutting, due to development of permanent deformations in the unbound granular layers and subgrade soil. For these pavements, more realistic modelling and design methods, taking into account the non-linear behaviour of their materials, need to be developed.

The objective of the work presented in this paper is to develop a modelling approach for low traffic pavements, allowing the determination of the permanent deformations of the unbound granular layers. This approach is based on the following principles:

- The resilient and permanent response of unbound granular materials (UGMs) is described using appropriate non linear models;
- The model of resilient behaviour is implemented in a finite element code and used to determine the stress distribution in the pavement structure;
- The stresses are used to calculate the permanent strains at different points, using the permanent deformation model;
- Finally, the strains are integrated along the vertical direction, to determine the vertical displacements (rut depths).

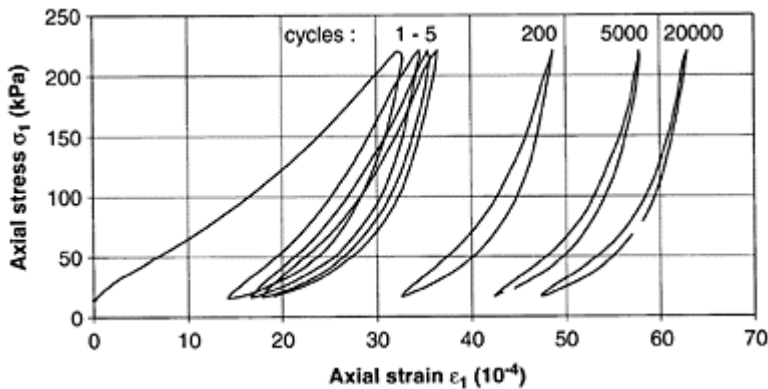


Figure 1. Example of stress-strain cycles obtained in a repeated load triaxial test on a UGM.

2 MODELLING OF PERMANENT DEFORMATIONS OF UNBOUND GRANULAR MATERIALS

2.1 *Cyclic behaviour of unbound granular materials*

This work is based on results from cyclic triaxial tests. Figure 1 presents a typical example of response of an unbound granular material in a cyclic triaxial test at low stress conditions. The response of the material is essentially elasto-plastic, and it can be observed that:

- After a rapid increase during the first load cycles, the permanent strains tend to stabilise (or increase at a very low rate), and the response of the material becomes essentially elastic.
- The elastic part of the response of the material is strongly non-linear (stress-dependent).

The approach used in this study to describe the behaviour of UGMs consists in modelling separately:

- The resilient behaviour, considered independent of the number of load cycles, and described using a non linear elastic model;
- The accumulation of permanent strains, which will be described successively using two different models, an empirical relationship and an elastoplastic model.

2.2 Permanent deformation behaviour

The most common procedure to study permanent deformations of UGMs using the repeated load triaxial apparatus consists in applying a large number of load cycles (10^5 and more), with one stress level (as on Figure 1).

To describe this evolution of permanent deformations of UGMs under cyclic triaxial loading, two main approaches are generally used. The first, frequently used by pavement engineers, consists of describing the evolution of permanent deformations under cyclic loading using empirical relationships, relating directly the permanent strains to the applied cyclic loading (number of load cycles N , amplitude of cyclic stresses). The second, developed in soil mechanics, is based on elasto-plastic modelling. In this research, the two approaches are successively used and compared.

It should be noted, however, that all these approaches based on cyclic triaxial tests do not take into account the rotation of principal stresses, observed under real traffic loads. Various experimental results indicate that this rotation has the effect of increasing the permanent deformations (Chan [1990], Hornych et al. [2000]).

2.3 Empirical permanent deformation models

Many empirical permanent deformation relationships based on triaxial tests can be found in the literature. Some describe only the evolution of permanent deformation with the number of load cycles N . Well known relationships of this type have been proposed by Barksdale [1972] and by Sweere [1990]:

$$\epsilon_1^p = a + b \log(N) \qquad \epsilon_1^p = a N^b \qquad (1,2)$$

These two relationships suppose that ϵ_1^p increases indefinitely with N . Hornych et al. [1993], who tested three French unbound granular materials, obtained good predictions with the following relationship, which assumes that ϵ_1^p stabilises for an infinite number of cycles:

$$\epsilon_1^p = A \left(1 - \left(\frac{N}{100} \right)^{-B} \right) \qquad (3)$$

The previous relationships cannot be applied to pavement structure calculations, because they do not take into account the applied stresses. Other researchers tried to relate the cyclic permanent deformations to the applied stresses. Gidel [2001] proposed a comprehensive relationship, taking into account the number of load cycles and the maximum applied cyclic stresses $p_{\max}$, $q_{\max}$:

$$\epsilon_1^p(N) = \epsilon_{10}^p \cdot \left[1 - \left(\frac{N}{N_0} \right)^{-B} \right] \cdot \left[\frac{L_{\max}}{p_a} \right]^n \cdot \frac{1}{\left(m + \frac{s}{p_{\max}} - \frac{q_{\max}}{p_{\max}} \right)} \quad (4)$$

$$L_{\max} = \sqrt{p_{\max}^2 + q_{\max}^2}, \quad p_a = 100 \text{ kPa},$$

with

ϵ_{10}^p , B , n , model parameters m , s : parameters of the failure line of the material, of equation $q = m \cdot p + s$

In this research, it was decided to test the empirical model of Gidel, which is written as the product of a function of the number of load cycles by a function of the maximum stresses, of the form $\epsilon_1^p(N) = f(N) \cdot g(p_{\max}, q_{\max})$. Comparisons with experimental results indicated that the function g proposed by Gidel gave satisfactory predictions, but that the function $f(N)$ could not be used, because the tests performed in this study did not show a complete stabilisation of permanent strains. It was therefore replaced by the function proposed by Sweere, $f(N) = AN^B$ which gave better results. This empirical relationship was finally adopted to describe the permanent deformations.

2.4 Elastoplastic models

Elastoplastic models separate the strains into elastic and plastic parts: $\underline{\epsilon} = \underline{\epsilon}^e + \underline{\epsilon}^p$ where $\underline{\epsilon}$ is the total strain, $\underline{\epsilon}^e$ is the elastic strain and $\underline{\epsilon}^p$ is the plastic strain. These models link the stress tensor to the strain tensor with the incremental equation, $d\sigma = \underline{H} d\underline{\epsilon}$, where $\underline{H}$ is a fourth order tensor. This equations system is solved by incremental calculations. The major advantage of this incremental approach is that it is possible to describe the response to any type of loading history.

The modelling of cyclic behaviour requires elaborate elasto-plastic models, which generate plastic strains during loading and unloading, like models with kinematic hardening. Models of this type have been developed in soil mechanics, often for earthquake applications and are well suited for low numbers of load cycles (less than 100 cycles). However, their application to pavement conditions, where the number of loads is much larger (typically 10^5 to 10^7), and where the permanent strain increments are very low (less than 10^{-6} per load cycle), is generally not possible without substantial modifications. Recently, some elastoplastic cyclic models adapted to pavement applications have been proposed by Bonaquist & Witczak [1997], Richer et al. [1999] and Chazallon [2000].

2.5 The elastoplastic model of Chazallon

The elastoplastic model adopted in this research is the model proposed by Chazallon [2000]. This model incorporates isotropic and kinematic hardening, and allows to simulate both monotonic response and cyclic response with large numbers of load cycles (10^5 to 10^6). The elastic behaviour is described by the non-linear elastic Boyce model [1980] as improved by Hornyh et al. [1998].

2.5.1 Modelling of plastic strains

The elastoplastic model developed by Chazallon is based on the yield function and plastic potential of the non-associated model of Hujeux [1985] in its simplest formulation.

In the (p, q) plane where p is the confining pressure, $p=(\sigma_1+2\sigma_3)/3$ and q the deviatoric stress, $q=\sigma_1-\sigma_3$, the yield function is given by:

$$f(\underline{\sigma}) = \sqrt{\frac{27}{2}} \frac{S_{II}(\underline{\sigma} - \underline{X})}{M I_I(\underline{\sigma} - \underline{X})} + r b \ln\left(\frac{I_I(\underline{\sigma} - \underline{X})}{3p_c}\right) \quad (5)$$

where $\underline{X}=(p_x, q_x)$ is a reference stress state; I_I is the trace operator, $I_I(\underline{\sigma} - \underline{X})=3(p-p_x)$ and S_{II} is the deviatoric stress operator, $S_{II}(\underline{\sigma} - \underline{X})=(2/3)^{0.5}(q-q_x)$. b is a parameter which controls the shape of the yield surface. M is the slope of the critical state line in the (p, q) plane and p_c is the critical pressure corresponding to the actual void ratio.

This equation is completed by the definition of a non-associated plastic potential g

$$g(\underline{\sigma}) = \sqrt{\frac{27}{2}} \frac{S_{II}(\underline{\sigma} - \underline{X})}{M I_I(\underline{\sigma} - \underline{X})} + b \ln\left(\frac{I_I(\underline{\sigma} - \underline{X})}{3p_c}\right) \quad (6)$$

When the applied stresses are inside the yield surface, $f(\underline{\sigma}) < 0$, the behaviour is elastic. When the stresses reach the yield surface $f(\underline{\sigma}) = 0$, the plastic strains increment is given by:

$$d\epsilon_p = d\lambda \frac{\partial g(\underline{\sigma})}{\partial \underline{\sigma}} \quad (7)$$

where $d\lambda$ is the plastic multiplier.

Hardening (isotropic and kinematic) is governed by three hardening variables:

- A hardening variable associated to the volumetric plastic strains ϵ_v^p :

$$p_c = p_{c0} \exp(\beta \epsilon_v^p) \quad (8)$$

where p_{c0} and β are parameters of the model.

- A hardening variable r associated to the deviatoric plastic strain ϵ_d^p . During the first loading, r is defined by:

$$r = r_m^{el} + \frac{\epsilon_d^p}{a_m + \epsilon_d^p} \quad (9)$$

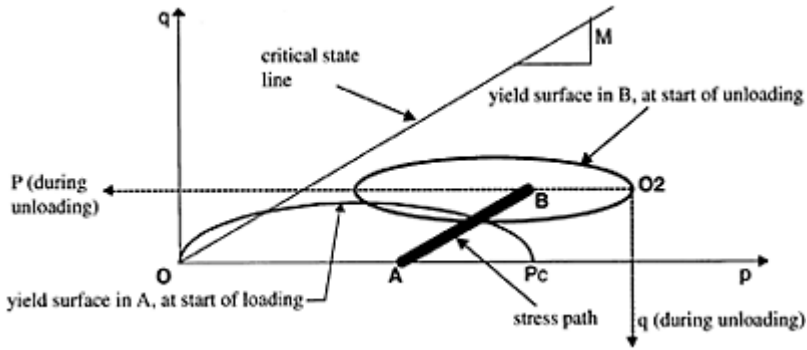


Figure 2. Evolution of the yield surface of the elastoplastic model of Chazallon.

where r_m^{el} represents the initial elastic domain and a_m controls the evolution of r . During subsequent unloading and reloading, it is modified to:

$$r = r_c^{el} + \frac{\epsilon_d^p}{a_c + \epsilon_d^p} \quad (10)$$

where r_c^{el} and a_c are two parameters associated with cyclic loading.

- A tensorial kinematic hardening variable $\underline{X}$, which defines the new position of the yield surface during unloading and reloading. During the first loading $\underline{X}$ is zero. Then, at the beginning of unloading, and during subsequent reloading, the position of the yield surface is moved according to the rule :

$$\text{For unloading : } \underline{X} = \underline{\sigma} + P_{uc} p_c \underline{I} \quad (11)$$

$$\text{For reloading: } \underline{X} = \underline{\sigma} + P_{lc} p_c \underline{I} \quad (12)$$

where : $\underline{I}$ is the identity tensor, and P_{uc} , P_{lc} and μ are model parameters. And the evolution of $\underline{X}$ is given by $d\underline{X} = \mu d\underline{\sigma}$.

Figure 2 shows the evolution of the yield surface in the (p, q) plane, for a linear loading between points A and B. During the first loading, the yield surface is centred on the p axis, with its origin in O. When unloading starts from B, the surface moves, and its new origin becomes O₂.

3 MODELLING OF TRIAXIAL TEST RESULTS

3.1 Test programme

In order to determine the parameters of the two models, a programme of repeated load triaxial tests was performed, on a 0/10 mm continuously graded granular material (crushed gneiss). The tests were performed using a cyclic triaxial apparatus for soils, with a cell for 76 mm diameter specimens. The material was tested at a water content of 6% and at a density of $2,1 \text{ g/cm}^3$, (modified Proctor optimum values).

The test programme included :

- Four monotonic triaxial shear tests, with confining pressures $\sigma_3=0, 10, 20$ and 50 kPa .
- One repeated load triaxial test to determine the resilient behaviour. This test included a cyclic conditioning (20 000 load cycles with cyclic stresses $p=100 \text{ kPa}$ and $q=200 \text{ kPa}$), and then a series of short loading (100 cycles each), following different stress paths.

Table 1. Stress paths applied in the permanent deformation tests.

Test	Load sequence	$\Delta p \text{ (KPa)}$	$\Delta q \text{ (KPa)}$	$\Delta q/\Delta p$
1	1	60	60	1
	2	100	100	
	3	150	150	
2	1	15	30	2
	2	30	60	
	3	50	100	
	4	75	150	
3	1	24	60	2.5

Table 2. Parameters of the empirical model obtained for the crushed gneiss.

$\epsilon_1^{p0}(10^{-4})$	B	n	m	s (kPa)
26.08	-0.123	0.122	2.30	63.65

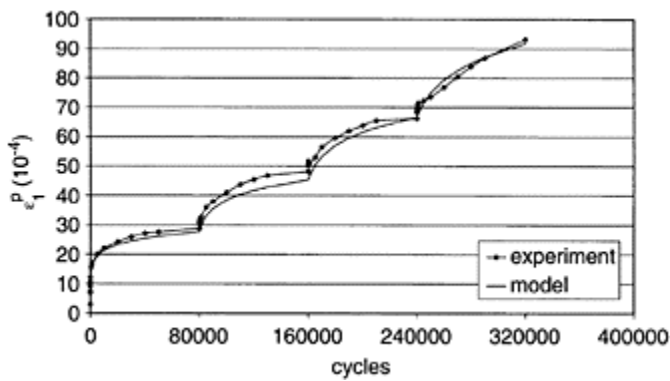


Figure 3. Predictions obtained with the empirical model, for $q/p=2$.

- Three permanent deformation tests, with both cyclic axial stress and confining pressure, performed using the procedure proposed by Gidel [2001]. It consists in applying, on the same specimen, several loading sequences with the same stress ratio q/p , but with increasing stress amplitudes, as defined in Table 1. Each loading sequence was performed during 80000 cycles.

3.2 Adjustment of the parameters of the empirical model

The empirical permanent deformation relationship is the product of a function of the number of load cycles $f(N)$ and a function of the applied stresses $g(p_{max}, q_{max})$. The two functions were fitted separately. First the parameters of the function $f(N)=A.N^B$ were determined, using the results obtained in the three permanent deformation tests for the first stress level. Then, all the parameters of the function g were determined by fitting the cumulated values of ϵ_p obtained in the permanent deformation tests, for the different stress levels, for a constant number of load cycles ($N=80\ 000$). The values obtained for the model parameters are summarised in Table 2. An example of prediction obtained for the triaxial test with $q/p=2$ is shown on Figure 3.

Table 3. Values of the parameters of the model of Chazallon obtained for the crushed gneiss.

Non-linear elastic parameters				Monotonic loading parameters					Cyclic loading parameters				
n	K_a Mpa	G_a Mpa	γ	C_0 kpa	M	β	am 10^{-3}	b	r_m^{el}	p_{uc} 10^{-3}	$p1c$ 10^{-3}	r_c^{el}	a_c 10^{-5}
0.125	6.47	25.78	0.625	14	2.46	400	1.33	0.188	0.012	3	1.7	0.01	2

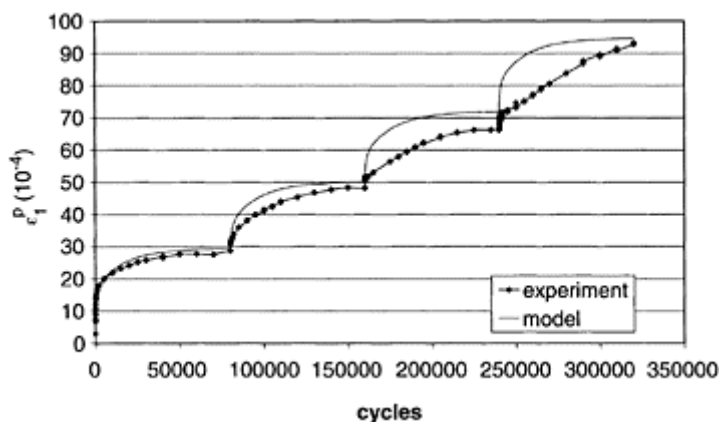


Figure 4. Simulation of the permanent deformation test at $q/p=2$ with the elastoplastic model.

3.3 Adjustment of the parameters of the elastoplastic model

The elastoplastic model includes 16 parameters. They can be divided into 4 non-linear elastic parameters, 6 plastic parameters describing monotonic loading and 4 plastic parameters describing cyclic loading. These experimental parameters are summarised in Table 3. Two additional parameters, p_{c0} and μ , have been arbitrarily fixed at $p_{c0}=10\text{MPa}$ and $\mu=0.5$, as recommended by Chazallon [2000]. These three groups of parameters are uncoupled, and can be determined separately.

The elastic parameters (K_a , G_a , n , γ) were determined using the resilient behaviour test. The monotonic loading parameters were determined using the monotonic triaxial tests at different initial confining pressures. C_0 is the cohesion. M is the slope of the critical state line in (p, q) space, separating contractant and dilatant behaviour. In the Hujeux model, the critical pressure p_{c0} and β require, for their determination, isotropic compression tests up to at least 20 MPa, which could not be performed with the triaxial equipment used. The parameters a_m , b , and β were determined using a specific fitting programme which finds their optimum values by comparing the calculated σ_1 - ε_1 monotonic stress strain curves with the experimental values.

The cyclic parameters (P_{uc} , P_{lc} , r_c^{cl} , a_c) were then adjusted manually, using the results of the permanent deformation tests. This is possible because each parameter plays a different roles: the cyclic plastic parameter r_c^{cl} gives the size of the yield surface at the beginning of each half-cycle; the parameters P_{uc} and P_{lc} determine the final level of ε_1^p at stabilisation; a_c influences the number of cycles necessary to reach stabilisation.

The elastoplastic model has been implemented in a program which calculates incrementally its response to a given stress path (Piganeau, [2003]). Figure 4 shows the predictions obtained for the permanent deformation test with $q/p=2$

4 PREDICTION OF RUT DEPTHS IN PAVEMENT STRUCTURES

4.1 *Principle of the method of prediction of the rut depth*

In this paper, a first simplified computation method is presented, for the calculation of the rut depth in a pavement structure. For the moment, it is assumed that the loads applied on the pavement are all identical (same value of the load, same position), and only the permanent vertical deformations of the unbound granular layers are calculated (no model is available to calculate the permanent deformations in the bituminous layers or in the subgrade soil). The rut depth is calculated in two stages:

- First, a finite element calculation is performed, in 3D, to determine the resilient stress field induced in the pavement by the chosen load. This analysis is performed with the finite element program CESAR-LCPC, with the module CVCR, developed for modelling of pavements under moving wheel loads (Heck [1998], [2001]).
- Then the calculation of the permanent strains is performed in 2D, in the plane (0, y, z) perpendicular to the axis of displacement of the load (axis Ox). The CVCR results allow to determine the resilient stress paths in different points in this plane, due to the passage of the load. Then, these stress paths are used to calculate the permanent axial strains after N load cycles, using one of the two permanent deformation models. Finally, the strains are integrated in the vertical direction, to obtain the vertical displacements at the surface of the pavement.

The application of this method is detailed in an example below.

4.2 *Example of calculation of the rut depth in a pavement structure*

4.2.1 *Pavement structure and assumptions*

This part presents the application of the method to the calculation of the rut depth in a low traffic pavement structure, which has been tested on the LCPC accelerated pavement testing facility. This pavement structure consists of:

- An 8 cm thick bituminous wearing course;
- A 40 cm thick granular base and subbase;
- A clayey sand subgrade (thickness 2.30 metres), resting on a rigid concrete slab.

The granular material is the crushed gneiss tested in Section 3 of the paper. The bituminous concrete and the subgrade soil are supposed linear elastic, with the following characteristics:

- Bituminous concrete: $E=5400$ MPa, $\nu=0.25$; Soil: $E=100$ MPa, $\nu=0.35$.

4.2.2 Modelling of the resilient behaviour of the pavement

The module CVCR of CESAR-LCPC includes non-linear elastic models for UGMs, (the K-theta model and the Boyce model), and a visco-elastic model for bituminous materials, the Huet-Sayegh model (Heck et al. [1998]). The calculations are performed in 3D, under static or moving loads.

Here, a static load consisting of 2 twinned wheels, with a total load of 65 kN was considered. The materials were considered linear elastic, except for the granular base, described with the anisotropic Boyce model, with the parameter values determined in the laboratory triaxial tests (see Table 3).

4.2.3 Determination of the stress paths in the granular layer

The next step consists in determining the stress paths in the granular layer from the CVCR results. These stress paths (and the permanent strains) are calculated in the plane (0,y,z) perpendicular to the direction of movement of the load (Ox). In our case (elastic calculations, with a static load), the stress path at a point M (y,z) is given by the variation of the resilient stresses (Ox) along the x-axis.

Figure 5 shows examples of stress paths obtained with this approach, at different depths z, under the centre of one wheel (position $y=0.185$ m). In the (p,q) stress space, these stress paths

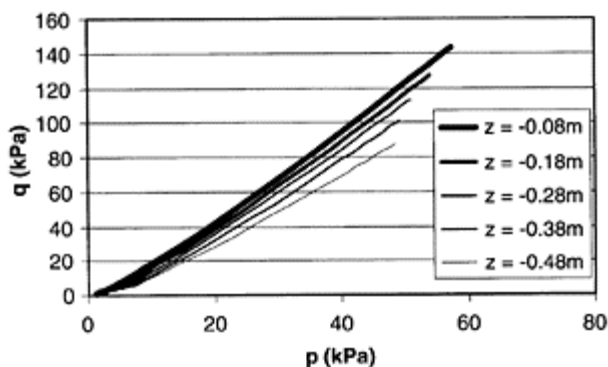


Figure 5. Stress paths in the granular layer, under the centre of one wheel, at different depths.

are practically linear, with values of slopes q/p between approximately 2 and 2.5. This is also true at other points in the granular layer and confirms that the stress paths in the pavement are very close to those applied in the permanent deformation tests (linear stress paths, with slopes $q/p=1$, 2 and 2.5).

4.2.4 Rut depth calculation procedure

The rut depths are calculated as follows :

- The granular layer is divided into j horizontal sub-layers of thickness h_j .
- Several points M_{ij} (y_i, z_j) are defined at mid-height of each sublayer, and at different lateral positions y_i . The stress paths $\underline{\sigma}(x, y_i, z_j)$ at these points are then determined from the CVCR calculation.
- With the empirical model, the permanent vertical strain at each point M_{ij} , after N cycles, is calculated directly by equation (4), using the maximum values of p and q of the stress path.
- With the elastoplastic model, the permanent strains at each point M_{ij} are calculated incrementally, cycle by cycle, under the application of the complete stress path using again the program developed by Piganeau.
- Finally, the rut depth $d(y_i)$ at a given lateral position y_i is obtained by cumulating the vertical strains in each sublayer along z :

$$d(y_i) = \sum_{j=1}^n \epsilon_{ij}^p h_j \quad (13)$$

4.2.5 Prediction of the rut depths with the two permanent deformation models

In our example, the rut depth is calculated under the centre of one wheel, where the stresses in the granular layer are maximum, after 100 000 load cycles. The same calculation could be performed at other lateral positions in the pavement (under the centre of the half-axle, for example).

With the empirical model, the layer was divided into 8 sublayers of 5 cm thickness each. The cumulated vertical displacement in the granular layer is equal to 3.42 mm.

With the elastoplastic model, considering that the stresses decrease relatively slowly with depth, it was decided to use only 5 sublayers, to reduce the number of permanent strain calculations (at each point, it is necessary to calculate incrementally the response to 100 000 cycles, which is time consuming). In this manner, the total vertical displacement is 2.03 mm.

In order to compare the results obtained with the two models, the total vertical displacements in the granular layer have been calculated with the previous method after 20 000, 40 000, 60 000,

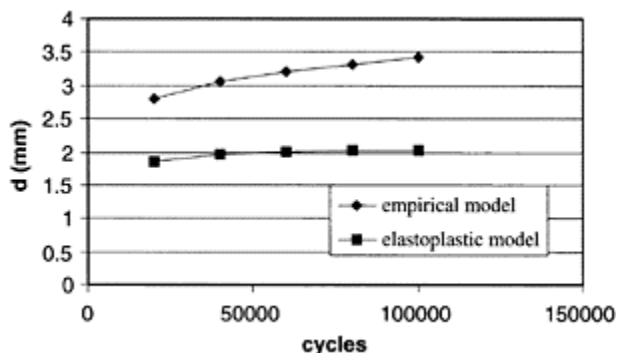


Figure 6. Comparison of rut depths calculated with the empirical and elastoplastic models

80000 and 100000 load applications. Figure 6 illustrates the evolution of the vertical displacements with the number of load cycles.

The levels of rut depths obtained in the calculations (2 and 3.4 mm of rutting after 100 000 cycles) appear reasonable for this type of pavement. However, the evolution obtained with the two models is different: the elastoplastic model leads to lower rut depths, and indicates a greater stabilisation of permanent strains after 100 000 cycles than the empirical model. For the moment, it is not possible to compare these predictions with real measurements on the experimental pavement, because the full scale experiment is under way. However, the difference between the predictions with the two models seems to come from the adjustment of the triaxial test results. Clearly, the empirical model predicts better the triaxial behaviour at the highest stress levels, where no clear stabilisation of permanent strains is observed, and the same trend can be seen in the rut depth predictions.

These results are presented only to illustrate the capability of the permanent deformation models, for predicting rutting in granular pavement layers. Clearly, further work will be necessary:

- To optimise the laboratory test procedures used to determine the model parameters. In particular, the number of load cycles applied at each stress level has an influence on the adjustment of the models and the prediction of the behaviour at large numbers of load cycles.
- To improve the method of adjustment of the elastoplastic model, which is complex with the high number of parameters involved.
- To evaluate the proposed method for different types of pavement structures, with different thicknesses of bituminous materials, and thus different stress levels in the granular layers.

5 CONCLUSION

A simplified method has been proposed for modelling the permanent axial strains in granular pavement layers, in order to predict the rutting of a low traffic pavement. Two permanent deformation models have been tested: an empirical model and an elastoplastic model with both isotropic and kinematic hardening. At the present time, this study constitutes a preliminary work allowing to illustrate the capability of the permanent deformation models used.

With the realisation of the full scale experiment, it is planned to perform a more complete triaxial test programme, to improve the method of adjustment of the model parameters, and finally to validate in detail the predictions obtained with the two models, by comparison with the site results.

A second objective will be to generalise the prediction method, to take into account variable loading conditions during the life of the pavement (different load values, lateral distribution of loads). With the elastoplastic model, it will also be necessary to simplify the actual incremental calculation procedure, which is too time consuming for large numbers of load cycles.

REFERENCES

- Barksdale, R.D. 1972. *Laboratory evaluation of rutting in base course materials*. Proc. 3rd Int. Conf. On the Structure Design of Asphalt Pavements, London, pp 167–174.
- Bonaquist, R.F., Witczak, M.W. 1997. *A comprehensive constitutive model for granular materials inflexible pavement structures*. Proc. 8th Int. Conf. On Asphalt Pavements, Seattle, Washington, vol. 1, pp 783–802.
- Boyce, H.R. 1980. *A non-linear model for the elastic behaviour of granular materials under repeated loading*. Proc. Int. Symposium on Soils under Cyclic and Transient loading, pp 285–294.
- Chan, F.W. K. 1990. *Permanent deformation resistance of granular layers in pavements*. PhD thesis, Dept. Of Civil Engineering, University of Nottingham, England.
- Chazallon, C. 2000. *An elastoplastic model for unbound granular materials for roads*. UNBAR 5, Balkema, Nottingham, England, pp 254–260.
- Gidel, G., Horny, P., Chauvin, J.J., Breyse, D., Denis, A. 2001a. *Nouvelle approche pour l'étude des déformations permanentes des graves non traitées à l'appareil triaxial à chargements répétés*. Bulletin de Liaison des Laboratoires des Ponts et Chaussées, n°233, pp 5–21.
- Gidel, G. 2001b. *Comportement et valorisation des graves non traitées calcaires utilisées pour les assises de chaussées souples*. PhD Thesis, University of Bordeaux 1, France.
- Heck, J.V., Piau, J.M., Gramsamer, J.C., Kerzreho, J.P., Odeon, H. 1998. *Thermo-visco-elastic modelling of pavements behaviour and comparison with experimental data from LCPC test track*. Proc. 5th Int. Conf. on Bearing Capacity of Roads and Airfields, Trondheim, Norway.
- Heck, J.V. 2001. *Modélisation des déformations réversibles et permanentes des enrobés bitumineux—Application à l'ornièrage des chaussées*. PhD Thesis, University of Nantes, France.
- Hicher, P.Y., Daouadji, A., Fedghouche, D. 1999. *Elastoplastic modelling of the cyclic behaviour of granular materials*. Proc. Int. Workshop on Modelling and Advanced Testing for Unbound Granular Materials, Lisbon, Portugal, pp 161–168.

- Hornych, P., Corte, J.F., Paute, J.L. 1993. *Etude des deformations permanentes sous chargements répétés de trois graves non traitées*. Bulletin de Liaison des LPC, n°184, pp 45–55.
- Hornych, P., Kazaï, A., Piau, J.M. 1998. *Study of the resilient behaviour of unbound granular materials*, Proc. 5th Int. Conf. on Bearing Capacity of Roads and Airfields, Trondheim, Norway.
- Hornych P., Kazaï A., Quibel A. 2000. *Modelling a full scale experiment on two flexible pavements with unbound granular bases*. UNBAR5, Int. Symposium on Unbound Aggregates in Roads, Nottingham, pp 359–367.
- Hujeux, J.C. 1985. *Une loi de comportement pour le chargement cyclique des sols*. In Génie parasismique, Presse des Ponts et Chaussées, Paris, pp 316–331.
- Piganeau, N, Hornych P., Tamagny, P. 2003. *Analyse des deformations irréversibles des plate-formes ferroviaires sous sollicitations cycliques*. Rapport de stage.
- Sweere, G.T.H. 1990. *Unbound granular base for roads*. PhD Thesis, University of Delft, Netherlands.

Simplified model based on the shakedown theory for flexible pavements

Taha Habiballah¹, Cyrille Chazallon¹ & Pierre Hornych²

¹ *Laboratoire de Mécanique et Modélisation des Matériaux et Structures
du Génie Civil,*

Université de Limoges, France

² *Laboratoire Central des Ponts et Chaussées, Bouguennais cedex,
France*

ABSTRACT: Rutting, due to permanent deformations of unbound granular layers, is the principal mode of degradation of flexible pavements (bituminous layer thickness less than 12 cm). The large number of load cycles necessary to describe the long-term behaviour of pavements is the main difficulty when plasticity based incremental formulations are used. This difficulty can be overcome using the shakedown theory. The elastoplastic model presented in this paper is based on the simplified method of Zarka. This method is usually used to describe the cyclic behaviour of materials with kinematic hardening like steels. This method has been adapted for unbound granular materials by introducing a yield surface taking into account the influence of the mean pressure on the mechanical behaviour of the granular media. The model is validated by comparing its predictions with results of repeated load triaxial test carried out on Missillac sand. Finally, the results of finite elements modelling of the long-term behaviour of a flexible pavement with the simplified model are presented.

1 INTRODUCTION

Flexible pavement structures are multi-layer systems consisting of unbound granular road base layers, carrying a thin bituminous wearing course. These pavements have a low structural capacity. Thus, they support only low traffics. Their principal mode of distress is rutting due to development of plastic deformations in the unbound granular layers, under the repeated loading due to road traffic.

The design of flexible pavements consists in defining the thickness of pavement layers necessary to avoid excessive deformations of the subgrade. Two design criteria are taken into consideration. The first criterion related to asphalt concrete fatigue and based on experimental fatigue curves, limits the tensile strains at the bottom of the bituminous wearing course. The second criterion is related to rutting, and is defined as a limit vertical elastic strain at the top of the subgrade, function of the number of load cycles. No criterion in connection with the plastic strains is taken into account.

The large number of load cycles necessary to describe the long-term behaviour of pavements, is the main difficulty when plasticity based incremental formulations are used (Bonaquist et al. 1997, Richer et al. 1999, Chazallon 2000).

The study of the deformations of the structure requires the knowledge of the stabilised stress and strain cycles. The difficulties caused by the large number of load cycles can be overcome using the shakedown theory. The elastoplastic model presented in this paper uses the simplified method of Zarka (Zarka & Casier 1979) as a starting point. This method is usually used to describe the cyclic behaviour of materials with kinematic hardening like steels. Zarka introduces a field of structural variables to define the elastic and plastic shakedown concept. Then he calculates the inelastic fields in the structure using an elastic analysis. This method has been modified to describe the behaviour of granular materials submitted to large numbers of load cycles. The Drucker-Prager plasticity criterion is used with a strictly positive linear kinematic hardening, and Von Mises non associated plastic potential. The model is validated by comparing its predictions with results of repeated load triaxial tests carried out on Missillac sand (Hornych 2003). Finally, the model is applied to finite element modelling of a flexible pavement.

2 SIMPLIFIED ELASTOPLASTIC MODEL PRINCIPLES

Let us consider a body with a volume V and a boundary δV . It is submitted to body forces $X^d(t)$, initial strains $\varepsilon_{ij}^I(x)$, surface tractions $F_i^d(x,t)$ applied on the contour $\delta_{F_i}V$ of δV and surface displacements $U_j^d(x,t)$ applied on the contour $\delta_{U_j}V$ of δV .

The general mechanical solution is:

$$\begin{cases} \varepsilon_{ij}(x,t) = M_{ijkl}\sigma_{kl}(x,t) + \varepsilon_{ij}^p(x,t) + \varepsilon_{ij}^I(x) \\ \sigma_{ij}(x,t) \end{cases} \quad (1)$$

where $\varepsilon_{ij}(x,t)$, $\varepsilon_{ij}^p(x,t)$ are respectively the actual and plastic strain fields. $\sigma_{ij}(x,t)$ is the actual stress field and M_{ijkl} is the elasticity compliance matrix.

This mechanical problem is split into inelastic and elastic problems, the actual stress field expression becomes:

$$\sigma_{ij}(x,t) = \sigma_{ij}^{el}(x,t) + \rho_{ij}(x,t) \quad (2)$$

where $\rho_{ij}(x,t)$ is the residual stress field. $\sigma_{ij}^{el}(x,t)$ is the response of the body with a purely elastic behaviour assumption. Equation 2 is rewritten in the deviatoric plane:

$$S_{ij}(x, t) = S_{ij}^{el}(x, t) + dev\rho_{ij}(x, t) \quad (3)$$

$S_{ij}(x, t)$, $S_{ij}^{el}(x, t)$ and $dev\rho_{ij}(x, t)$ are respectively deviatoric parts of the actual, elastic and residual stress field. The expression of the elastic problem is:

$$\begin{cases} \varepsilon_{ij}^{el}(x, t) = M_{ijkl}\sigma_{kl}^{el}(x, t) + \varepsilon_{ij}^l(x) \\ \sigma_{ij}^{el}(x, t) \end{cases} \quad (4)$$

$\varepsilon_{ij}^{el}(x, t)$ is kinematically admissible with $U_{ij}^d(x, t)\delta U_j V$ on and $\sigma_{ij}^{el}(x, t)$ is statically admissible $F_i^d(x, t)$ on $X_{ij}^d(x, t)$ and $\delta F_i V$ in V . The calculation of the elastic fields $U_{ij}^{el}(x, t)$ and $\varepsilon_{ij}^{el}(x, t)$ requires the previous boundary conditions, the elasticity matrix M_{ijkl} and the initial strains $\varepsilon_{ij}^l(x)$. This calculation is performed with an elastic analysis.

When a radial cyclic loading is performed, the elastic response is expressed at any time t and at any body point x by:

$$\sigma_{ij}^{el}(x, t) = (1 - \Lambda(t))\sigma_{ij}^{el\ min}(x) + \Lambda(t)\sigma_{ij}^{el\ max}(x) \quad (5)$$

$\Lambda(t)$ is a monotonic periodic function varying between 0 and 1 with time. $\sigma_{ij}^{el\ min}(x, t)$ and $\sigma_{ij}^{el\ max}(x, t)$ are respectively the minimum and maximum elastic stresses. The expression of the cyclic elastic response in the deviatoric plane is:

$$\begin{aligned} S_{ij}^{el}(x, t) &= S_{ij}^{el\ min}(x, t) + \Lambda(t)\Delta S_{ij}^{el}(x, t) \\ \text{with } \Delta S_{ij}^{el}(x, t) &= S_{ij}^{el\ max}(x, t) - S_{ij}^{el\ min}(x, t) \end{aligned} \quad (6)$$

$S_{ij}^{el\ max}(x, t)$ and $S_{ij}^{el\ min}(x, t)$ are respectively the deviatoric parts of $\sigma_{ij}^{el\ max}(x, t)$ and $\sigma_{ij}^{el\ min}(x, t)$.

The inelastic problem is obtained by difference between the general and elastic problems:

$$\begin{cases} \varepsilon_{ij}^{ine}(x, t) = M_{ijkl}\rho_{kl}(x, t) + \varepsilon_{ij}^p(x, t) \\ \rho_{ij}(x, t) \end{cases} \quad (7)$$

The inelastic analysis is applied to standard linear kinematic hardening materials. When plastic flow occurs, the translation of the yield surface is governed by the kinematic hardening variable $y_{ij}(x, t)$, its expression is:

$$y_{ij}(x, t) = \frac{2H}{3}\varepsilon_{ij}^p(x, t) \quad (8)$$

where H is the hardening modulus.

The inelastic problem is rewritten as:

$$\begin{cases} \varepsilon_{ij}^{ine}(x,t) = M_{ijk} \rho_k(x,t) + \frac{3}{2H} \gamma_{ij}(x,t) \\ \rho_{ij}(x,t) \end{cases} \quad (9)$$

$\varepsilon_{ij}^{ine}(x,t)$ is kinematically admissible with 0 on $\delta U_j V$ and $\rho_{ij}(x,t)$ is statically admissible with 0 on $\delta F_i V$ and with 0 in V . The inelastic fields $U_{ij}^{ine}(x,t)$ and $\varepsilon_{ij}^{ine}(x,t)$ are calculated with an elastic analysis using null stress boundary conditions, the initial strains $\underline{2H} \gamma_{ij}(x,t)$ and the elasticity compliance matrix M_{ijkl} .

We consider the field of structural variables defined by:

$$Y_{ij}(x,t) = \gamma_{ij}(x,t) - \text{dev} \rho_{ij}(x,t) \quad (10)$$

Using Equations 9 and 10, the expression of the inelastic strain field becomes:

$$\varepsilon_{ij}^{ine}(x,t) = M'_{ijk} \rho_k(x,t) + \frac{3}{2H} [Y_{ij}(x,t)] \quad (11)$$

If the field of structural variables is known, an elastic analysis gives the fields $U_{ij}^{ine}(x,t)$ and $\varepsilon_{ij}^{ine}(x,t)$ using null stress boundary conditions and the modified elasticity compliance matrix. The residual stress field is then deduced using Equation 11:

$$\rho_{ij}(x,t) = M'_{ijk}{}^{-1} [\varepsilon_{ij}^{ine}(x,t) - \frac{3}{2H} Y_{ij}(x,t)] \quad (12)$$

We can write schematically:

$$\rho_{ij}(x,t) = \bar{Z}_{ijk} [Y_{ij}(x,t)] \quad (13)$$

where $\bar{Z}_{ijk}$ is a general linear operator.

The non-associated Von Mises flow rule is considered and the Drucker-Prager yield surface is chosen:

$$f = \sqrt{\frac{1}{2} (S_y - y_y)^T : (S_y - y_y)} - k - \alpha I_1(\sigma_y) \quad (14)$$

with k and α : Drucker-Prager parameters. The expression of the yield surface in the plane of the structural variables $f(S_y^I - Y_{ij}) \leq 0$.

Thus, the plasticity convex is centred on $S_y^I(x,t)$. The position and radius of the plasticity convex are known at any time. On the one hand, the convex undergoes translations with loading, carrying onto its boundary the active plastic mechanisms of structural variables. On the other hand, it is possible to prove that the plasticity convex contains all the structural variables (Habiballah 2003). The elastic and plastic shakedown definitions according to Zarka are based on those key points.

Depending on the amplitude of $\Delta S_{ij}^{el}(x,t)$, if the extreme positions of the convex centred in $S_{ij \min}^{el}(x,t)$ and $S_{ij \max}^{el}(x,t)$ have a common part in each point of the structure, elastic shakedown occurs when t becomes large. We have plastic shakedown when no common part is found.

For plastic shakedown, the structural variables of the stabilised behaviour are determined according to geometrical conditions. It is the distance between the extreme positions of the plasticity convex for each active plastic mechanism (Zarka & Casier 1979, Habiballah 2003):

$$\Delta Y_{ij} = Y_{ij \max} - Y_{ij \min} = \Delta S_{ij}^{el} \left(1 - \frac{R_1 + R_2}{f(\Delta S_{ij}^{el})}\right)$$

where R_1 and R_2 are radii of the convex centred respectively in $S_{ij \min}^{el}(x,t)$ and $S_{ij \max}^{el}(x,t)$.

The structural variables evolve between two extreme positions with the translations of the plasticity convex. According to Equation 13, the residual stress varies periodically also. Consequently, the plastic deformation varies according to a closed cycle. The initial simplified method proposed by Zarka does not take into consideration the influence of the mean stress on granular material behaviour. Thus, the Drucker-Prager yield surface takes into account the influence of the mean stress and the theoretical formulations of Zarka are kept.

3 MODEL PARAMETERS IDENTIFICATION

The simplified model requires the elasticity parameters E and ν ; Drucker-Prager parameters ψ (the elasticity cone aperture) and p^* (the elasticity cone node position on the isotropic axis), and the hardening modulus H . The identification of the elasticity and plasticity parameters requires repeated load triaxial test results, performed with the same water content and the same compaction degree. The tests were performed on Missillac sand at LCPC in Nantes.

3.1 Elasticity parameters

The elasticity parameters are determined using the results of repeated load triaxial tests (Hornych 2003), used to study the resilient behaviour. The $K-\theta$ model (Hicks & Monismith 1972) is chosen for the adjustment of the test results.

$$E = K_1 \left(\frac{p}{p_a} \right)^{K_2}$$

where p is the mean stress and p_a is a reference stress equal to 100 KPa. K_1 and K_2 are model parameters: $K_1=92$ MPa, $K_2=0.04$, $\nu=0.2$ (Hornych 2003).

3.2 Plasticity parameters

The determination of the Drucker-Prager parameters requires at least three monotonic triaxial tests, p^* is the intersection of the failure line with the isotropic axis. ψ is determined in order to obtain a reduced initial elastic domain. Indeed, before the plastic flow, the elastic strain must be about 10^{-5} $\psi = 15^\circ$ for low stress ratios q/p . The following parameters have been obtained: $p^*=15$ KPa,

The determination of the hardening modulus requires an adjustment on repeated load triaxial tests used to study the plastic behaviour, performed with different stress ratios q/p ($q/p=1$, $q/p=1.5$, $q/p=2$, $q/p=3$) and different stress levels. Each loading stage includes 10,000 cycles (Hornych 1993).

The identification of the kinematic hardening modulus is done in three steps:

- First, the stabilised vertical plastic strains are determined starting from the repeated load triaxial test results for all the loading stages and all the stress ratios q/p with the Hornych model (Hornych 1993). Indeed, the simplified model gives stabilised plastic strains which must be used for the determination of the kinematic hardening modulus. The repeated load triaxial test results and Hornych's model calibration are represented on Figure 1.
- In a second step, for each loading stage, the stabilised plastic strains are used to determine the kinematic hardening moduli of the simplified model, which has been implemented in the finite elements code Cast3M. The calculations are carried out using the Drucker Prager plasticity parameters $p^*=15$ kPa, $\psi=15^\circ$ and the elasticity parameters determined for the maximum value of the mean stress for each loading stage, $\nu=0.2$, $E=92 (p_{max}/p_a)^{0.04}$.
- Finally, we obtain the law of evolution of the kinematic hardening modulus. It depends on the mean stress and the stress ratios q/p .

The calculated hardening moduli divided by the stress path length are represented against the normalized pressure in a bi-logarithmic diagram (Figure 2).

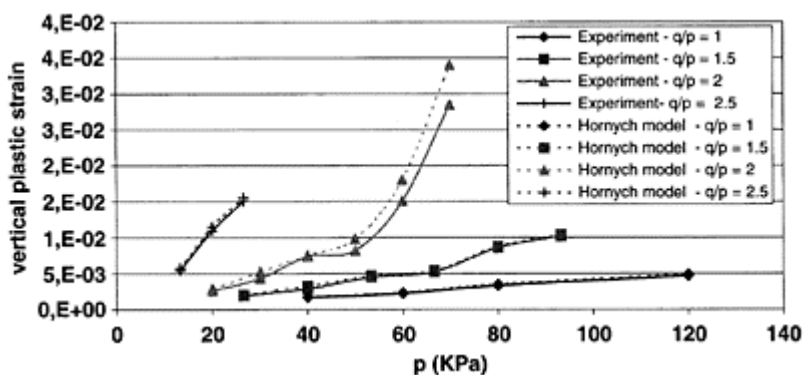


Figure 1. Experiments and Hornych model calibrations.

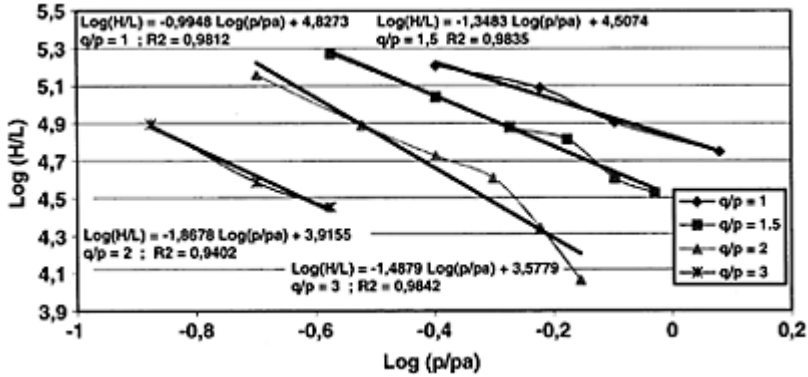


Figure 2. Hardening modulus evolution with a Log-Log approach.

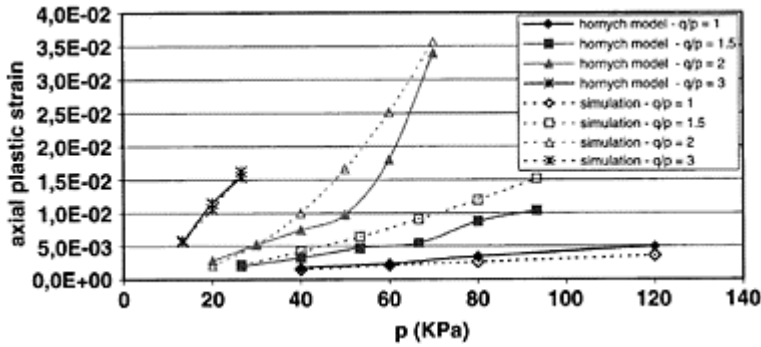


Figure 3. Comparison of the calculated vertical plastic strains with the experiments.

In this plane, we assume that the evolution of the hardening modulus divided by the stress path length against the normalized pressure is linear (Habiballah 2003). Thus we can write:

$$\text{Log}\left(\frac{H}{L}\right) = a \cdot \text{Log}\left(\frac{p}{p_a}\right) + b \quad (15)$$

where a and b are material parameters.

From the Equation 14, the hardening modulus is related to loading parameters:

$$H = 10^{\left[(a+1) \cdot \text{Log}\left(\frac{p}{p_a}\right) + \text{Log}\left(1 + \left(\frac{q}{p}\right)^b\right) \right]} \cdot p_a \quad (16)$$

From Figure 2, the a and b parameters are determined using a linear regression. Then, an evolution of these parameters with the q/p ratio is obtained (Habiballah 2003).

According to the mean stress p and the evolutions of a and b with the q/p ratio, the hardening modulus is calculated for each loading stage. The inelastic fields are then calculated and inelastic displacement and plastic strain fields are deduced. The simulation results are compared with experimental values in a strain-mean stress plane (Figure 3).

Globally, the model gives accurate results. For the stress paths $q/p=1$ and $q/p=3$, simulations and Hornych model curves are superimposed. The numerical results are rather close to Hornych model curves for the stress path $q/p=1.5$ and $q/p=2$. The authors underline that the present model has been developed for finite elements structural analysis contrary to the Hornych model.

4 FINITE ELEMENTS STRUCTURAL ANALYSIS

The pavement structure which has been modelled is a three-layer system. We consider at the bottom of this system a 551 cm thick subgrade layer made of the Missillac sand, carrying a 45 cm thick granular layer of Poulmarch (0/10 mm) unbound granular material. This last is covered by a 4 cm thick bituminous concrete layer. Stresses and displacements due to the surface loading decrease with depth. Consequently, the subgrade layer thickness determination is based on sensitivity studies. The goal is to choose a subgrade thickness so that negligible perturbations are obtained far from the loading zone.

Cyclic plate loading is considered, the maximum load is specified as a uniform pressure of 676 kPa acting over a radius of 17.5 cm. For numerical reasons, 10 Pa is taken for the minimal load.

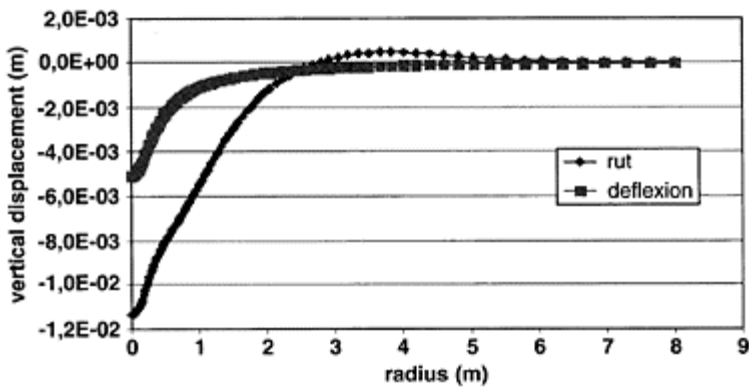


Figure 4. Inelastic and elastic displacement comparison.

The calculations are performed with Cast3M finite elements code in 2D and axisymmetric conditions.

Along the symmetry axis, nodes are fixed only in the horizontal direction. The same condition is assumed for the shoulder edge nodes. The nodes at the bottom of the subgrade are constrained horizontally and vertically.

Linear elasticity is assumed for the bituminous concrete ($E=5,400$ MPa; $\nu=0.25$). The constitutive models of the base and subgrade layers are the simplified elastoplastic model. Repeated load triaxial tests were performed on the Poulmarch unbound granular material by Hornyh (2003) to determine its resilient and permanent deformations. More details about the determination of the parameters of the Poulmarch unbound granular material are presented in Habiballah's thesis (Habiballah 2003).

A non linear elastic analysis is performed with the $K-\theta$ model to determine the elastic stress fields with the following parameters:

The parameters determined in section 3.1 are used for the Missillac sand.

Another parameter set has been considered for the Poulmarch material (Hornyh 2003).

Then, the shakedown state (elastic or plastic shakedown) is determined at each point of the structure using the Drucker-Prager parameters and the deviatoric elastic stress fields and $S_{ij}^{el} \max(x, t)$. Depending on the shakedown state, the field of structural variables is calculated locally. The mean stress and the q/p ratio fields are deduced from the calculated elastic stress field, consequently, the field of hardening moduli is obtained. Once the inelastic fields ensuring the equilibrium of the structure are calculated, the inelastic displacements and plastic strain fields are deduced. The rut depth is given by the inelastic displacements at the surface of the pavement. Rut depths and elastic vertical displacements (deflections) are compared on Figure 4.

One can see that the model is able to simulate flexible pavement rutting. The influence of the thickness of the bituminous wearing course on the rut depth has been studied by Habiballah (2003). It was found that the maximum rut depth is multiplied by 7 when the thickness of the bituminous layer is decreased from 12 cm (rut depth=3.11 mm) to 2 cm (rut dept=23.3 mm).

5 CONCLUSION

The long-term behaviour of unbound granular materials under cyclic loading is complex. The large number of load cycles necessary to describe long-term road behaviour is the main obstacle for incremental methods. This obstacle can be overcome successfully with the shakedown theory. The elastoplastic model presented in this paper uses the simplified method of Zarka as a starting point. Modifications are performed in order to take into account the influence of the mean stress on the mechanical behaviour of granular materials. The simplified model requires cyclic triaxial tests, to determine the model parameters. The Drucker-Prager parameters are determined from at least three monotonic triaxial tests. The elasticity parameters require one repeated load triaxial test for the determination of the resilient behaviour and three repeated load triaxial tests with at least three loading stages for the determination of the permanent deformations. Currently the capabilities of the model to simulate the development of vertical plastic strains for large numbers of cycles have been evaluated for three unbound granular materials for flexible pavements (Habiballah 2003) and the model gives realistic results. In spite of the large

number of finite elements used for the structural analysis (over 2,000), the computing time is rather short. Although the structural analysis is encouraging, the comparison of the model predictions with the results of a full scale instrumented experiment remains to be done.

REFERENCES

- Bonaquist, R.F. & Witczack, M.W. 1997. A comprehensive constitutive model for granular materials in flexible pavement structures. *Proc. 4th international conference on Asphalt Pavement*: 783–802. Seattle, Washington.
- Chazallon, C. 2000. An elastoplastic model with kinematics hardening for unbound aggregates in road. In Dawson (ed), *Unbound Aggregates in Road Construction UNBAR5*:265–270. Rotterdam: Balkema.
- Habiballah, T.M. 2003. Modélisation des déformations permanentes des graves non traitées: Application au calcul de l'orniérage des chaussées souples. *Ph.D Thesis*, Université de Limoges.
- Richer, P., Daouadji, A. & Fedghouche, D. 1999. Elastoplastic modelling of the cyclic behaviour of granular materials. In Gomes Correia (ed.), *Unbound granular testing, In-situ testing and modelling*: 161–168. Rotterdam: Balkema.
- Hicks, R.G. & Monismith, C.L. 1972. Prediction of the resilient response of pavement containing granular layers using non-linear elastic theory. *Proc. 3th International Conference on Asphalt Pavements*, Vol. 1: 410–429.
- Hornych, P. et al. 1993. Rapport interne confidentiel, L.C.P.C. Nantes.
- Hornych, P. 2003. Etude des déformations permanentes sous chargements répétés de trois graves non traitées. *Bull. liaison Laboratoire Ponts et Chaussées.*, n°184, mars- avril: 77–84.
- Zarka, J. & Casier, J. 1979. Elastic plastic response of structure to cyclic loading: practical rules. In Nemat-Nasse (ed.), *Mechanics today*, Vol 6, Pergamon Press: 93–198.

Empirical shear strength models for unbound road-building materials

H.L.Theyse

CSIR Transportek, Pretoria, South Africa

Pavements Unbound—Dawson (ed.)

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ABSTRACT: The use of unbound structural pavement layers is fairly common in many countries. A number of mechanistic-empirical design models have been developed for these layers relating the plastic deformation response of the unbound material to the ratio between imposed shear stress and the shear strength of the material. In the case of South Africa, the shear strength of the material used to be characterised by the Mohr-Coulomb shear strength parameters. The shear strength of unbound material is, however, highly dependent on the relative density and degree of saturation of the material. This paper presents a model that characterises the shear strength of unbound materials including the effect of relative density and degree of saturation. The model is calibrated with very high accuracy for two examples consisting of a crushed stone and natural gravel material.

1 INTRODUCTION

The use of granular base layers in combination with a thin surfacing layer less than 50 mm thick is common pavement engineering practice in South Africa. Crushed stone is normally used for the construction of these base layers on roads carrying high traffic volumes while natural gravel is used for roads carrying lesser traffic. This type of pavement is shown by the example in Figure 1. Given the design described above, the unbound base layer is subjected to extremely high stress conditions under traffic loading and may contribute substantially to the plastic deformation of the total pavement. Generally, the permanent deformation response of these base layers is affected by a number of factors of which the following four are probably the most important:

- The material's quality that determines the material's resistance to permanent deformation;

- The density achieved during the compaction of the layer;

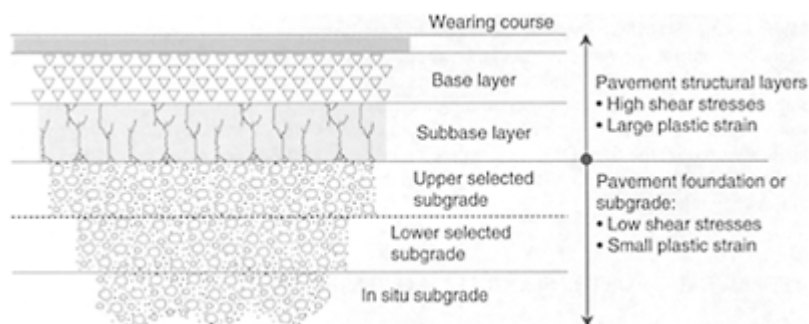


Figure 1. Stress regimes in a typical South African pavement.

- The moisture content of the base layer that may increase significantly when the surfacing layer and drainage systems are not maintained properly; and
- The magnitude and number of stress cycles to which the layer is subjected.

The density and moisture content (or degree of saturation) of the material, together with the material quality, essentially determine the material's resistance to permanent deformation. The permanent deformation resistance of unbound base layers have been found to be excellent when well constructed (high density) and well maintained (dry). Under these conditions the permanent deformation of the unbound layer will accumulate at a very gradual rate that remains stable. On the other hand, when poorly compacted and allowed to become wet, these layers deform rapidly and may even shear, entering an unstable condition.

Maree, 1978 developed a mechanistic-empirical model to safeguard against the rapid shear failure of unbound base layers by setting minimum requirements in terms of the ratio between the imposed shear stress and the shear strength of the material (called the Factor of Safety by Maree). In time, the work done by Maree was also interpreted as relating the Factor of Safety to the number of load repetitions that will result in the layer reaching an unacceptable level of permanent deformation through gradual deformation while the imposed shear stress remains well below the shear strength of the material. Although this approach is questioned by Lekarp, 2000, other researchers (Huurman, 1997 and Theyse, 2000) developed similar models relating the permanent or plastic deformation of unbound material to the ratio between the imposed shear stress and the shear strength of the material (Stress Ratio) even at stress conditions below the static shear strength of the material.

The approach followed by Maree, 1978 is believed to be sound in principle as it identifies the two important aspects that determines the permanent deformation behaviour of unbound material, namely the imposed shear stress and the material's resistance to permanent deformation, in this case measured by the static shear strength of the material. A derivative of this approach will therefore be used in future versions of the South African Mechanistic-Empirical Design Method (SAMDM). It is however, the author's

opinion that the application of Maree's design model for unbound materials in the SAMDM placed too much emphasis on the imposed shear stresses (wheel load) while not fully allowing for the complex nature of the shear strength or permanent deformation resistance of the material. Evidence of this can readily be found on the South African road network. A section of road with a consistent design deforms in certain locations while the traffic load is the same for the full length of the section or a section of road that performed well in the past, suddenly exhibits shear failure in a wet season. What is the cause of the failure, a sudden increase in the imposed stress condition? The answer is in fact no, it is rather a case of a localised or sudden reduction in the permanent deformation resistance or shear strength of the material. So, instead of the imposed shear stress approaching the shear strength of the material, the shear strength is reduced to approach the imposed shear stress. This reduction may be caused by an increase in the degree of saturation as is the case for the two examples mentioned above or localised compaction problems resulting in "weak spots", or even worse a combination of these two factors.

Whereas the SAMDM previously considered the shear strength of unbound materials to be largely constant for a given material class, the shear strength is in fact a complex function of the density, degree of saturation and confinement pressure. This paper presents data to illustrate the complex nature of the shear strength of unbound road-building material and suggests an alternative model for the shear strength of the material in place of the Mohr-Coulomb failure envelope previously used by the SAMDM. Although the model is illustrated and calibrated at the hand of data for South African materials, it is believed that the model may be calibrated for unbound material from any other region.

2 FORMULATION OF THE SHEAR STRENGTH MODEL

The formulation of the Stress Ratio (SR) derived from the original formulation of the Factor of Safety is given by two alternative formulations in Equations 1 and 2. In the case of Equation 1, the Stress Ratio is defined in terms of the deviator stress while Equation 2 defines the Stress Ratio in terms of the major principal stress. In both cases the shear strength of the material is calculated from the Mohr-Coulomb shear strength parameters, namely cohesion and angle of internal friction.

$$SR = \frac{\sigma_1^a - \sigma_3}{\sigma_1^m - \sigma_3} = \frac{\sigma_1^a - \sigma_3}{\sigma_3 \left(\tan^2 \left(45^\circ + \frac{\phi}{2} \right) - 1 \right) + 2C \tan \left(45^\circ + \frac{\phi}{2} \right)} \quad (1)$$

$$SR = \frac{\sigma_1^a}{\sigma_1^m} = \frac{\sigma_1^a - \sigma_3}{\sigma_3 \tan^2 \left(45^\circ + \frac{\phi}{2} \right) + 2C \tan \left(45^\circ + \frac{\phi}{2} \right)} \quad (2)$$

Where σ =principal stress (kPa); τ =shear stress (kPa); ϕ =angle of internal friction ($^\circ$); C =cohesion (kPa); σ_1^m =maximum allowable major principal stress (kPa) given C , ϕ and σ_3 ; σ_1^a = applied major principal stress (kPa); σ_3 =minor principal stress or confining pressure for the tri-axial test (kPa).

The Mohr-Coulomb shear strength parameters are however, superfluous in the application of the Stress Ratio. The reason for this is that static tri-axial experimental work in the laboratory records the minor principal stress (confinement pressure) and deviator stress ($\sigma_d = \sigma_1 - \sigma_3$) from which the major principal stress can be calculated ($\sigma_1 = \sigma_d + \sigma_3$). These data are then used to obtain the linear Mohr-Coulomb shear strength parameters for the given density and degree of saturation at which the specimens were prepared. The Mohr-Coulomb shear strength parameters are then used in the design calculations to calculate either the maximum allowable principal stress or deviator stress depending on the formulation used for the Stress Ratio. The Mohr-Coulomb shear strength model is therefore an intermediate step to convert from experimental minor and major principal stress values to shear strength parameters and then back again to maximum allowable major principal stress given the value of the minor principal stress. If a model could be found to estimate the maximum allowable principal stress directly from the minor principal stress, density and degree of saturation of the material, the intermediate step will not be required any longer. This model should preferably incorporate the non-linear (Maree, 1978) characteristics of the shear strength of unbound material and should estimate the shear strength of the material over a range of appropriate density and degree of saturation values at which the material will function.

Although a large volume of work has been done to develop shear strength models for unsaturated granular material by considering the apparent cohesion as a function of the soil suction (Bishop et al, 1960, Fredlund et al, 1978 and Heath, 2002), the effect of density is not directly incorporated in these models. After consideration of the tri-axial results for a range of unbound road-building materials at different combinations of dry density and degree of saturation, a new model given by Equation 3 was formulated. Figures 2 and 3 show illustrations of the model calibrated for a recycled, crushed hornfels material.

$$\sigma_1^m = S \left(\frac{e^{aRD} \times \sigma_3^c}{e^{bS}} \right) \quad (3)$$

The model in Figure 2 is plotted as a function of the confining stress and degree of saturation at three levels of relative density, 80, 84 and 88% of solid density while the model in Figure 3 is shown as a function of relative density and degree of saturation at four levels of confinement, 20, 80, 140 and 200 kPa. The shear strength model is highly non-linear for degree of saturation with a peak shear strength occurring at about 25 to 30% saturation for the example shown. The shear strength drops of rapidly to either side of this saturation level.

An advantage of the shear strength model given by Equation 3 is the fact that it can be transformed to the equivalent shown in Equation 4 to allow for linear regression of the experimental data. The variables are according to the definition in Equation 3.

$$\ln \left(\frac{\sigma_1^m}{S} \right) = aRD - bS + c \ln \sigma_3 \quad (4)$$

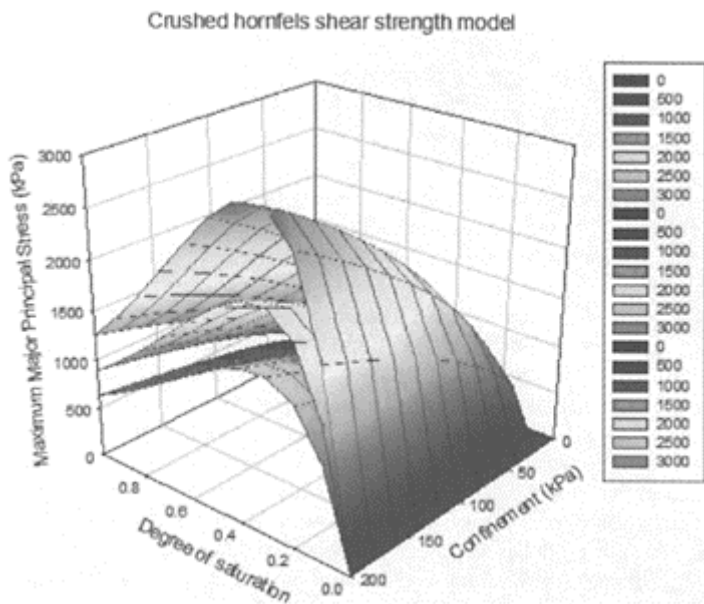


Figure 2. Shear strength model for crushed hornfels.

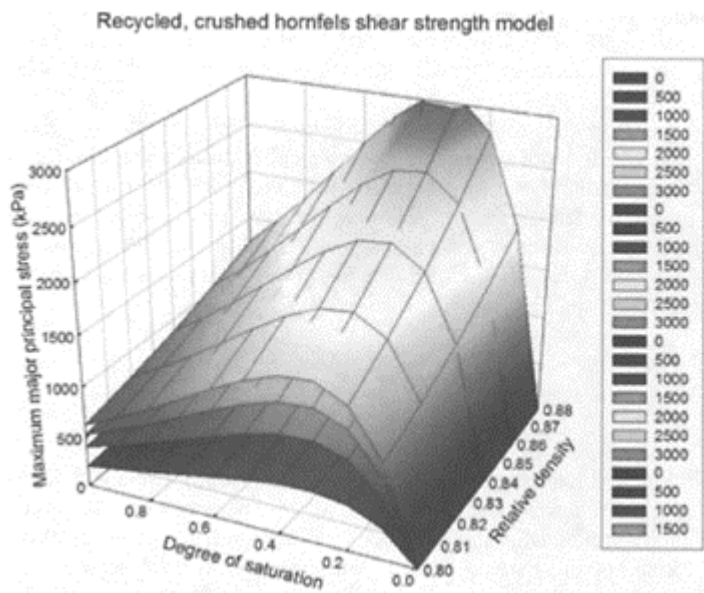


Figure 3. Shear strength model for crushed hornfels.

3 MATERIAL PROPERTIES OF THE EXAMPLE CASES

The data from two material samples are used as examples to illustrate the calibration and characteristics of the shear strength model. The one material is recycled, crushed hornfels and the other is shale natural gravel. Figures 4 and 5 shows the grading of these two materials plotted with the grading envelopes that are specified for crushed stone and natural gravel base layer material in South Africa. The grading of the crushed hornfels material is on the coarse side of the grading envelope which should assist in the compaction of the material (Simmelink, 1991). The grading of the shale natural gravel on the other hand is on the fine side of the grading envelope for natural

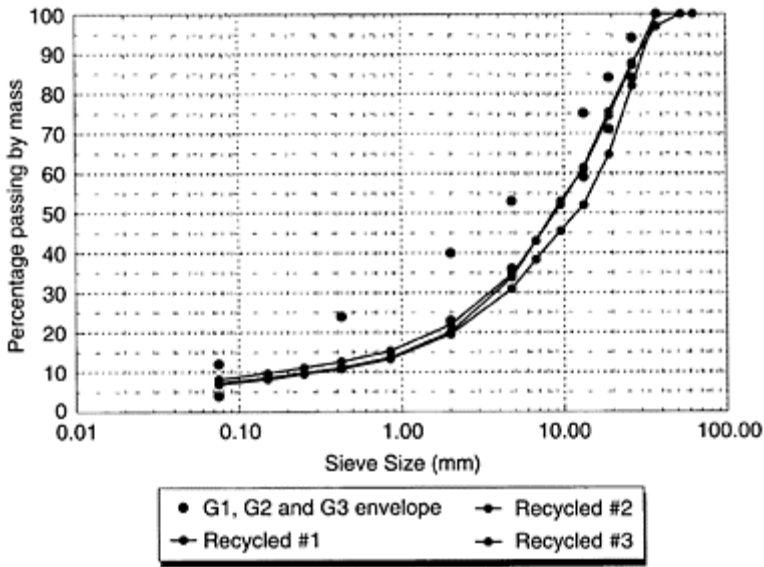


Figure 4. Grading of the recycled crushed hornfels.

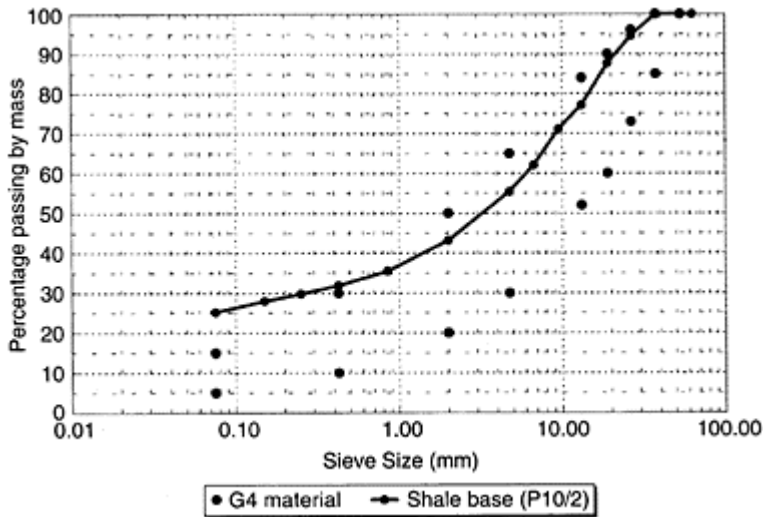


Figure 5. Grading of the shale natural gravel.

Table 1. Material properties for the recycled, crushed hornfels and shale natural gravel.

Property	Material	
	Recycled, crushed hornfels	Shale natural gravel
Liquid limit	29	23
Plastic limit	19	15
Plasticity index	10	8
Bar linear shrinkage	7	—
Swell (%)	0,8	—
Maximum dry density (kg/m^3)	2062	2322
Optimum moisture content (%)	7,9	5,8
Apparent relative density	2,619	2,711

gravel with an excess of fines. This material is not expected to compact well. The other properties of these materials are summarised in Table 1.

4 TRI-AXIAL TESTING AND MODEL CALIBRATION

Twelve 152 mm diameter by 300 mm high specimens were prepared from the recycled, crushed hornfels for tri-axial testing using vibratory table compaction. These were prepared in batches of 4 each with the following target relative density and degree of saturation levels:

- 87% relative density and 75% saturation;
- 87% relative density and 40% saturation;
- 81% relative density and 75% saturation; and
- 81% relative density and 75% saturation.

A total of 6 batches consisting of four samples each were prepared from the shale at the following combinations of relative density and degree of saturation:

- 73% relative density and 89% saturation;
- 73% relative density and 75% saturation;
- 73% relative density and 45% saturation;
- 73% relative density and 25% saturation;
- 67% relative density and 75% saturation; and
- 67% relative density and 75% saturation.

The four specimens from each batch were tested in the conventional static tri-axial test at target confinement pressure levels of 20, 80, 140 and 200 kPa and a constant displacement rate of 2 mm/min. The actual cell pressure and the load were recorded at 1 second intervals during the tests. The volume and wet mass of each specimen were recorded prior to each test and the moisture content was determined after completion of the test. The relative density and degree of saturation could therefore be calculated for each specimen. The instantaneous and maximum major principal stresses were calculated from the recorded load. The relative density, degree of saturation, confinement pressure and maximum allowable major principal stress were therefore known for each test and the data for the full set of specimens prepared from each of the materials were combined in the calibration of the shear strength models for the materials. Table 2 summarises the calibration data of the transformed models (Equation 4) for these two materials with the variables defined in accordance with Equation 3.

Each of the three variables included in the model were found to be statistically significant at the 99% confidence level for both materials. The model explains 99.9% of the variability in the

Table 2. Calibration data of the transformed shear strength model for the recycled, crushed hornfels and shale natural gravel.

Model parameters	Material	
	Recycled, crushed hornfels	Shale natural gravel

a (RD)	8,590	9,950
b (S)	−3,006	−4,090
c (σ_3)	0,486	0,483
R ²	0,9997	0,9998
SEE	0,1359	0,0892

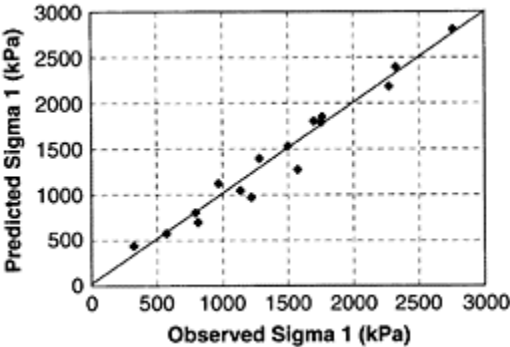


Figure 6. Observed and predicted σ_1 results for the recycled, crushed hornfels.

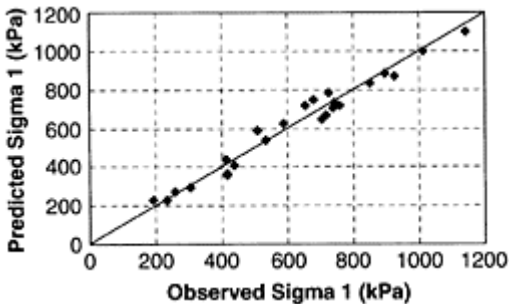


Figure 7. Observed and predicted σ_1 results for the shale natural gravel.

transformed maximum major principal stress. This accuracy is, however, reduced when the data is transformed back to maximum major principal stress. The accuracy of the model is illustrated in the following section at the hand of the actual data.

It is interesting to note that the range of relative densities that could be achieved in the laboratory varied between 80 and 88% for the recycled, crushed hornfels and 65 to 75% for the shale natural gravel. These ranges were expected given the grading of the material as discussed earlier and are appropriate ranges of relative density that may realistically be

achieved during construction. It should be kept in mind that the shear strength model is only applicable to these ranges of relative density. Given these differences in relative density, the allowable major principal stress of the two materials also differed significantly. Figures 6 and 7 show the observed and predicted allowable major principal stress results for the two materials. The maximum result recorded for the shale natural gravel is less than 1200 kPa while the corresponding result for the recycled, crushed hornfels is just less than 3000 kPa.

5 CHARACTERISTICS AND APPLICATION OF THE MODEL

The calibrated allowable major principal stress models (Equation 3 and Table 2) are used in this section to illustrate the characteristics and application of the models. A set of static tri-axial test results may be generated artificially from the model for any required combination of relative density, degree of saturation and confinement pressure within the calibration ranges for the two materials.

Figure 8 shows the Mohr-Coulomb representation of the actual static tri-axial results for the recycled, crushed hornfels at 87% target relative density and 40% target saturation levels as well as the results predicted from the model using the actual relative densities and saturation levels of the individual specimens.

In the example shown in Figure 8, the predicted result at 20 kPa confinement is well below the observed result. Closer inspection of the data did, however, reveal that the saturation level of the particular specimen used for this test was 3% below the average of the saturation levels of the other three specimens which only varied by 0.5% from the lowest to the highest value. The observed result for the 20 kPa confinement case is therefore not a valid observation for this set of results and the model gives a better estimate of the result at the appropriate density and saturation values than the observed result. The Mohr-Coulomb failure envelope fitted to the data is also influenced by the error in the specimen preparation. This type of experimental error does not pose a problem to the model presented in this paper as every result is used with the actual relative density and saturation level in the calibration of the model. A similar example to the one described above is shown for the shale natural gravel in Figures 9 and 10.

In this example, the relative densities of the two specimens tested at 80 and 140 kPa were higher than that of the specimens tested at 20 and 200 kPa resulting in the two middle circles being disproportionately large for the actual data. In addition to this, the saturation of the specimen tested at 200 kPa was 1% higher than the average saturation level of the other three specimens. This higher saturation combined with the relatively lower relative density resulted in the very low actual allowable major principal stress result for this specimen. Again the model results that were generated at the target density and saturation levels give more representative results than the actual observations because of experimental error. The non-linear nature of the failure envelope generated from the model can also be observed in Figure 10. This characteristic of the model ties in with experimental observations by Maree, 1978.

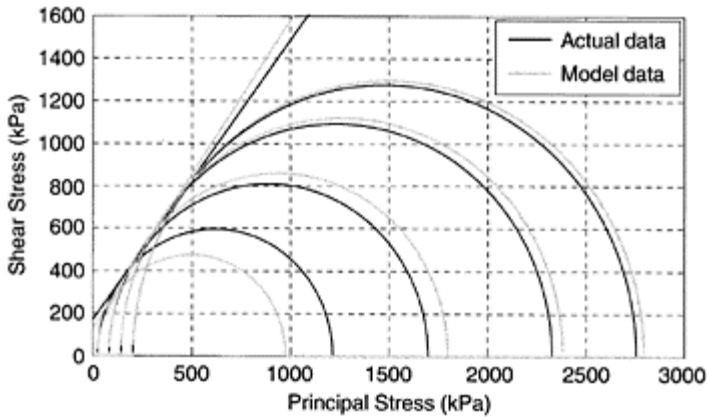


Figure 8. Mohr-Coulomb representation of the actual and predicted static tri-axial test results.

6 CONCLUSIONS AND RECOMMENDATIONS

This paper presents the successful development of a model to determine shear failure surfaces for unbound road-building material incorporating the effect of relative density, degree of saturation and minor principal stress. The model is empirical but so is the Mohr-Coulomb failure envelope and the model reflects the correct characteristics as would be anticipated from analytical considerations.

The calibration and characteristics of the model were illustrated at the hand of two examples, the one a crushed stone and the other a natural gravel. The model was calibrated with a high degree of accuracy for both materials and all three variables included in the model were found to be significant at the 99% confidence level.

The calibration of the model will be continued in future for a range of natural gravel and crushed stone materials. The application of the model must, however, be limited to the ranges of the variables for which it is calibrated. Practical ranges for these variables will, however, be determined by the characteristics of the material under investigation. The grading of the material will for example largely determine the level of compaction that may be achieved.

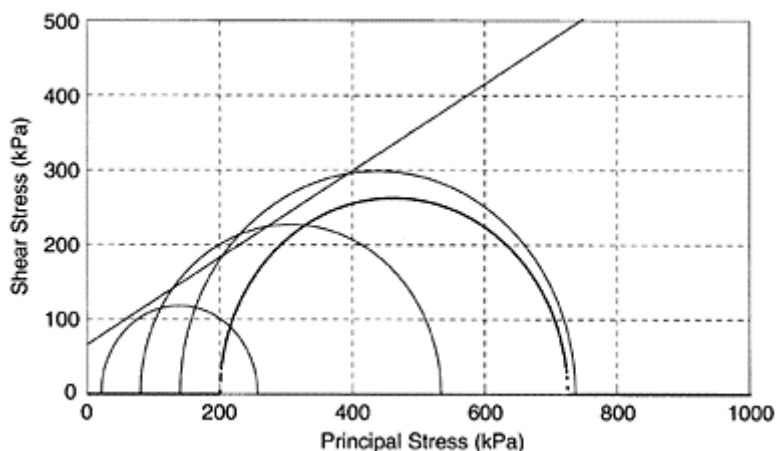


Figure 9. Mohr-Coulomb representation of the actual static tri-axial test results.

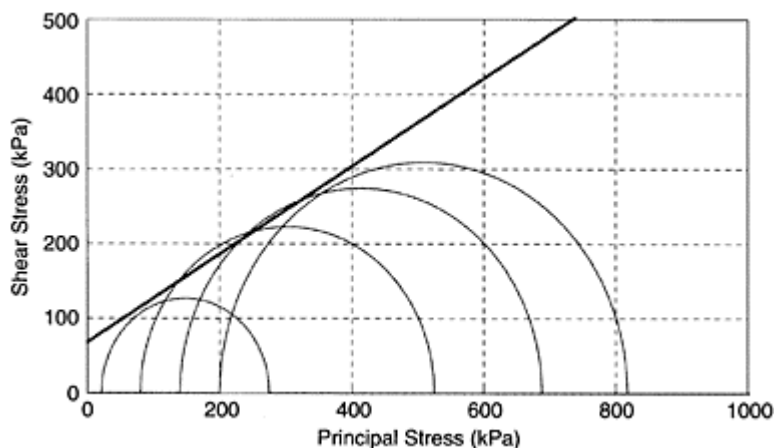


Figure 10. Mohr-Coulomb representation of the predicted static tri-axial test results.

The model appears to be robust as far as experimental error is concerned. In fact, experimental error contributes positively to the calibration of the model.

The implementation of the model (calibrated for a range of materials) in a mechanistic-empirical design procedure will allow the quality of construction and maintenance to be introduced in the design method in a rational way. The spatial

variation of the density and saturation of the material along a section of road could also be introduced in the design method using the model in the same way.

REFERENCES

- Bishop, A W, Alpan, I, Blight, G E, & Donald, I B. 1960. Factors controlling the shear strength of partly saturated cohesive soils. *Proceeding of the American Society of Civil Engineers Research Conference on Shear Strength of Cohesive Soils*, Boulder, Colorado, pp. 505–532.
- Fredlund, D G, Morgenstern, N R, & Widgen, R A. 1978. The shear strength of unsaturated soils. *Canadian Geotechnical Journal*, 15, pp. 313–321.
- Heath, 2002. *Modelling unsaturated granular pavement materials using bounding surface plasticity*. Ph D thesis, University of California, Berkeley.
- Huurman, M. 1997, *Permanent deformation in concrete block pavements*, Ph D thesis, Technical University of Delft, The Netherlands.
- Lekarp, F.Isacsson, U. & Dawson, A. 2000. State of the Art. II: Permanent strain response of unbound aggregates. *ASCE Journal of Transportation Engineering*, January 2000. pp. 76–83.
- Maree, J H. 1978, *Ontwerpparameters vir klipslag in plaveisels. (Design parameters for crushed stone in pavements)*, M Eng thesis, University of Pretoria, South Africa.
- Semmelink, C J. 1991. *The effect of material properties on the comparability of some untreated road-building materials*, Ph D thesis, University of Pretoria, South Africa.
- Theyse, H L. 2000. The Development of Mechanistic-Empirical Permanent Deformation Design Models for Unbound Pavement Materials from Laboratory and Accelerated Pavement Test Data, in *Unbound Aggregates in Roads (Proc. 5th International Symposium 'UNBAR5')*, ed. Dawson, A.R., Balkema, 2000, pp. 285–293.

Design criteria of granular pavement layers

S.Werkmeister, F.Wellner, M.Oeser & B.Moeller

*Chair of pavement engineering, Chair of mechanics, Dresden University
of Technology, Germany*

Pavements Unbound—Dawson (ed.)

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ABSTRACT: A new simple design approach will be described that utilizes test results from the Repeated Load Triaxial Apparatus to establish the risk level of permanent deformations in the unbound granular layers (UGL) in pavement constructions under consideration of the properties of different unbound granular materials (UGM). From this data a serviceability limit line (plastic shakedown limit) stress boundary for different UGM was defined. Below this line the material will have stable behavior. It will be shown that the proposed design approach is a very satisfactory simple method to assess the risk against rutting in the UGL, even without the calculation of the exact permanent deformation of the pavement construction. Additionally, a plastic model (DRESDEN-Model) was developed from the data's of Repeated Load Triaxial Tests. The plastic model and a nonlinear elastic model (DRESDEN-Model), which is also described in the paper, were implemented into the 3-D FE-program FALTFEM. To check the validity of the plastic DRESDEN-Model, field tests in a test section (2.5m×2.5 m×1.5 m) on UGLs using cyclic loading (1,000,000 load repetitions) were carried out. The surface deformation induced by dynamic loading was predicted using the 3-D FE-program FALTFEM. A comparison was carried out to assess the accuracy of the plastic model by comparing the results of calculated deformations from the plastic model against the measured values.

1 INTRODUCTION

In order to determine the most economical combination of layer thickness and material types for a pavement, it is necessary to develop analytical pavement design methods on the basis of finite-element (FE)-calculations as opposed to empirical design methods. Furthermore analytical design methods would need to take into account the properties of the soil foundation and the traffic to be carried during the service life of the road. A prerequisite for any successful analytical design methodology is the acquisition of reliable measurements from representative experimental investigations followed by appropriate mathematical characterization of the deformation behaviour of both the bound and unbound materials used in pavement construction.

Numerical models permit a realistic description of the load-bearing behaviour of pavement constructions. With the aid of numerical models the effects of different external factors or structural parameters may be systematically assessed for practical engineering applications. To achieve this, the numerical models to be developed must be adequate for dealing with the particular problem in question. In order to analyze the load-bearing behaviour of pavement constructions, 3-D models should be chosen which take account of a variety of factors such as the highly variable stress fields encountered e.g. in regions of rut formation, the rheological properties of the asphalt and the properties of Unbound Granular Materials (UGM) in the non-cohesive base layer.

2 RESEARCH PROJECT AND TESTING PROCEDURE

This paper reports on a research project at the chair of pavement engineering, Dresden University of Technology, which is aimed at developing a model to describe the resilient and permanent deformation behaviour of Unbound Granular Materials (UGM) in pavement constructions. A serviceability based design method (analytical approach) will be described that utilizes test results from the Repeated Load Triaxial (RLT) apparatus to establish the risk level of permanent deformations in the UGL. This research on the deformation behaviour of UGM is a prerequisite for an analytical design program for flexible pavements, which is under development at the Dresden University of Technology. Finally the plastic model and a nonlinear elastic model (DRESDEN Model) were implemented into the 3-D FE-program FALTFEM.

The RLT apparatus used in the project has been developed at the University of Nottingham, and can simulate dynamic pavement loadings. A Sandy Gravel and a Granodiorite with a maximum grain size of 32 mm were tested (Werkmeister et al. 2001). The tests were conducted at 3.4% (Sandy Gravel) and 4% (Granodiorite) moisture content. For these tests the constant confining pressure levels were set at 40, 70, 140 and 210 kPa. For each test, once the confining pressure was achieved, an additional dynamic vertical stress (deviator stress) was applied at a frequency of 5 Hz. The triaxial tests were carried out using dynamic axial stresses with stress ratios (σ_D/σ_3) in the range 0.5 to 11.

3 MODELING OF THE DEFORMATION BEHAVIOUR

3.1 *Shakedown analysis of pavement constructions*

The essence of a shakedown analysis is to determine the critical shakedown load for a given pavement. Pavements operating above the critical shakedown load are predicted to exhibit increased accumulation of permanent strains under long term repeated loading conditions that eventually lead to incremental collapse (e.g. rutting). Those pavements operating at load levels below this critical shakedown load may exhibit some distress, but should settle down and reach an equilibrium state in which no further mechanical deterioration occurs (Werkmeister et al. 2001). Traditional pavement design methods (e.g. German pavement design guidelines, (RStO 2001)) assume that the pavement deteriorates indefinitely. However, there is ample field evidence that this is not always

true and that steady-state conditions are frequently achieved. The shakedown approach can be used to characterize the deformation behavior of UGM in pavement constructions. The application of the shakedown concept to UGM as used in pavement construction is possible, although adaptations have to be made to allow for the particular response of UGM to repeated loading. Behavior can be categorized into 3 possible Ranges A, B or C (Figure 1) (Werkmeister et al. 2001).

If the UGLs behave in a manner corresponding to Range A, the pavement will “shake down”. After post-compaction deformations, no further significant permanent strains develop and the material subsequently responds nearly elastically. Thus Range A is permitted in a pavement, provided that the accumulated strain before the development of fully resilient behaviour is sufficiently small. The material in Range B does not “shake down”, rather it will achieve failure at a very high number of load repetitions. In that case the resilient strains are no longer constant and

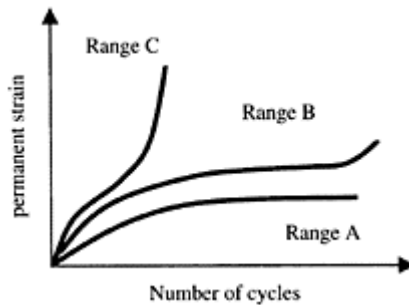


Figure 1. Indicative permanent strain behavior (Werkmeister 2003).

will increase slowly (decrease of stiffness). Range C behaviour—incremental collapse or failure—should not be allowed to occur in a pavement (Werkmeister et al. 2002). However dilatancy (dilatant behaviour allows heavily stressed areas to build up increasing confinement and thus reduces permanent deformation) should be considered in Range B and C for modelling the resilient and permanent deformation behaviour of UGMs.

Shakedown limit calculations (critical shakedown load) can be used to predict whether or not stable behaviour occurs in the UGL of the pavement construction (Werkmeister et al. 2001). The shakedown analysis of Repeated Load Triaxial (RLT) test results can be used for ranking materials as a performance specification method to determine the resistance against rutting of UGMs (Werkmeister et al. 2003). Of course the shakedown limits of the UGL are also strongly dependent on seasonal effects (mainly moisture content). The moisture content has been identified as the factor having the largest influence on the mechanical properties of UGM (e.g. Werkmeister et al. 2003).

Analysis of the results from many permanent deformation RLT tests revealed an exponential relationship (Equation 1) between the applied stresses ($s_{1\max}/s_3$) and the boundaries of the various deformation responses (i.e. between Ranges A, B and C as shown in Figure 2).

$$\sigma_{1\max} = \alpha \cdot \left(\frac{\sigma_{1\max}}{\sigma_3} \right)^\beta \quad (1)$$

where $\sigma_{1\max}$ [kPa]=peak axial stress; σ_3 [kPa]=cell pressure (minor principal stress); α [kPa]=material parameter; β [-]=material parameter (Werkmeister 2003).

With this equation it is possible to deduce the shakedown limit even at small stress ratios (Figure 2).

As a practical method of defining the range boundaries (which define the stress conditions at which the type of permanent strain response changes) and, hence, the material parameters for Equation 1, RLT tests are performed on a series of specimens (or in a multi-stage test on one specimen) at increasing $\sigma_{1\max}/\sigma_3$ ratios. When the plastic axial strain accumulated from 3,000 to 5,000 load applications is 0.045×10^{-3} strain, the range A-B boundary (the “Shakedown Limit”) is reached. When this strain equals 0.4×10^{-3} strain, the range B-C boundary (the “Plastic Creep Limit”) is reached (Werkmeister 2003). As there is an associated change in resilient behaviour for

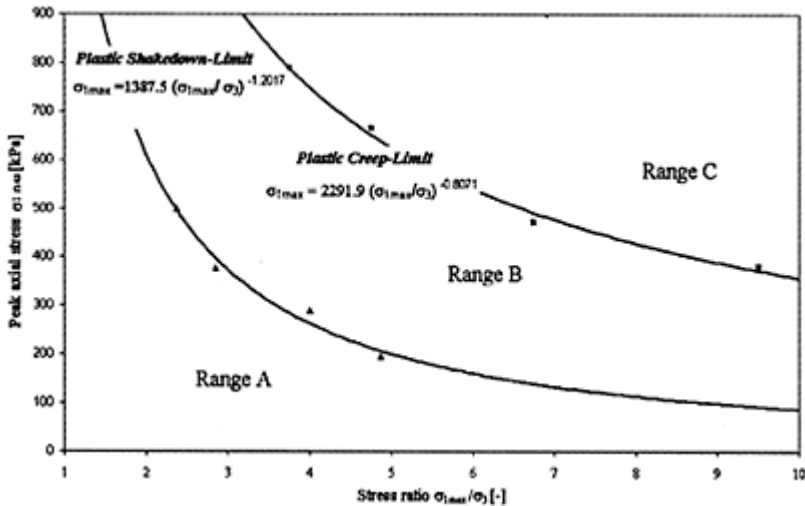


Figure 2. Stress ratio versus peak axial stress, Granodiorite at 4% water content (3).

materials operating in the various ranges, it is recommended that the observed response Ranges A, B and C should form the basis for modelling permanent and resilient deformation behaviour. Thus material laws have to be developed for each separate range. Range A is the most important range because stable behaviour will be the predominant requirement for UGLs in high trafficked pavement constructions.

4 DESIGN CHART

The two most critical forms of distress in flexible pavements, which all analytical pavement design methods attempt to control are:

- The risk of rutting within the pavement layers and
- asphalt fatigue cracking.

In this paper a design criterion will be presented to evaluate the performance of the UGL.

Granular base and asphalt rutting are a common form of flexible pavement distress. In a flexible pavement, asphalt rutting must be controlled by proper material selection and mix design. This subject will not be discussed further in this paper. Granular base rutting can be controlled by using a better (grading, aggregate shape etc.) material for the UGL. Also, increasing the thickness of the asphalt layers reduces the stresses in the UGL and hence, reduces rutting in the granular base. In fact, granular base rutting can be avoided by limiting the stress in the UGL. In most common analytical-mechanistic design methods, the pavement life, in terms of the allowable number of load cycles of a certain magnitude, is determined on one hand by calculating the vertical compressive stress on top of the subgrade (and on the other hand by determination of the maximum tensile stress at the bottom of the asphalt layer, i.e. risk of asphalt fatigue cracking).

This research support the work done by Peterson & Maree (Peterson et al. 1980), that not only the major stress but also the minor principal stress within the UGL should be considered. For this reason a critical stress level must be defined for the UGL by considering the principle stresses σ_1 and σ_3 (using shakedown analysis). This critical stress level can be used as a simple design method to avoid granular base rutting in pavement constructions. The design process proposed can be used as a check on whether or not a stabilizing behaviour will occur in the UGL. This is performed by comparison of the shakedown limit with the maximum expected stresses in the unbound pavement layer (Figure 3).

The periods with very high asphalt surface temperatures are the most critical periods of each year. The pavement must survive these periods without incurring excessive surface rutting or other forms of distress caused by the UGL. The moisture effects and the influence of the asphalt temperature must be taken into consideration in the design process. Should the plastic shakedown limit be exceeded, then a risk of high permanent deformation in the UGL exists. Nevertheless, it may be possible to accept Range B behaviour in UGLs for Low Volume Roads, because of the lower number of load repetitions and provided softer covering asphalt layers are used which can follow the occurring deformations without deterioration. In that case the determination of the amount of permanent deformation might be necessary.

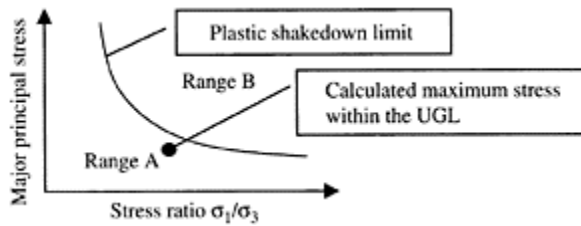


Figure 3. Design chart.

Table 1. Elastic model-coefficients for the Sandy Gravel grading M, DOC=100%, Range A (Numrich 2003).

Parameter	Moisture content	
Elastic	DRESDEN-Model	3.5%
Q	$[\text{kPa}]^{1-Q_2}$	16, 158.3
C	$[\text{kPa}]^{1-Q_1-Q_2}$	3862.0
Q ₁	[-]	0.561
Q ₂	[-]	0.33
R	[-]	0.015
A	$[\text{kPa}]^{-1}$	-0.0018
B	[-]	0.285

4.1 Modelling the deformation behaviour of Unbound Granular Materials

4.1.1 DRESDEN model (resilient deformation behaviour)

Investigations of the non-linear elastic stress-strain-behaviour of UGM have been carried out for the past 10 years at Dresden University of Technology. In this section only a short overview on the modelling of the UGM can be given. Further details are available elsewhere (Gleitz 1996), (Wellner 1994), (Wellner 1996).

Modified plate-bearings tests with cyclic loadings (Wellner 1996) were carried out on UGLs. Heaving was observed at a distance range of 450–1200 mm from the load axis. At all measured stress-levels the same behaviour was observed. Linear elastic analysis did not predict this heaving and therefore RLT tests on the same UGM as used for the plate-bearings tests were conducted to investigate the non-linear behaviour. As a result of the data from the RLT testing a new material law—the DRESDEN Model was developed (Gleitz 1996). This non-linear elastic model is expressed in terms of modulus of elasticity E and Poisson's ratio ν as follows:

$$E = (Q + C \cdot \sigma_3^{Q_1}) \cdot \sigma_1^{Q_2} + D \quad (2)$$

$$\nu = R \cdot \frac{\sigma_1}{\sigma_3} + A \cdot \sigma_1 + B \quad (0 < \nu < 0.5) \quad (3)$$

where σ_3 [kPa]=minor principal stress (absolute value); σ_1 [kPa]=major principal stress (absolute value); D [kPa]=constant term of modulus of elasticity; Q, C, Q_1 , Q_2 , R, A, B=model parameters, determined with RLT.

The model includes a stress independent stiffness of 38 kPa for crushed aggregates and 30 kPa for sand and gravel (parameter D) consequent upon the residual confining stress in-situ. The residual stress has the effect of reducing the strains at small stress levels and could be assumed by examining modified plate-bearing test results carried out by KLEMT (Klemt 2001). The parameter D is mainly influenced by macroscopic parameters like the degree of compaction of the UGM, content of fines, shape of the grains and water content. The RLT results do not allow determination of the parameter D because the residual stress needs some time to develop in a real pavement construction. To obtain the model parameters the RLT apparatus at the Nottingham University was used.

5 DRESDEN MODEL (PERMANENT DEFORMATION BEHAVIOUR)

The available models of permanent deformation behaviour of UGM are much less developed than those of resilient deformation behaviour. In modelling the long-term behaviour of pavements, it is

Table 2. Plastic model-coefficients as found for the Equations 6 and 7, Sandy Gravel, grading M, DOC=100%, Range A.

Parameter		Moisture content
Plastic	DRESDEN-Model	3.4%
a_1	[-]	0.00004
a_2	[kPa ⁻¹]	-0.0247
a_3	[kPa ⁻¹]	0.00005
a_4	[-]	0.4257
b_1	[-]	0.0009
b_2	[kPa ⁻¹]	-0.0107
b_3	[kPa ⁻¹]	0.0067
b_4	[-]	0.5579

essential for the analysis to take into account the gradual accumulation of permanent strain with the number of load repetitions and the important role played by stresses. Hence the main objective of research into long term behaviour should be to establish a

constitutive model which predicts the amount of permanent strain at any number of load repetitions at a given stress level. Ideally it should take into consideration the different deformation behaviour in the Ranges A, B and C.

A material law for the permanent deformation behaviour in Range A has been developed. The stress-dependent HUURMAN-Model serves as basis for the new model (Huurmann 1996). The first part of the model describes the deformation behaviour in Range A—a linear relationship between $\log(\epsilon_p)$ and $\log(N)$ (degressive increase of ϵ_p 1 with N), where A gives ϵ_p 1 at $N=1,000$ and B gives the slope of ϵ_p 1 with $\log(N)$. Using the second part of the HUURMAN-Model we are able to describe the behaviour also in Ranges B and C (collapse) with an exponential increase of ϵ_p 1 with N .

$$\epsilon_p^1(N) = A \cdot \left(\frac{N}{1000} \right)^B + C \left(e^{\frac{D \cdot N}{1000}} - 1 \right) \quad (4)$$

where $\epsilon_p^1 [10^{-3}]$ = vertical permanent strains; A, B, C, D [-] = model coefficients; N [-] = number of load repetitions.

The model coefficients A, B are defined for the Range A:

$$A = (a_1 \cdot e^{a_2 \cdot \sigma_1}) \sigma_1^2 + (a_3 \cdot \sigma_3^{a_4}) \sigma_1 \quad (5)$$

$$B = (b_1 \cdot e^{b_2 \cdot \sigma_1}) \sigma_1 + (b_3 \cdot \sigma_3^{b_4}) \quad (6)$$

where a_{1-4} , b_{1-4} = model parameters, determined by the RL.T.

However, it is necessary to determine different parameters for the different ranges. The parameters given in Table 2 for Equation 5 and 6 are only valid for Sandy Gravel in Range A. Further study is necessary to define parameters C and D. Details about the hypo-plasticity theory, the DRESDEN-Model and information regarding its application and calibration can be found in (Werkmeister 2003) and (Oeser et al. 2004).

6 MODEL VALIDATION

6.1 Field tests

Field tests were conducted (Rossberg et al. 1996) to check the validity of the elastic and plastic DRESDEN Model. The test section had a footprint of 2.5 m×2.5 m. A Sandy Gravel 0/32 was

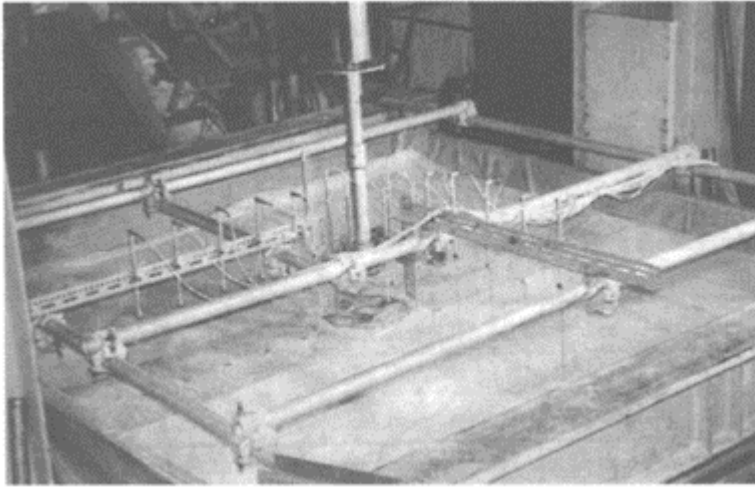


Figure 4. Test section and measuring equipment.

placed in the test section in 4 layers and compacted using a vibrating plate. On top of the UGL a degree of compaction of 98% was reached. The total thickness of all layers came up to 0.9 m.

The Sandy Gravel was loaded by a steel loading plate (rigid load action) with a diameter of 30 cm. Up to 1 million vertical stress pulses were applied with a frequency of 5 Hz and a force of 3 kN at minimum and 14.1 kN at maximum. This loading led to a contact pressure between the steel loading plate and the Sandy Gravel ranging from -0.0425 to -0.2 N/mm². In the test section several test geotextiles were placed in the construction, but as the test results showed they had no effect on the deformation behaviour of the whole construction (Rossberg et al. 1996). The permanent and resilient deformations on the surface were measured for different numbers of load cycles. Furthermore the deflection was not only measured at the loading plate, but also at different distances up to 1.5 m from the load axis (Figure 4).

6.2 FE-calculation

The deformation behaviour of the Sandy Gravel was modelled using the elastic and plastic DRESDEN-Model for Range A as introduced in this paper. The DRESDEN-Model was implemented into a 3-D Finite Element program (FALTFEM), which was developed by the chair of mechanics at the Dresden University of Technology. Special isoparametric finite elements (Bathe 2002) were used applying 60 degrees of freedom and tri-quadratic displacement shape functions.

The mounting parts considered within the FE-calculation equalled the test section of the construction. Taking into account the symmetry of the system only a quarter of this area had to be cross-linked within the calculation.

Figure 5 shows the FE-mesh used. The degree of cross-linking was especially high for ranges of high stress and strain gradients. Boundary areas with little impact on the calculation were networked more loosely. The mesh consisted of 893 elements with 13,593 displacement degrees of freedoms. The boundary-nodes of the FE-mesh were rigid. The rigid bearing corresponded to the real support of the fixing of the structure in the test section (concrete frame).

Figure 6 shows the deformation due to a vertical load of 14.1 kN and the stresses in the vertical direction. The model was depicted in its displaced configuration. The deformation was inflated by the factor 25.

Figure 7 shows the calculated and measured permanent and resilient deformation for the Sandy Gravel 0/32. The permanent deformation of two conducted tests at a time was averaged. The permanent deformation of the first 1,000 load cycles was not taken into account because the adjustment

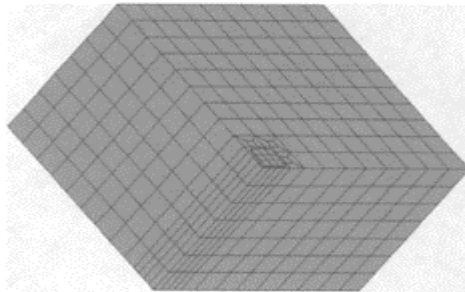


Figure 5. 3D-FE-mesh.

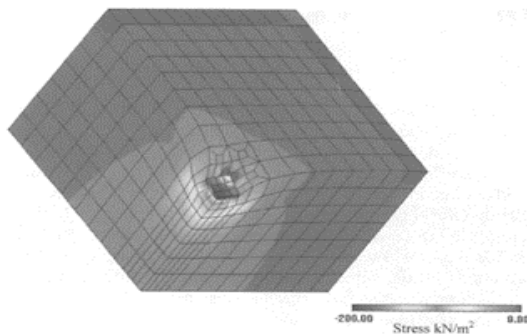


Figure 6. Stress in the vertical direction and inflated deformation $25 \cdot v_3$.

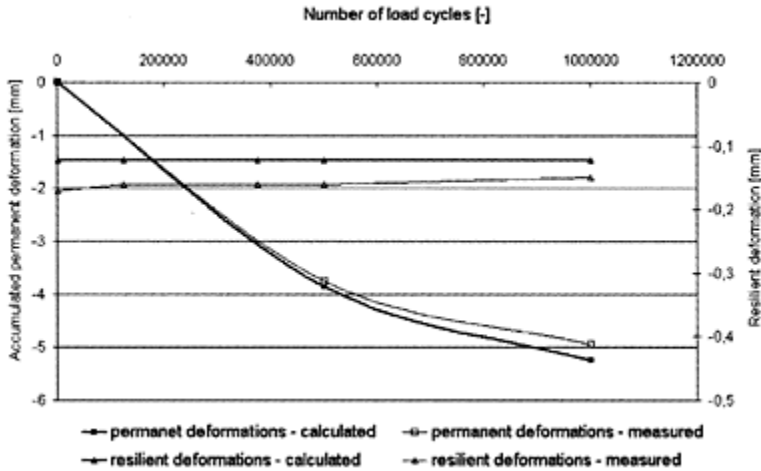


Figure 7. Permanent and resilient deformation as a function of the number of load cycles and measured and calculated data in the load axis.

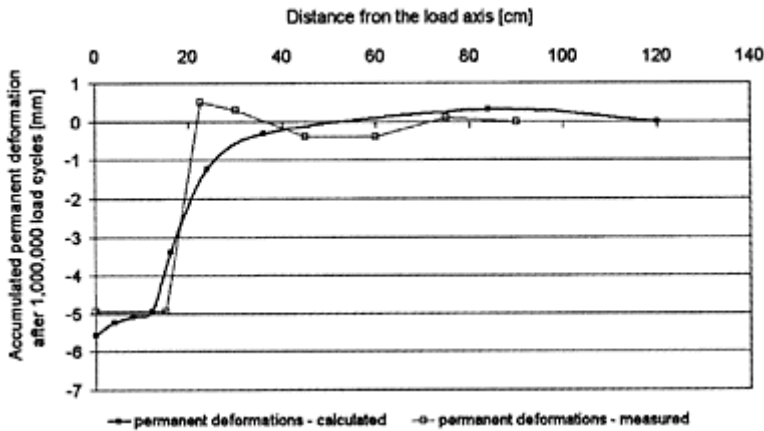


Figure 8. Permanent deformation in x direction; measured and calculated data at 1,000,000 load cycles.

of the loading plate affected the deformation behaviour significantly. As Figure 7 shows a good approximation for the measured and calculated deformation was generated by the elastic and plastic DRESDEN-Model. In addition permanent heaving (away from the loading plate) could be observed applying the DRESDEN-Model (Figure 8).

7 DISCUSSION AND CONCLUSIONS

A new simple design approach was described utilizing test results from the Repeated Load Triaxial Apparatus to establish the risk level of permanent deformations in the unbound granular layers (UGL) in pavement constructions under consideration of the properties of different unbound granular materials (UGM). By means of this data a serviceability limit line (plastic shakedown limit) stress boundary for an UGM was defined. Below this line the material showed a stable behaviour. The proposed design approach proved to be a very satisfactory simple method to assess the risk against rutting in the UGL, even without the calculation of the exact permanent deformation of the pavement construction.

In addition a plastic and an elastic model (DRESDEN Model) were developed from the data of the Repeated Load Triaxial Tests. These models were implemented into the 3-D FE-program FALTFEM. The results of field tests were used to check the validity of the elastic and plastic DRESDEN Model. The field tests were conducted in a test section (2.5 m×2.5 m×1.5 m) on Sandy Gravel under cyclic loading (1,000,000 load repetitions). The surface deformation of the investigated Sandy Gravel was determined in-situ by means of field testing. Furthermore the surface deformation could be predicted by the use of the 3-D FE-program (FALTFEM). A comparison was carried out to assess the accuracy of the DRESDEN Model by evaluating the results of the calculated deformations and the measured values. A good approximation could be confirmed. The maximum deflection under loading corresponded to the one from the measured values. In addition heaving beneath the loading plate could be detected with this model. However, further model validations are necessary. Field tests at the CAPTIF (Canterbury Accelerated Pavement Testing Indoor Facility) in Christchurch/New Zealand are planned.

ACKNOWLEDGEMENTS

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REFERENCES

- Arnold, G., Dawson A.R., Hughes, D., Werkmeister S. & Robinson, D. 2002. Serviceability Design of Granular Pavement Materials. In *Bearing Capacity of Roads, Railways and Airfields Proceedings of the 6th international Symposium on the Bearing Capacity of Roads and Airfields (BCRA)*: 957–966. Lisbon: Balkema.
- Bathe, K.J. 2002. *Finite Element-Methods*. Berlin: Springer Verlag.
- Gleitz, T. 1996. *Calculation of non linear behaviour of granular base materials in flexible pavements*, Ph D thesis (in German), University of Technology Dresden.
- Huurman, M. 1996. Rut development in concrete block pavements due to permanent strain in the substructure. In *Pave Israel*: 293–304.

- Klemt, R. 1990. *The influence of the friction on the deformation behaviour of granular base materials*. In Diploma thesis 494, University of Technology Dresden.
- Numrich, R. 2003. *Non-linear-resilient deformation behavior of unbound granular materials*, Ph D thesis (in German), University of Technology Dresden.
- Oeser, M., Werkmeister, S., Wellner, F. & Moeller, B. 2004. 3-D Numerical Model for Pavements Considering nonlinear Deformation Behaviour of Unbound Granular Materials, *ICCES'04, International Conference on Computational and Experimental Engineering and Science*. Madeira.
- Paterson, W.D.O. & MAREE, J.H. 1980. *An Interim Mechanistic Procedure for the Structural design of Asphalt Pavements*, Transportek CSIR, Pretoria, South Africa (Technical Report RP/5/78).
- Rossberg, K., Wellner, F. & Gleitz, T. 2002. Geotextiles in pavement constructions—dynamic loading tests in a test section. *Schriftenreihe des Lehrstuhls Straßenbau* Heft 4, TU Dresden.
- RStO 01 2001, German pavement design guideline. Forschungsgesellschaft für Straßen- und Verkehrswesen (in German). Köln.
- Wellner, F. 1994, *Basics elements for design of flexible pavements with granular material*. Professorial Dissertation (in German), University of Technology Dresden.
- Wellner, F. 1996. Influence of the stress dependent strain behaviour of unbound road bases on the stress of superposined top layers. In *Flexible Pavements, [Proceedings of the Euroflex Symposium, 1993, Lisbon]*, ed. A. Gomes Correia: 311–318. Rotterdam: Balkema.
- Werkmeister, S. 2003. *Permanent deformation behaviour of unbound granular materials*. Ph D thesis, University of Technology Dresden.
- Werkmeister, S., Dawson, A.R. & Wellner, F. 2001. Permanent Deformation Behaviour of Unbound Granular Materials and the Shakedown-Theory. *Transportation Research Record* 1757:75–81, TRB, National Research Council. Washington, D.C.
- Werkmeister, S., Dawson, A.R. & Wellner, F. 2003. Design of granular pavement layers considering climatic conditions, *Transportation Research Board Meeting*: Paper No. 03–2645. Washington, D.C.
- Werkmeister, S., Numrich, R. & Wellner, F. 2002. Modeling of Granular Layers in Pavement Constructions, *Proceedings, 9th Int. Soc. Asphalt Pavements Conference*. Copenhagen.

Design of low-volume roads in Lithuania

D.Zilioniene, D.Cygis & A.A.Juzenas

Vilnius Gediminas Technical University, Lithuania

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ABSTRACT: A lot of attention is paying to the solution of problems related to the renovation of gravel roads in Lithuania. Characteristics of low-volume roads depend on the type and structure of pavement, traffic volume and driving speed, road significance as well as materials used for road construction. Application of typical pavement structures is practically impossible when a gravel road is used as base or subbase course for a low-volume road. At present, the usage of four pavement types (two types per settlement and on road sections between settlements) of low-volume roads is not related to geological and hydrological conditions of a location, the thickness of the gravel road and characteristics of materials and subgrade soils. The carried out calculations of pavement structures complying with local conditions using software enable to use road construction materials more economically and guarantee the required strength of low-volume road pavement structure in each specific case.

1 INTRODUCTION

Regional roads are of utmost importance in Lithuania, and are used for communication on the territories of administrative units. They connect urban and rural areas with the main roads of the country, included in the European international road network, as well as national roads. Regional roads, which belong to the low traffic volume road group, make up 69% or more than 14,700 kilometres of the Lithuanian Republic roads of national significance. The growth of economic co-operation with Nordic and West European countries influences not only on the increasing transit flows on the motorways but also on the traffic volumes on the whole road network of Lithuania. A rapidly increasing number of vehicles (currently, the ratio is 426 vehicles per 1000 inhabitants) pose a lot of problems related to social welfare, environmental protection, life and traffic quality improvement, which have to be solved immediately. A lot of such problems are caused by the people who live on the sanitary protection zones of gravel roads, which make up more than 60% of all roads of regional significance, as well as by users of these roads.

Renovating and paving gravel roads improve the quality of life and traffic on regional roads. In 1998–2000, the Paving of Gravel Roads Program (hereafter called the Program)

was implemented, according to which 935 kilometres of gravel roads were paved. The implementation of the 2001–2004 Investment Project of Lithuanian Roads (Gravel Roads) (hereafter called the Project) according to which being paved 189 sections of gravel roads, the total length of which is 634 kilometres, will be completed in 2004. The estimate of this Project is 97 million euros. 75% of the total value of this Project is co-financed by the Nordic Investment Bank. It is planned to pave more than 2,200 kilometres of gravel roads and to improve the communication between administrative centres, settlements, to reduce accident rates as well as the need and costs to maintain and repair regional roads until 2015. However, the solution of problems posed by road users, when gravel roads are paved, shall be long-standing. The experience of implementing the Program and Project shows that one of the most important tasks in the future is to increase the dependability of the structures of low-volume roads, which is related to a proper evaluation of structures of paved gravel roads

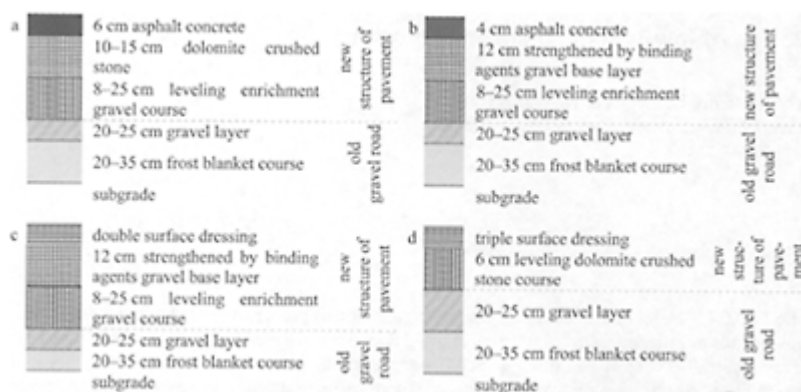


Figure 1. Low-volume road structures recommended in settlements (a, b) and between settlements (c, d).

when planning gravel road strengthening techniques. So far too little attention has been paid to the design of low-volume road structures in Lithuania.

At present, the structures of non-rigid road pavement (including low-volume roads and gravel roads) are selected according to the Regulation of Motor Roads in Lithuania (Regulation of Motor Roads 2001). The selected road pavement structures frequently do not conform to the real conditions (vehicle loads, climatic factors and etc.). Hence, the application of typical, not verified through calculations, pavement structures in road construction, reconstruction and repair is not substantiated. The application of typical road pavement structures is practically impossible when widening the carriageway of the road or strengthening road pavement by additional layers.

The consequences of inappropriate road pavement structure selection or design are defects and plastic deformations in road pavement. At present, various design methods of non-rigid road pavement structures are used in the world. The California Bearing Ratio (CBR) Method, Chvim Method, Triaxle Compression Method, Grouped Index Method,

Dawn's Method, etc. are used (Cost 333 1999, Cygas 2003). DORNII (The Method's name is from the Russian abbreviation) method was selected to design non-rigid pavement structures under Lithuanian climatic conditions (Cygas 2003, International Road Standard 2002). Having carried out theoretical and experimental investigations of this method and corrected the main data bases, this method was successfully used when designing road non-rigid pavement structures in Lithuania. In 2003, this method was used when designing low-volume road structures as well.

2 STRUCTURES OF LOW-VOLUME ROADS

Four types of road pavement structures were recommended for gravel roads being paved according to the Program. According to the recommended pavement structures, 6 cm asphalt layer may be performed on the 20–25 cm thick gravel layer and 20–35 cm thick road base (frost blanket course), by strengthening the old gravel road structure with the 8–25 cm thick levelling course and by setting up 10–15 cm thick base course of dolomite crushed stone or 12 cm thick base course of gravel, strengthened with binders, in addition. There were no requirements concerning strength characteristics for these structures. When implementing the Project, it was recommended to use standardized pavement structures besides structures used before. 25 cm base course of gravel or 20–22 cm thick dolomite crushed stone course, or 15–18 cm thick cement or stabilized base course may be laid in these structures on the old gravel road (Fig. 1) (Regulation of Motor Roads 2001).

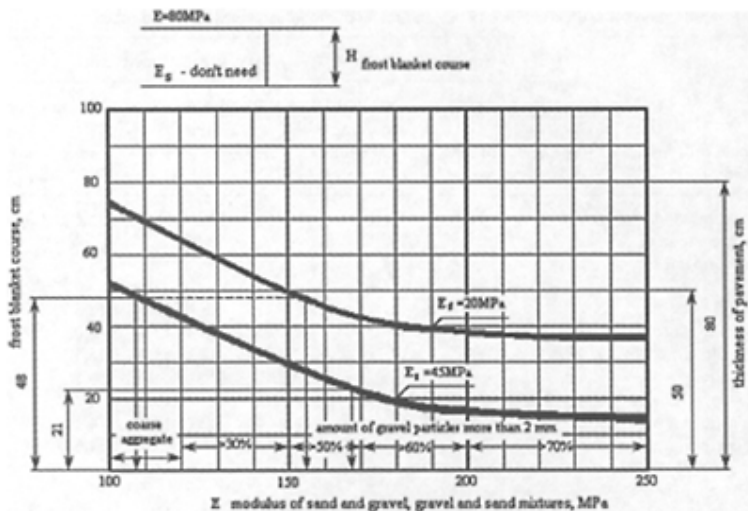


Figure 2. Identification graph of the thickness of a frost blanket course of various strengths of a subgrade soil (5).

The strength of the structures of low-volume roads is influenced not only by the thickness of the layers of the old gravel road structure, but also by the strength of the subgrade as well as qualitative characteristics of gravel (grading, quantity of fines in gravel) (Fig. 2). If characteristics of subgrade soil and pavement as well as gravel in the pavement are known, which are important when designing the structures of low-volume roads, their dependability may be guaranteed. A necessity to design gravel road being paved pavement structures may be substantiated by our gravel road research.

3 CHARACTERISTICS OF THE SUBGRADE AND SOIL OF MAINTAINED GRAVEL ROADS

A variety of the subgrade soils of maintained gravel roads is influenced by the road subgrade formation since the subgrade of gravel roads, the horizontal alignment of which did not change considerably under maintenance them, was formed from the local soil frequently mixed with the topsoil in the first stage. These and later activities related to digging water removal ditches, deepening, lifting and widening the subgrade influenced on the variety of subgrade types and their heterogeneity (Zilioniene et al. 2003).

The investigation of seven road sections showed that various groups of soils of different strength and humidity are found in their subgrades (Table 1).

Soil of highly silty sand is found in the subgrade of all roads. Soils of this group and low plastic clay soils, which are found in the subgrades rather frequently, are characterized by a high resilient modulus variation (variation coefficient values are 43.9 and 48.8% respectively). This variation of the resilient modulus is very important since soils mostly used for the subgrade are characterized by comparatively low resilient modulus. The data presented in the Table 2 show difference of gravel road subgrade soils in their physical and mechanical characteristics. These characteristics of subgrade soils shall be taken into account when designing low-volume roads of sufficient strength, effective, suitable for traffic and dependable.

Table 1. Subgrade soils, their characteristics and resilient modulus values on tested roads.

Subgrade soils in the test roads	Soil groups according to the Lithuanian Standards	Soils frequency in %		Statistical characteristics of soils						
		In the test roads	Calculated according to the total number of samples	Resilient modulus (E_v), MPa			Moisture content, %			
				E_v	S	C_v , (%)	W	S	C_v	
Re-digging sand, fine aggregate sand, coarse	S	86	16	85	24	28.2	5.6	1.3	23.2	

aggregate sand									
Silty sand	SD	57	15	75	18	24.0	12.3	3.9	31.7
Highly silty sand	SD ₀	100	35	57	25	43.9	13.0	2.9	22.3
Gravel, silty gravel	Z, ZD	28	12	87	42	48.3	5.5	1.0	18.2
Highly silty gravel	ZD ₀	28	3	40	–	–	15.4	–	–
Highly clayey gravel	ZM ₀								
Low plastic clay	ML	71	17	41	20	48.8	14.1	3.6	25.5
Low plastic silt	DL	14	2	32	–	–	14.6	–	–

Gravel road research data and their statistical analysis show that gravel road structure thickness varies a lot (variation coefficient values, calculated for frost blanket course, vary from 24.7 to 101.9%). The course particles in gravel road structures vary a lot; hence, its strength, influenced by the content of fines in gravel, varies as well. These and other data on different types of soil presented above show that a complex solution of the dependability of structures of low-volume roads is possible if gravel road structure design methods are used.

4 DESIGN OF STRUCTURES OF LOW-VOLUME ROADS

The following four stages of designing the paving, repair and upgrade of gravel road structure are pointed out:

- initial data collection, analysis and assessment,
- initial pavement structure selection,
- design of pavement structure when solving the issues of setting up and dependability,
- service (life) cycle analysis.

The structure of the low-volume is mostly set up through the use of the old gravel road structure. The subgrade in the old gravel road is not replaced; therefore, the dependability of the design structure depends on the amount and the dependability of collected data on gravel road structure and traffic. In Lithuania, types of pavement are selected and constructed by taking into account heavy vehicle flow, i.e. according to the heavy-vehicle traffic's index (Regulation of Motor Roads 2001) and are designed according to the following scheme of calculation (Fig. 3). The structure of low-volume road is usually designed with a certain degree of dependability. Since gravel roads mostly belong to low-volume road technical categories IV or V; a sufficient degree of dependability is $p=0.95$.

Values of other coefficients shall also be set (stiffness according to resilient modulus, strength to shear and tensile strength by bending); on which the optimal designed pavement structure depends. First of all, optimal pavement structure shall comply with the following conditions (Batrakov 1985):

- Pavement structure shall be rather rigid and dependable, which may be guaranteed by the occurrence of elastic deflection, under the influence of car wheel loads, which is expressed as

Table 2. Statistical characteristics of the main parameters of gravel roads maintained by Lithuanian regional enterprises (5, 8).

Parameters	Statistical characteristics	Statistical characteristics for gravel roads in regions									
		Alytus	Kaunas	Klaipėda	Marijampolė	Panevėžys	Siauliai	Taurage	Telsiai	Utena	Vilnius
Road width, m	$\bar{x}$	8.20	8.32	7.43	7.85	8.45	8.38	7.90	7.35	7.91	8.6
	S	0.94	1.02	1.36	1.35	1.26	1.37	1.40	1.26	1.10	1.15
	Cv, %	11.4	12.3	18.4	17.2	15.0	16.4	17.6	17.1	13.9	13.4
Thickness of gravel road, cm	$\bar{x}$	8.3	19.3	35.7	19.2	34.0	19.2	19.8	19.0	19.5	19.1
	S	1.7	2.3	12.0	2.2	13.6	2.5	1.3	4.0	2.7	1.5
	Cv, %	9.3	11.8	33.6	11.5	40.5	12.9	6.8	21.2	13.9	7.9
Frost blanket course cm	$\bar{x}$	31.6	19.7	38.2	17.1	35.4	19.5	10.0	17.6	18.3	34.0
	S	10.5	13.3	12.4	13.5	12.9	15.9	7.8	17.9	12.6	8.4
	Cv, %	3.2	67.5	32.3	78.9	35.8	81.7	77.9	101.9	68.8	24.7
Cross fall, ‰	$\bar{x}$	36.0	34.6	34.9	28.0	32.0	33.6	37.4	31.3	26.4	36.0
	S	15.0	15.5	15.3	15.4	14.8	15.6	16.4	14.2	16.3	14.0
	Cv, %	41.7	45.0	44.0	54.8	46.4	46.5	43.8	45.4	61.7	38.9
Coarse particles in gravel road, %	$\bar{x}$	37.0	28.5	31.3	32.4	35.0	31.0	33.2	24.6	28.4	33.0
	S	9.5	7.8	7.8	8.3	10.8	12.2	7.6	8.9	8.2	8.0
	Cv, %	25.7	27.4	24.9	25.7	31.0	39.4	22.9	36.2	29.0	24.2
Fines in	$\bar{x}$	5.6	4.8	4.5	4.2	4.3	5.9	4.2	8.3	4.6	6.1

gravel, %											
S	1.5	1.7	1.1	1.5	1.0	3.2	1.2	2.8	2.4	1.5	
Cv, %	26.8	35.9	25.1	36.6	22.8	53.8	29.7	34.2	52.3	24.6	

The diagram illustrates the vertical structure of a road. It consists of three distinct layers, each represented by a different hatching pattern. The top layer is labeled 'surfacing and strengthened road foundation' and features a diagonal hatching pattern sloping from the top-left to the bottom-right. The middle layer is labeled 'old gravel road' and features a vertical hatching pattern. The bottom layer is labeled 'subgrade' and features a diagonal hatching pattern sloping from the top-right to the bottom-left. A vertical line on the right side of the diagram, with arrows at both ends, indicates the vertical extent of these layers.

Figure 3. Scheme of design of the structures of low-volume roads.

pavement strength coefficient (K_{st}):

$$K_{st} = \frac{E_{total}}{E_{need}} \quad (1)$$

where K_{st} =pavement structure strength coefficient, identified taking the required degree of dependability into account; E_{total} =pavement structure (all layers) general resilient modulus; E_{need} =required pavement structure resilient modulus, identified taking the type of pavement, funds and work costs to set it up as well as traffic into account.

When calculating the general pavement structure resilient modulus, the thickness of all pavement structure layers (H_d) is taken into account:

$$H_d = \sum_{i=1}^n H_i \quad (2)$$

where H_i =i layer thickness; n =number of layers.

–Under the influence of various duration static and dynamic loads on the structural courses of low-volume roads residual deformation of shear shall not occur. Coefficient (k_{sh}) of strength to shear shall be as follows:

$$k_{sh} = \frac{[\tau]}{\tau_a} \quad (3)$$

where $[\tau]$ =permitted shear deflections described by material or soil cohesion characteristics; τ_a =active shear stress occurring due to an acting load.

–Low-volume road layers shall not crack, i.e. there shall not be any indications of material deterioration, i.e. low-volume road asphalt concrete pavement shall be resistant to tension by bending. Pavement resistance to tension by bending is described by asphalt concrete pavement layers' tensile strength by bending coefficient (K_b):

$$K_b \leq \frac{[\sigma_b]}{\sigma} \quad (4)$$

where $[\sigma_b]$ =asphalt concrete pavement layer material permitted tensile tensions, determined taking pavement fatigue phenomena into account; σ =maximum tensile tensions in the layer under investigation.

Moreover, the designed low-volume road structure shall be resistant to the impact of frost. Frost-resistance depends on the relevant pavement structure thickness.

When designing the structures of low-volume roads, values of coefficients specified in formulae 1, 3 and 4 are accepted higher than the degree of dependability, i.e. strength coefficient is ≥ 0.92 , coefficient of shear strength is ≥ 0.90 and tensile strength by bending is ≥ 0.95 .

The required resilient modulus for low-volume road is calculated according to the following formula:

$$E_{need} = \bar{E}(1 + tV_E) \quad (5)$$

where E =standard value of modulus; V_p =modulus variation coefficient; t =standard modulus deviation coefficient at the degree of dependability set during designing.

Permitted shear tensions in the frost blanket course, which makes up the old gravel road, is calculated according to the following formula:

$$[\tau] = Ck_1k_2k_3 \quad (6)$$

where C =cohesion during the design period, MPa; k_1 =coefficient assessing short-term dangerous load recurrence frequency (it is usually accepted as 0.6); k_2 =resource coefficient due to the structure operation conditions heterogeneity, insufficient assessment of climatic conditions, etc. ($k_2=0.96$); k_3 =coefficient assessing the increased adhesion due to reducing fraction and cohesion of materials ($k_3=6.6$).

Cohesion in the subgrade soil (C_s) is calculated as follows:

$$C_s = \bar{C}(1 - tV_C) \quad (7)$$

where C =standard soil cohesion value; V_C =cohesion variation coefficient ($V_C=0.1$); t =standard cohesion deviation coefficient ($t=1.32$).

Cohesion degree in soil mostly depends on humidity. The estimated soil humidity (W_{sk}) is calculated as follows:

$$W_{sk} = \bar{W}(1 + tV_W) \quad (8)$$

where W =average, of several years maximum soil, contained in the active part of the subgrade, humidity values; V_w =soil humidity variation coefficient value ($V_w=0.1$); t =standard humid deviation coefficient ($t=1.32$).

When designing asphalt concrete pavement structures for the old gravel roads, the elasticity theory is taken into account (Croney and Croney, 1997):

- the pavement being designed is homogeneous;
- there is a linear connection between tensions and deformations, i.e. Hooke's law is applied. If coefficients in equations, defining the dependence of tensions and

deformations, are stable, such pavement is called homogeneous. If they are unstable, they are heterogeneous;

– relative deformations are too low.

Dependability of designed structures for low-volume roads depends on their strength, which is mostly influenced by the old structure of the pavement (its thickness, gravel qualitative characteristics). When designing the structures of low-volume roads, a certain methodology shall be applied, taking all factors influencing the strength and dependability of pavement structures into account. The constructed and maintained pavement structure shall be rather stable due to impacting loads and durable inspire of the impact of various climatic factors. The scheme of constructing low-volume road methodology is presented in Figure 4. Visual observation of gravel roads during frost season in spring is necessary to identify the places where heaving occurs.

When solving gravel road structure strengthening tasks during asphalt paving works, it was noticed that soils under the pavement structure couldn't be changed. However, it is important to reduce the humidity of the subgrade soil through various maintenance and technical measures. When the thickness of pavement and frost blanket course is stable, the thickness of pavement varies a lot due to the increased humidity of soil. The increased soil humidity during different

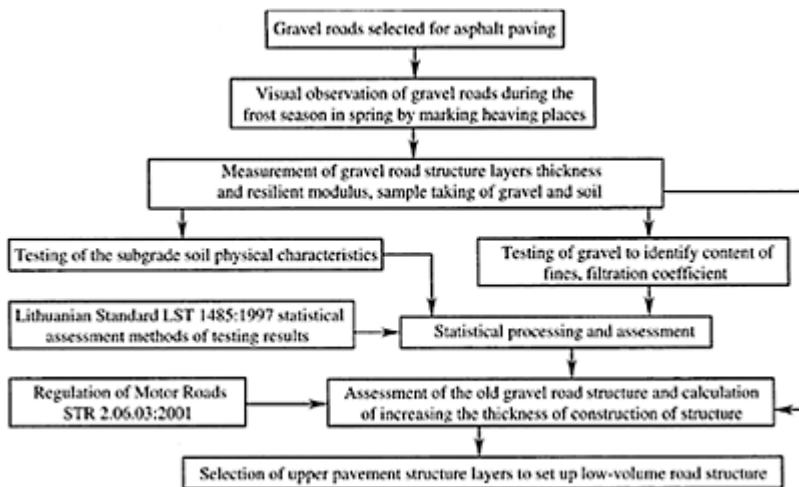


Figure 4. The scheme of the methodology of pavement construction of low-volume roads.

Table 3. Values of the lowest pavement resilient modulus for Lithuanian low-volume roads.

Road category	Total number of impact of ES A (European Standard Axes Load)	Project annual average daily	Required minimal pavement resilient

	during maintenance period of pavement, ΣN_p , unit per day	traffic, N, vehicles per day	modulus, E_{min} , MPa
IV	632.103	500–1500	265
V	390.103	<500	200

times of the year is related to local hydrological conditions and the condition of the water removal system on a particular road.

Quarry mixture of gravel and sand is mostly used to set up base, frost blanket course on gravel roads. A major part (particles size larger than 5 mm) of gravel-sand mixture, which in most cases is the main structural material of pavement, shall be rather resistant and shall bear not only the impact of various forces for preparing optimal mixtures, but the impact of road construction machinery and vehicles as well. The thickness of the old gravel roads shall be increased so that on the top of it the minimum required resilient modulus is achieved.

To ensure compliance of the designed gravel road strength characteristics with the current Lithuanian road conditions (vehicle traffic volumes and loads to pavement), the Road Department at Vilnius Gediminas Technical University determined minimum pavement structure resilient modulus for roads of state significance. Minimal resilient modulus for pavement structures of low-volume road technical categories IV and V are presented in Table 3. Typical pavement structures (Table 4, Fig. 1) were calculated through the use of the gravel road design methodology (the DORNII method was used when designing non-rigid pavement in the former USSR, does not meet the changed conditions of setting up and maintaining the Lithuanian road pavement, and it was improved considerably) presented in this article (Fig. 1), by applying requirements presented in Table 2 (required $E_{min}=200$ MPa) (Cygis 2003).

Table 4. Results of low-volume road structures of V road category calculated with program packet “Nonrigid Road Pavement” based on DORNII method.

Pavement structure	Layer No.	Code of material in program data base	Material	Layer Thickness (cm)	Resilient Modulus, (MPa)	Note
Fig.1 (a)	1	23	asphalt concrete 0/1 6V, B 70/100	6	192	$E_{total}=192$ MPa inadequate
	2	55	dolomite crushed stone	15	153	
	3	71	enrichment gravel	25	108	
	4	72	gravel layer	25	104	

Fig.1 (a)	5	174	coarse aggregate sand	35	103	
	6	175	silty sand (subgrade)			
	1	23	asphalt concrete 0/1 6V, B 70/100	6	200	$E_{total}=200$ MPa Thickening of
	2	55	dolomite crushed stone	17	160	dolomite
	3	71	enrichment gravel	25	108	crushed stone
	4	72	gravel layer	25	104	layer (17 cm)
Fig.1 (b)	5	174	coarse aggregate sand	35	103	adequate
	6	175	silty sand (subgrade)			
	1	23	asphalt concrete 0/1 6V, B 70/100	4	192	$E_{total}=192$ MPa inadequate
	2	74	strength, by binding agents gravel	12	177	
	3	71	enrichment gravel	25	108	
	4	72	gravel layer	25	104	
Fig.1 (b)	5	174	coarse aggregate sand	35	103	
	6	175	silty sand (subgrade)			
	1	23	asphalt concrete 0/1 6V, B 70/100	4	201	$E_{total}=201$ MPa Thickening of
	2	74	strength, by binding agents gravel	13	186	strength, by binding agents
	3	71	enrichment gravel	25	108	gravel base layer
	4	72	gravel layer	25	104	(13 cm)

Fig.1 (c)	5	174	coarse aggregate sand	35	103	adequate
	6	175	silty sand (subgrade)			
	1	52	double surface dressing	4	185	$E_{total}=185\text{ MPa}$
	2	74	strength, by binding agents gravel	12	177	inadequate
	3	71	strength.by binding agents gravel	25	108	
	4	72	gravel	25	104	
Fig.1 (d)	5	174	coarse aggregate sand	35	103	
	6	175	silty sand (subgrade)			
	1	52	triple surface dressing	6	131	$E_{total}=131\text{ MPa}$
	2	55	dolomite crushed stone	6	115	inadequate
	3	72	gravel	25	104	
	4	174	coarse aggregate sand	35	103	
	5	175	silty sand (subgrade)			

Design results showed that the thickness of separate layers of the typical pavement structure mostly depends on the materials used to set up separate layers and the subgrade, and the thickness of typical structure layers does not always guarantee the required minimal elasticity modulus. Therefore, in each specific case it is important to evaluate the current and future road pavement operation conditions, used materials, heavy vehicle traffic loads as precisely as possible and to design road pavement structures. The use of typical pavement structures shall be extremely limited. Setting up pavement structures based on calculations will enable to use expensive road construction materials economically and to ensure the required strength of pavement structures in each specific case.

5 CONCLUSIONS

Lithuanian gravel road asphalt paving works require immediate solution of tasks related to increasing the stability and dependability of the structures of low-volume roads, which are closely related to proper assessment of the old gravel road structures (subgrade, road foundation and surfacing) when planning the methods of their strengthening.

Due to the heterogeneity of physical and mechanical characteristics of medium frost-susceptible soils (silty sand, low plasticity clay) prevailing in the subgrade of gravel roads being asphalt paved and wide variation of gravel qualitative characteristics in gravel road, the use of typical pavement structures in different Lithuanian regions is not ensure the required stiffness and stiffness homogeneity as well as long-lasting structures of low-volume roads.

Design of low-volume road shall be carried out in stages when sufficient data are collected according to the pavement construction scheme proposed by us, which is based on visual observations and special investigations as well as research data analysis.

The application of the DORNII method when designing low-volume road enables to safeguard sufficient strength and guaranty their structures suitability to local conditions by calculations, to use expensive road construction materials more economically as well as to ensure the required stiffness of pavement structures in each specific case.

REFERENCES

- Batrakov, O.T. 1985. *Strengthening of Non-rigid Road Pavement* (in Russian) Moscow: Transport. Cost 333. 1999. *Development of New Bituminous Pavement Design Method*. Final Report of the Action. European Commission Directorate General Transport. Brussels.
- Croney D. & Croney, P. 1997. *Design and Performance of Road Pavement*. NY.: McGraw-Hill.
- Cygas, D. 2003. Designing Problems of Non-rigid Road Pavement Structures in Lithuania. *25th International Baltic Road Conference and Exhibition*.
<http://www.balticroads.org/conference/en/seminars.html#>. Accessed December 10, 2003.
- Dundulis, K., Gadeikis, S., Juzenas, A.A. & Rackauskas, V. 2000. Analysis of Roadbed Soil Properties in Siauliai Region. *Litosfera*. Vilnius, No. 4 p.p. 93–99.
- International Road Standart MODN 2–2001. 2002. Design of Flexible Pavements. Sojuzdornii. Regulation of Motor Roads (STR 2.06.03:2001). 2001. (in Lithuanian). Ministry of Environmental of the Republic of Lithuania, Ministry of Transport and Communications of the Republic of Lithuania. Vilnius.
- Zilioniene, D. 2003. *Renovation of Gravel Roads from the Aspect of Sustainable Development of Road Network*. Summary of Doctoral Thesis. Vilnius: Technica.
- Zilioniene, D., Cygas, D. & Dundulis K. 2003. Solutions of Gravel Road Renovation Based on Certain Local Conditions in Lithuania. *Transportation Research Record* 1819(2): 267–274. Washington, D.C.
- Zilioniene, D., Juzenas, A.A. & Laurinavicius, A. 2003. Experience to Improve and Maintain Gravel Roads in Lithuania. *25th International Baltic Road Conference and Exhibition*,
<http://www.balticroads.org/conference/en/seminars.html#>. Accessed December 10, 2003.

Mechanistic-empirical design models for pavement subgrades

H.L.Theyse

CSIR Transported Pretoria, South Africa

Pavements Unbound—Dawson (ed.)

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ABSTRACT: The vertical subgrade strain criterion is probably one of the best known and internationally accepted mechanistic-empirical design models in current use. This model relates the imposed vertical strain at the top of the subgrade to the number of load cycles that can be sustained before a certain terminal condition will be reached. Recent research in South Africa has shown that the vertical strain at the top of the subgrade is in fact a poor predictor of subgrade permanent deformation and therefore even worse in terms of predicting the permanent deformation of the full pavement system. A sample of 35 HVS tests for which reasonable material, MDD deflection and MDD permanent deformation data were available, were used in the investigation of subgrade permanent deformation to calibrate permanent deformation subgrade design models. Total subgrade elastic deflection was found to be a better predictor of subgrade permanent deformation than the vertical subgrade strain or the vertical stress at the top of the subgrade as long as the subgrade is protected well enough to prevent high shear stress from occurring. A set of design models was calibrated for different levels of subgrade permanent deformation using the subgrade elastic deflection (SED) as the critical parameter.

1 INTRODUCTION

Mechanistic-empirical pavement design is well established in South Africa. Publications on the South African Mechanistic-Empirical Design Method (SAMDM) dates from the 1970s (van Vuuren et al, 1974) until recently (Theyse and Muthen, 2000). The current version of the SAMDM is, however, a terminal-condition, critical-layer design method with the pavement bearing capacity being determined by the layer having the lowest bearing capacity. This paper presents research that forms part of a long-term process to develop a complete permanent deformation design procedure for the SAMDM. The aim is to eventually convert the permanent deformation modelling procedure in the SAMDM

from a terminal-condition, critical-layer approach to a full pavement system, distress simulation approach where each of the pavement layers contribute to the total permanent deformation.

Different modelling strategies are adopted for different stress regimes in the pavement and the pavement structure is accordingly divided into three zones based on the influence of the externally applied load on the pavement. These zones are illustrated in Figure 1 for a typical South African pavement and include:

- The wearing course that requires a material with sufficient shear strength, crushing strength and abrasion resistance to withstand the high and irregular contact stresses (de Beer et al, 1997) imposed on the pavement. In South Africa, this layer is also the primary waterproofing layer on many paved roads;
- The structural pavement layers (base and subbase layers) that are subjected to high bulk and shear stress conditions and therefore require high crushing and shear strengths. These layers distribute the high input stress intensity and protect the pavement subgrade; and

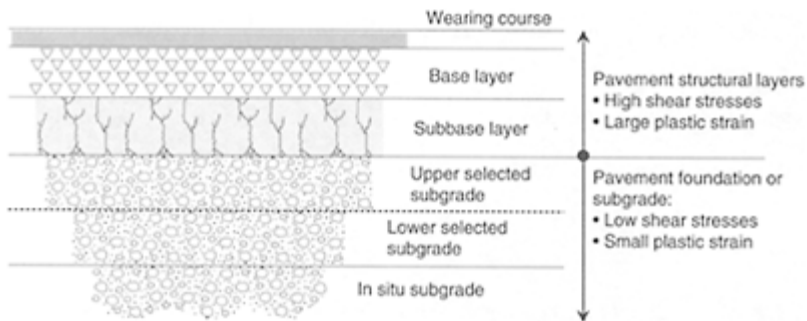


Figure 1. Stress regimes in a typical South African pavement.

- The pavement foundation or subgrade, consisting of the in situ and imported subgrade (when required). At this level, the shear stresses are supposed to be well dissipated for a sound pavement design and the relatively low quality subgrade material should operate at shear stresses well below the shear strength of the material for any given level of confinement.

The design procedure adopted for unbound structural layers was presented at UNBAR5 (Theyse, 2000) and is based on a stress ratio approach that relates the permanent deformation of the unbound material to the ratio between the applied shear stress and the shear strength of the material. The shear strength of the unbound material was also shown to be a function of the density and degree of saturation of the material. This paper presents the development of a design model for the pavement subgrade from Heavy Vehicle Simulator results whereas the design models for the structural layers were based on static and dynamic tri-axial test results.

2 BACKGROUND INFORMATION

The well known vertical strain criterion (Dorman and Metcalf, 1965) is included in the current SAMDM for subgrade design with some adjustments made for local conditions (Paterson and Maree, 1978). This criterion was, however, developed from the AASHO road test data and was based on the riding quality of the pavement rather than the permanent deformation of the subgrade. It was therefore decided to undertake a comprehensive study of permanent subgrade deformation in South Africa and to develop a permanent deformation design model for the subgrade.

The new pavement subgrade distress model was developed from a comprehensive Accelerated Pavement Testing (APT) database that was generated by a fleet of Heavy Vehicle Simulators (HVSs) over 20 years of testing in South Africa. The process focused on the development of a subgrade distress and design model using measured resilient and permanent deformation response variables as far as possible. This was done to eliminate potential errors that could be introduced from modelling inaccuracies.

The formulation of the conceptual permanent deformation model is the same as that used for the structural layers (Theyse 2000) and defines the permanent deformation (PD) as a function of two primary independent variables, a critical stress parameter (S) and the number of load repetitions (N). If one of these two primary independent variables is zero, there will be no traffic-induced permanent deformation in a pavement structure. The remaining independent variables are referred to as secondary independent variables and will not cause any traffic-related permanent deformation by themselves, but will control the rate of traffic-induced permanent deformation. If only the relationship between the dependent variable (permanent deformation) and the two primary independent variables (stress condition and the number of stress repetitions) is taken into

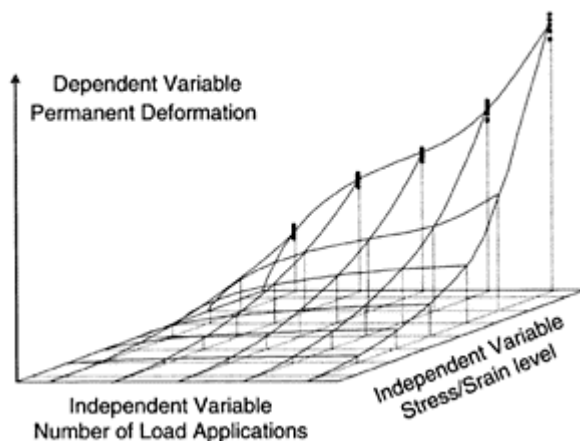


Figure 2. Conceptual permanent deformation model.

consideration, a three dimensional, non-linear model, such as the one illustrated in Figure 2, may be formulated. Since no traffic-induced permanent deformation is possible unless

both the primary independent variables are greater than zero, the 3-dimensional surface will have its origin along the two axes describing the horizontal plane in Figure 2. If the model is calibrated for the influence of all the secondary independent variables, a 3-dimensional surface, such as the one illustrated in Figure 2, will exist for each possible combination of all the secondary independent variables.

The data from 35 South African HVS tests, done on 10 sites and dating back to 1983 were used to calibrate the conceptual permanent subgrade deformation model. Table 1 provides a summary of all the HVS tests sites used in the study. The HVS sites listed in Table 1 were tested with trafficking half-axle loads and inflation pressures ranging from 40 to 150 kN and 520 to 1445 kPa respectively.

The main data source used in this study was the Multi-Depth Deflectometer (MDD) system that is used in conjunction with HVS testing. The MDD consists of a stack of modules, normally installed at the layer interfaces in the test pavement. The MDD allows the elastic depth deflection to be recorded as the wheel passes over the system. An example of a reduced set of depth deflection bowls is shown in Figure 3. The peak deflection may be extracted from each deflection bowl to obtain a depth deflection profile such as the example shown in Figure 4. The permanent displacement of the MDD modules under increasing number of load repetitions can also be recorded as shown by the example in Figure 5 (not according to scale).

3 ANALYSIS OF THE ELASTIC AND PERMANENT SUBGRADE DEFORMATION RESPONSE

The analysis of the MDD elastic response data consisted of a process of direct calculation of the vertical elastic subgrade strain, ϵ_v , and the subgrade elastic deflection, δ_s . The vertical elastic subgrade strain was calculated from the difference between the deflections of two successive MDD modules located in the subgrade, divided by the initial offset between the two modules and therefore represents the average vertical strain (slope of the deflection profile in Figure 4) between the two MDD modules. The subgrade elastic deflection is measured directly by the MDD modules installed in the subgrade. A back-calculation process was used in addition to the direct calculation procedure. During this back-calculation process, the measured MDD deflections were matched with a set of calculated deflections from a multi-layer, linear-elastic (MLLE) analysis package by adjusting the resilient modulus input values of the analysis package until the measured and calculated

Table 1. HVS sites used in the study of subgrade permanent deformation.

Region	Region description	Site	Predominant subgrade material
Eastern Cape coastal	Wet coastal region, Weinert N-value (Weinert, 1980) less than 2	Macleantown (East London) Port Elizabeth	Decomposed dolerite gravel Sandy limestone gravel

Central Cape Karoo	Semi-desert, Weinert N-value greater than 10	Richmond	Calcrete gravel
KwaZulu- Natal coastal	Sub-tropical coastal region, Weinert N- value less than 2	Umkomaas	Unknown
		Amanzimtoti	Decomposed dolerite gravel and Berea red sand
Gauteng highveld	Moderate highveld region, Weinert N- value between 2 and 10	Bapsfontein	Ferricrete selected subgrade on a weathered shale in situ subgrade
Gauteng highveld	Moderate highveld region, Weinert N- value between 2 and 10	Rooiwal	Ferricrete
		Bultfontein	Ferricrete
		Cullinan	Ferricrete selected subgrade on an existing gravel road wearing course Sandstone conglomerate selected subgrade on an existing gravel road wearing course Ferricrete selected subgrade on an existing gravel road wearing
Northern Province Bushveld	Hot, moderately wet bushveld region, Weinert N-value between 2 and 10	Louis Trichardt	Decomposed granite gravel

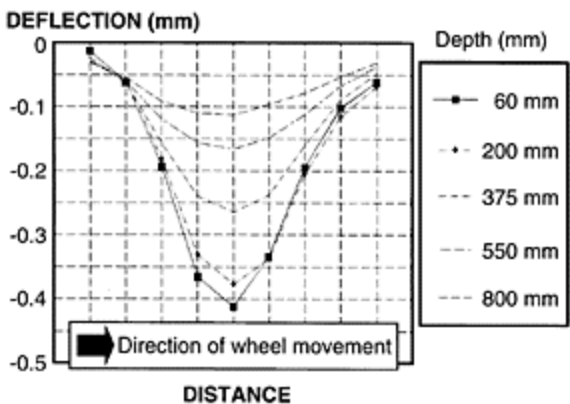


Figure 3. Reduced set of depth deflection bowls from the MDD system.

deflections agree within a certain tolerance. Once a set of effective resilient modulus values was obtained in this manner, the vertical stress, σ_s , applied to the top of the subgrade and the vertical elastic subgrade strain, ε_v , at the top of the subgrade were obtained from the output of the MLLE package.

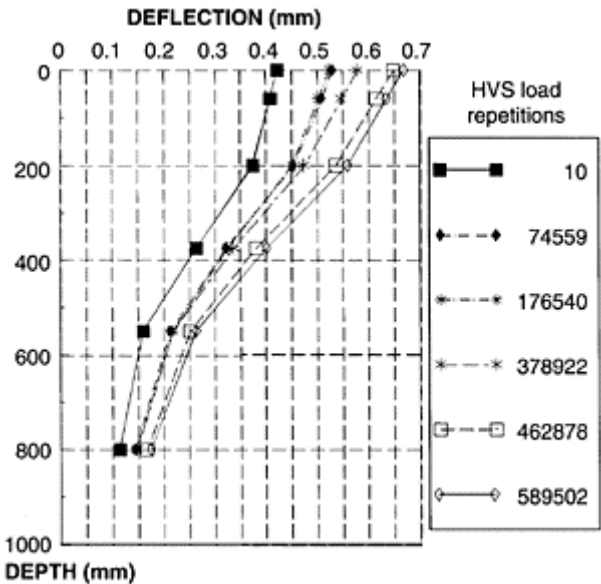


Figure 4. Depth deflection profile from the MDD system.

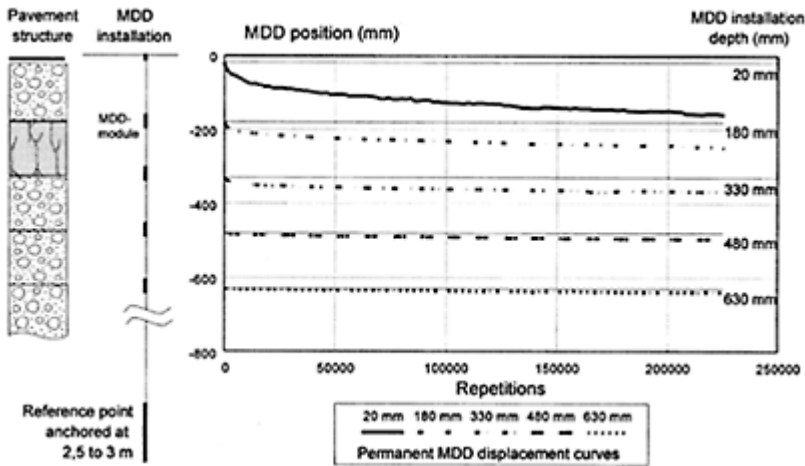


Figure 5. MDD permanent displacement traces.

The analysis of the permanent MDD displacement data consisted of fitting the function listed in Equation 1 to the permanent MDD displacement data of each MDD module in the subgrade of the HVS tests listed in Table 1 by non-linear regression analysis. The shape of the function in Equation 1 is illustrated in Figure 6 and two of the regression coefficients have a physical interpretation. These are the bedding-in parameter “a” and the eventual linear rate of permanent MDD displacement “m”. It should be noted that the function listed in Equation 1 does not allow

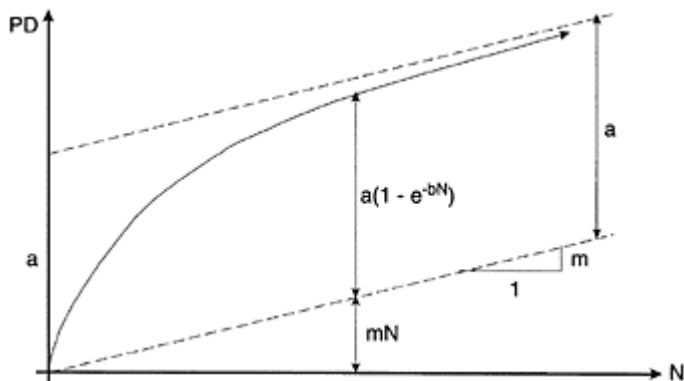


Figure 6. Non-linear regression model fitted to the MDD permanent displacement data.

for shakedown. Shakedown has not been observed for pavements tested with the HVS up to almost 4 million load repetitions. It has been observed that the plastic deformation rather settles into a linear rate than an ever decreasing rate as “shakedown” would require.

$$PD = mN + a(1 - e^{-bN}) \quad (1)$$

Where PD=Permanent subgrade deformation (mm); N=Number of load repetitions; a=Bedding in parameter (mm); m=Linear rate of displacement (mm/repetition).

Each MDD module installed on the HVS tests therefore has a set of “critical parameters” (ϵ_v , δ_s and σ_s) and permanent displacement parameters (a, m and b) associated with it. The most appropriate critical parameter was selected based upon considering the following:

- The critical parameter should distinguish between different load cases in terms of wheel load and contact stress.
- The critical parameter should distinguish between different amounts of protection provided to the subgrade in terms of the thickness and quality of the layers covering the subgrade.
- The critical parameter should be able to accommodate subgrade materials of different quality.

- The critical parameter should be able to accommodate new and rehabilitation design cases.
- Lastly, and most importantly, given the above characteristics, there should be a good correlation between the critical parameter and the permanent subgrade deformation.

The vertical stress applied to the top of the subgrade is largely determined by the load and contact stress applied on the surface of the pavement, and by the load-spreading ability of the cover provided to the subgrade but does not incorporate the response of the subgrade and therefore the effect of the quality of the subgrade material. The vertical subgrade strain, either at the top of the subgrade or the average value over a portion of the subgrade, reflects the partial response of the subgrade material to the applied stress and therefore incorporates the effect of the subgrade material quality to a limited extent. However, as the vertical strain only represents the response at the top of the subgrade or for a relatively small portion (layer) of the subgrade and not the total subgrade response, this parameter is largely influenced by the quality of the material at the top of the subgrade. The vertical subgrade strain therefore does not distinguish well between different load cases, with some of the lower load cases that were investigated causing more strain than the extremely high load of 150 kN. The subgrade elastic deflection on the other hand, distinguishes between different amounts of cover and different loading conditions and incorporates the total subgrade response represented by the integral of the subgrade strain from the top of the subgrade to a depth at which the applied load has no more effect. The quality of the material in the full depth

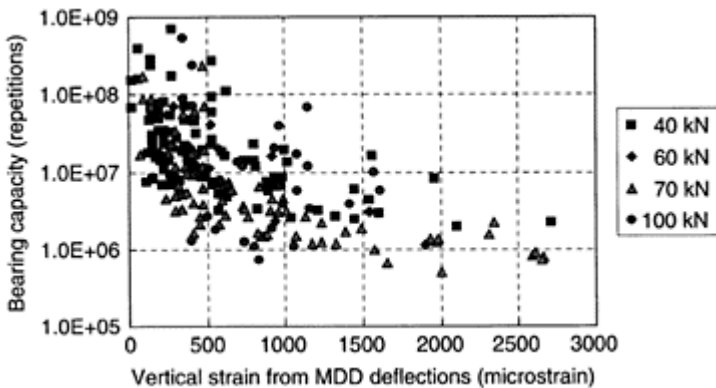


Figure 7. S-N plot for subgrade vertical strain.

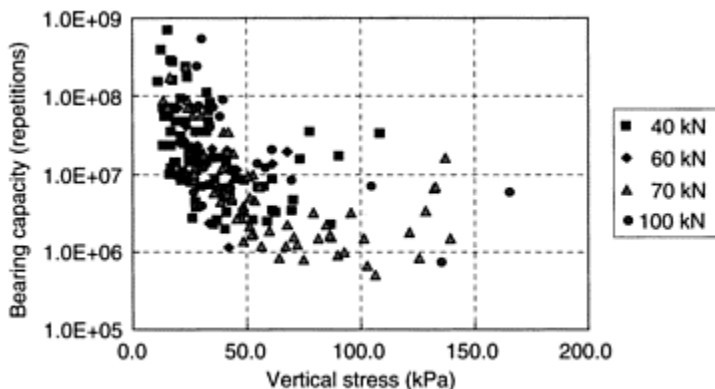


Figure 8. S-N plot for subgrade vertical stress.

of the subgrade and not only in the top portion of the subgrade is therefore incorporated in this critical parameter.

By studying the combined elastic and plastic response parameters and investigating the correlation between the number of load repetitions that can be sustained before a certain level of vertical permanent subgrade deformation is reached and each of the critical parameters, the most appropriate critical parameter was selected. The number of load repetitions to reach a certain level of subgrade permanent deformation was obtained by solving the function listed in Equation 1 for different values of subgrade permanent deformation (given the values of the regression coefficients for each subgrade MDD module).

The subgrade elastic deflection was found to have the best correlation with the number of repetitions to reach a certain level of subgrade plastic deformation. Figures 7, 8 and 9 shows the number of load repetitions (N , bearing capacity) to reach a permanent subgrade deformation of 7 mm plotted against each of the three candidate critical parameters (S), subgrade vertical strain, subgrade vertical stress and subgrade elastic deflection.

The data for the 150 kN load cases had to be omitted from these plots and the subsequent calibration of the subgrade permanent deformation model. A study of the elastic and permanent deformation response of the HVS test sections subjected to the 150 kN single-wheel loads indicated that these loads imposed excessively high shear stresses on the subgrade and the relationship between

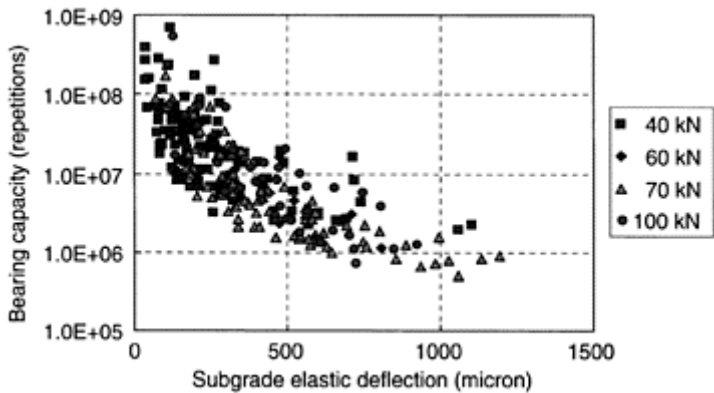


Figure 9. S-N plot for subgrade elastic deflection.

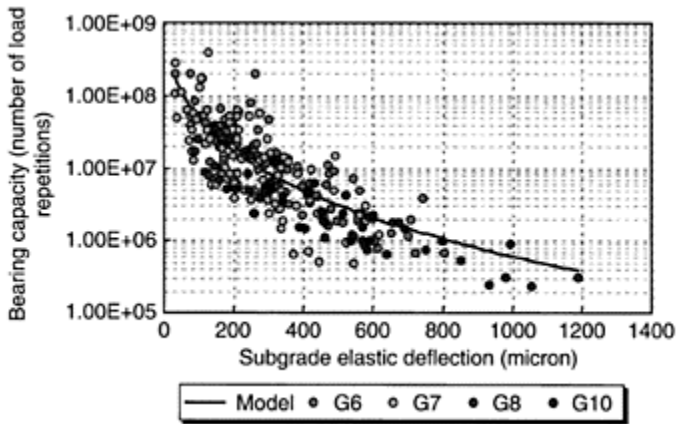


Figure 10. Regression model for 5 mm permanent subgrade deformation for a range of subgrade types.

the subgrade elastic deflection and subgrade permanent deformation was no longer valid under these conditions.

4 PERMANENT SUBGRADE DEFORMATION MODEL

A function of the type shown in Equation 2 was fitted to S-N data similar to the data shown in Figure 9 for 3, 5, 7, 13 and 19 mm permanent subgrade deformation. Figure 10 shows the fit for 5 mm permanent subgrade deformation for a range of subgrade materials.

$$\log N = C - \left(\frac{\delta_s}{a} \right)^b \quad (2)$$

Where N =Number of load repetitions; C , a and b =Regression coefficients; δ_s =Subgrade elastic deflection.

Figure 11 shows the final design model for all levels of permanent subgrade deformation. The lines in Figure 11 represent contour lines on the 3-dimensional conceptual model formulated in Section 2 and shown in Figure 2 when the model is viewed from the top.

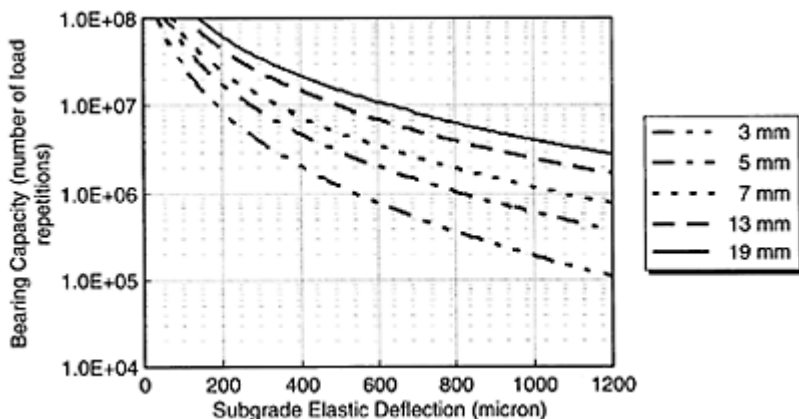


Figure 11. Permanent subgrade deformation design model.

5 CONCLUSIONS AND RECOMMENDATIONS

A conceptual empirical model was formulated and calibrated for subgrade permanent deformation using data from HVS tests done over a period of twenty years on different pavement types and subgrade conditions and in different environmental regions. The model appears to be valid for a range of different subgrade material classes.

A reasonable correlation was found to exist between the subgrade elastic deflection and subgrade permanent deformation as long as the shear stresses imposed on the subgrade are sufficiently dissipated by the structural pavement layers. The calibrated subgrade design model is therefore only valid for truck traffic on road pavements. Airfield design will require special consideration of the shear stresses imposed on the subgrade in relation to the shear strength of the subgrade material. The subgrade permanent deformation correlated poorly with the subgrade vertical strain.

Further research will investigate the effect of incorporating the distress model in the South African Mechanistic-Empirical Design Method using modelled response parameters and the impact of this on typical South African pavement designs.

REFERENCES

- Van Vuuren D J, Otte E and Paterson W D O, 1974. "The Structural Design of Flexible Pavements in South Africa". *Proc. 2nd Conference on Asphalt Pavements in South Africa*, Durban.
- Theyse H L and Muthen M. 2000. "Pavement Analysis and Design Software (PADS) based on the South African Mechanistic Design Method". *Proc. South African Transport Conference*, Pretoria.
- Theyse, H L 2000. The Development of Mechanistic-Empirical Permanent Deformation Design Models for Unbound Pavement Materials from Laboratory and Accelerated Pavement Test Data. In *Unbound Aggregates in Roads (Proc. 5th International Symposium 'UNBAR5')*, ed. Dawson, A.R., Balkema, 2000, pp. 285–293.
- De Beer M, Fisher C and Jooste F J, 1997. Determination of pneumatic tyre/pavement interface contact stresses under moving loads and some effects on pavements with thin asphalt surfacing layers. *Proc. 8th International Conference on Asphalt Pavements*, Seattle (p. 179).
- Dormon G M and Metcalf C T, 1965. Design curves for flexible pavements based on layered system theory. *Highway Research Record*, Number 71, Flexible Pavement Design, Highway Research Board, Washington, D.C., pp. 69–84.
- Paterson W D O and Maree J H, 1978. *An interim mechanistic procedure for the structural design of asphalt pavements*. National Institute for Transport and Road Research, CSIR, Pretoria. (Technical Report RP/5/78).
- Weinert H H, 1980. *The Natural Road Construction Materials of South Africa*. Cape Town: H & R Academica.

A timber piled road over deep peat in North West Ireland

T.Ryan & C.McGill

Coillte Teoranta, Galway, Ireland

P.Quigley

National University of Ireland, Galway, Ireland

Pavements Unbound—Dawson (ed.)

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ABSTRACT: A timber piled road was constructed over deep deposits of very compressible peat to access a small plantation in North Mayo. Attempts to construct the road using traditional methods were unsuccessful and the limited value of the crop meant that a cost-effective solution had to be used. This paper describes the site conditions, a description of the problems during the initial attempts to construct the road, details of the initial timber piled solution and subsequent alterations to the layout made during construction. Some observations from the successful completion of the road and recommendations for future piled roads are given.

1 INTRODUCTION

1.1 *The site*

Coillte Teoranta (The Irish Forestry Board) owns a 500 ha site at Corravokeen in North Co. Mayo which was planted between 1967 and 1974 (Fig. 1). Due to the presence of deep deposits of peat and high groundwater levels several sections of the site were unplanted or not developed. This has resulted in a 48 ha plantation being separated from the main plantation. Attempts to construct a

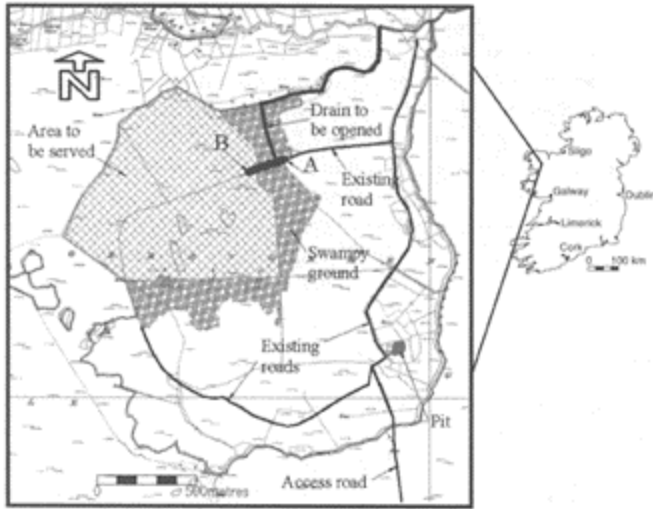


Figure 1. General location of the plantation and the piled forest road along AB.

120 m long road for the harvesting of the isolated plantation proved very difficult as the route passed over approximately 8 m of very compressible peat. The low value of the crop (€270,000) meant that a cost effective solution was required for the harvest to be viable.

1.2 Initial road construction

Typical Irish forest road formations are 5.5 m wide with a carriageway width of 3.4 m. The peat in the Corravokeen estate was excavated for the construction of forest roads once the depth was less than 2 m. For peat depths between 2 and 4 m a typical pavement thickness of 0.75 m was formed from a locally available sandy gravelly boulder clay placed over a closely spaced pole mat laid transversely across the road. Construction of roads over 4 m of peat is generally avoided.

Initial probing of the proposed route indicated that peat deposits between 4 m and 8 m deep were expected. Excavations at shallower locations exposed a well graded glacial till/boulder clay below the peat. The probing had also shown that no alternative route over shallower deposits was feasible so it was decided to construct a road along the shortest distance between the plantation and the existing roads. Parallel drains 10 m apart and 1 m deep were installed in accordance with general forest engineering practice to lower the groundwater and allow some consolidation of the peat to occur. The drains were installed 6 years before initial attempts to construct a road were made. A polypropylene biaxial geogrid Fortnit 40/40 (Huesker Synthetic GmbH) was laid over the peat and 0.75 m of well-graded sandy gravelly boulder clay was placed on top. The initial 15 m of the road was successfully completed but substantial deformation occurred when

the depth of peat started to exceed 3 m. A closely spaced pole mat was installed to stiffen the foundation and filling continued. A further 800 tonnes of fill was placed over 20 m long section. The mass of material used (40 tonnes per linear metre) indicated that the peat was undergoing both substantial settlement and also being displaced laterally on both sides of the road. Heaving of the ground surface at the edge of the road was evident. The completed sections of road were undergoing unacceptable distortions under construction traffic so it was decided to halt construction.

2 DESIGN SOLUTION

2.1 *Ground conditions*

The profile of the depth of peat along the route is shown in Figure 2. Typical moisture contents for peat ranged between 600–1200%. Uncorrected shear vane and piezocone tests carried out on similar peat at a nearby site showed shear strengths of 4–6 kPa.

The low value of the harvest meant that expensive options such as lightweight fill or two or more layers of geogrid reinforcing good quality imported fill were not viable. Transferring the timber from the plantation using a suspended cable system was considered instead of constructing a road but the layout of the plantation meant that this solution would also be prohibitively expensive. A more effective drainage scheme would require deepening the existing drains and reducing the spacing between the drains. The time required to satisfactorily drain the peat, the poor effectiveness of existing drains and potential problems with the silting of the drains meant Coillte were unwilling to use this option. It was decided to construct a piled road over the deepest deposits of peat.

2.2 *Piled forest roads*

Piled forest roads were rarely, if ever, considered in Ireland due to the high construction costs. The use of piled embankments has become popular in Ireland for major road projects (Quigley et al., 2003, O’Riordan, 1996). Despite the initial construction costs being comparatively expensive it is often the most appropriate solution for long term performance of embankments on soft soils. Barry et al. (1995) described the low cost use of native timber piles and two layers of geogrid to support a service road in Malaysia. However even this low cost solution would have been prohibitively

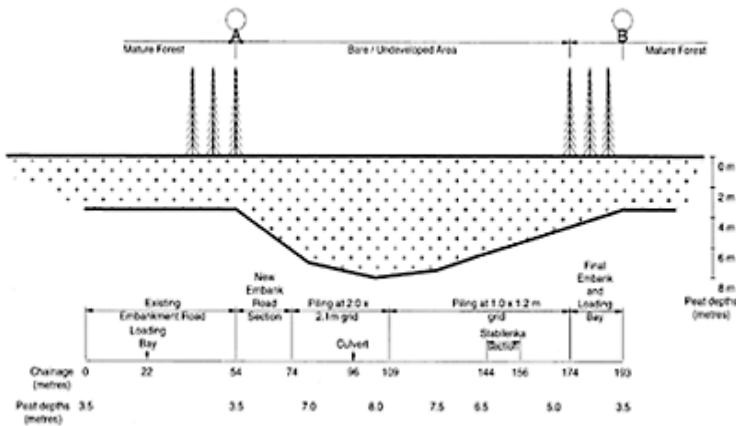


Figure 2. Long section showing the depth of peat and details of the road construction.

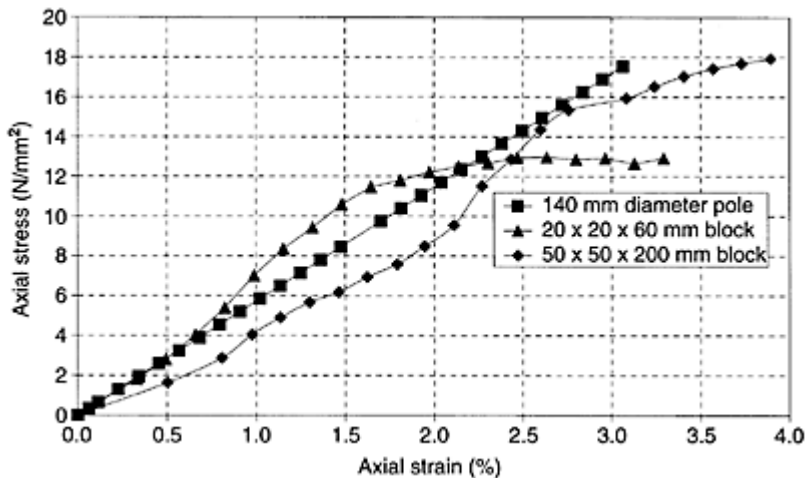


Figure 3. Compression tests carried out on pole and small blocks.

expensive for the proposed road. It was decided to design and construct a road at the ultimate limit state of the materials available in short 20 m sections and to alter the design layout if necessary.

To minimise the expenditure it was decided to attempt to maximise the spacing of the most expensive component of the road, the timber piles. A number of short, whole pole sections of Sitka spruce taken from nearby forests (140 to 150 mm in diameter and approximately 550 mm long) were tested at NUI, Galway using a Dension compression

machine. Figure 3 shows a good comparison between the results of compression tests carried out on small blocks carried out according to B.S. 373 (1957) and D 143–52 (1972) and the results on an uncut pole. Based on the results of the compression testing it was decided to using a design stress of 12 N/mm^2 for the Sitka spruce poles, which corresponds to pile load of 210 kN for a pole with a minimum diameter of 150 mm.

The saturated peat that surrounded most of the pile and also the short design life of the forest road would limit the degradation of the timber. Even if rotting of the timber occurs the pile should still offer enough support to the embankment for the duration of the harvesting over a two-year period, after which the road would be decommissioned.

It was assumed for design that the peat would support approximately 20% of the vertical load and the remainder of the load would be transferred to the pile by soil arching and the steel mesh. For the widest spacing ($2 \times 2.1 \text{ m}$ and an embankment height of 0.75 m above the piles) the total design dead load on the pile was 45 kN. The road was designed for an eight-wheel forest forwarder carrying a 10 tonne load with a corresponding maximum wheel load of 35 kN. The road was constructed using a 13 tonne excavator and a tractor and trailer. The weight of a fully laden tractor and trailer was 21 tonnes. Assuming 27% of the weight was supported by each of the trailer axles the maximum wheel load was 30 kN. It was decided that the construction traffic would be similar to the design loading and that the satisfactory performance of the newly constructed road by construction traffic would render the road safe to use by forest forwarders.

3 FIELD TRIALS AND CONSTRUCTION

3.1 *Initial section*

Locally harvested Sitka spruce poles, 9 m long with a tip diameter of 0.25 m, were installed through the peat using a 13 tonne excavator. Chains were wrapped around each pole to allow the poles to be lifted upright and easily dragged downwards through the peat. The chains were then removed and the pole was driven to refusal into the boulder clay using the bucket of the excavator. The maximum depth of penetration into the boulder clay was approximately 0.6 m. The poles were trimmed at formation level using a chainsaw. The timber butt had a typical diameter of 0.3 m. The excavator used the finished road embankment as a stable platform to work on for the piling operations.

The weak nature of the peat surrounding the pile gave rise to concerns about the ability of the pile to resist lateral loads. A lateral load was applied to the top of a selected number of piles using the excavator. The piles required a significant force from the excavator to push the pile laterally.

A readily available A252 high yield steel reinforcing mesh consisting of 8 mm diameter bars 200 mm apart was placed on top of the butt. The sheets had dimensions of $2.4 \times 4.8 \text{ m}$ and were laid across the width of the road and overlapped by 400 mm. A separating geotextile was placed over the mesh to confine the embankment fill on the mesh. The fill was placed and compacted in approximately two lifts using the tracked excavator. The embankment fill was supported by a combination of soil arching in the fill between the static pile butt and the yielding peat, the transfer of load through the mesh onto the timber pile and some limited support from the peat to the mesh and the

embankment fill. The initial layout consisted of rows of three piles with a typical butt diameter of 300 mm driven 2.1 m apart, with 2.0 m between rows (Fig. 4).

3.2 Revised layout

The initial 20 m section was successfully completed. However an unforeseen supply problem meant that the local supply was used up over the first 15 m and smaller poles with a typical tip diameter of 150 mm were used. The smaller timber butts resulted in punching shear failure through the embankment and reduced support to the mesh over a 15 m long section. The smaller pile butts supported as few as two of the 8 mm diameter steel bars, causing the welded joints to fail. Additional basal reinforcement was provided by a layer of 5.5 m long closely spaced poles placed transversely across the road. To avoid having to install a pole mat and reduce the overall volume of timber being used in the road the pile layout was revised. Five piles were driven across the road at 1.2 m apart and 1.1 m longitudinally using piles with butt diameters between 150 and 200 mm (Fig. 5).

The revised layout provided more support to the steel mesh and probably increased the load being transferred to the piles by soil arching. The construction of this section took place without any difficulties.

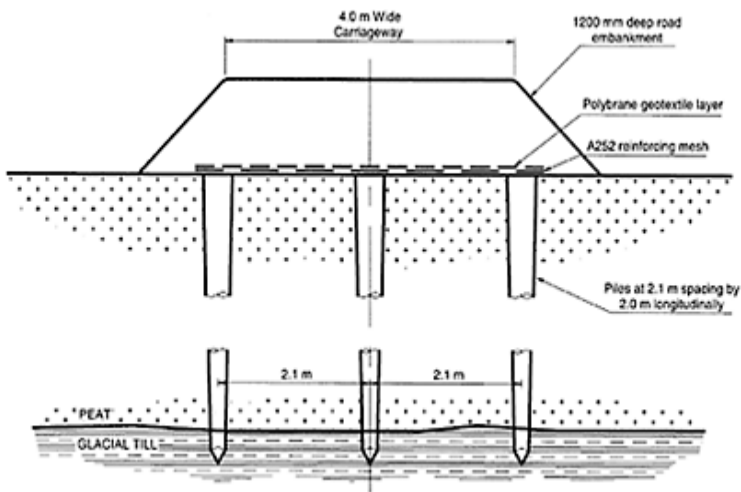


Figure 4. Initial layout of the piled road.

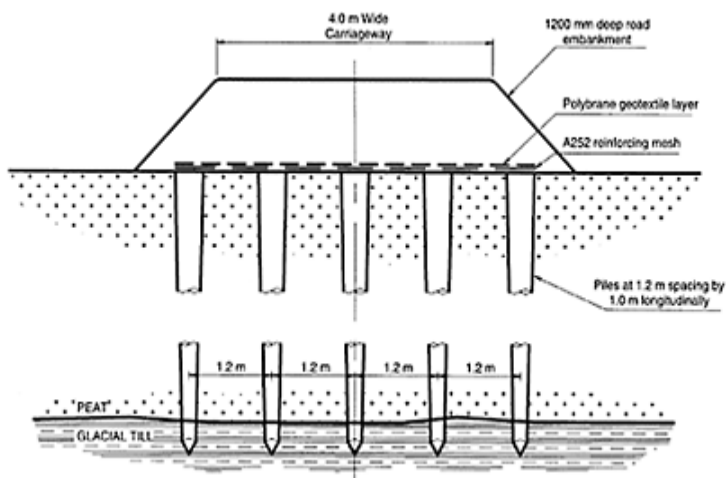


Figure 5. Revised piled road layout.

3.3 Geosynthetic reinforced section

Towards the end of the construction it was decided to examine replacing the steel mesh with a high strength geotextile. Two layers of a polyester geotextile (Stabilenka 200/45) were laid orthogonally over a 1.2×1.0 m pile grid. The transverse geotextile was lapped 2.5 m back into the fill to develop anchorage (Fig. 6). An identical fill height of 0.9 m over the piles was used to allow a comparison between the two support methods. The construction of this method was marginally

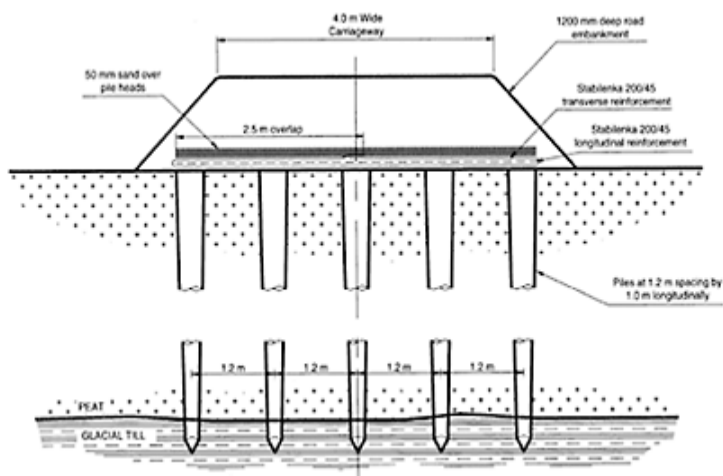


Figure 6. Layout of the geosynthetic reinforced piled forest road.

more difficult due to having to place, overlap and lap the two layers of geotextile into the embankment. The section performed well with no excessive rutting or deformations noted on the surface.

3.4 Post construction performance

The construction of the embankment began in January 2003 and was completed in 18 days at an average rate of 6.5 m per day. The road was left untrafficked and reassessed after six months. Some differential settlement occurred along the entire length of the route, to varying degrees. The initial 20 m section that was constructed using the larger timber performed adequately. The most problematic section encountered was where the rows of three piles had a reduced tip diameter of 150 mm. This section was reinforced by placing 5.5 m long poles across the width of the embankment surface and placing a further 900 mm of fill over the embankment. The composite construction was assessed again six months later and deemed satisfactory.

Shear cracking had developed along the edges of the top surface of the embankment during the construction period. Although the crack was narrow initially, the settlement of the support at the edges of the embankment and washing out of the finer material by rainwater during the interval caused the cracks to expand. The cracks do not greatly impinge on the road and will be repaired and monitored during the harvesting.

4 CONCLUSIONS AND RECOMMENDATIONS

The limited value of the plantation and the presence of deep deposits of peat prompted Coillte Teo to construct a piled forest road. The design philosophy was to construct the road using local materials as cheaply as possible in short sections and then modify the layout if necessary. The 120 m section was completed in 18 days at a total cost of €17,000 or €138/linear meter. This compares to typical budget estimates of €45 to €55/linear meter of forest road. The piled road method has proved to be a suitable alternative to traditional methods. It is suggested for future piled roads that the piles be closely spaced (1.2×1.0 m square grid) and that a sufficient number of piles should be installed to minimise the longitudinal cracking effects along the unsupported edges. The use of two or three layers of a geogrid rather than a polyester geotextile should be investigated. The geogrids require a shorter anchorage length and would help stabilise the shallow embankment. The cost of the geogrid is approximately the same as the A252 mesh.

The short design life of the road meant that degradation of the untreated timber poles above the watertable was unlikely to affect the usability of the road. For roads with a longer design life consideration will have to be given to protecting the upper portion from decay and also using geosynthetic reinforcement rather than a steel mesh.

REFERENCES

- Barry, A.J., Trigunaryah, B., Symes, T. & Younger, J. 1995. Geogrid Reinforced Piled Road Over Peat. *Engineering Geology of Construction*, The Geological Society, London: 205–210.
- B.S. 373 1957. *British Standard Method of Testing Small Clear Specimens of Timber*. British Standard Institution.
- D 143–52 1972. *Standard Method of Testing Small Clear Specimens of Timber*. American Society for Testing and Materials.
- O’Riordan, N.J. 1996. New Shannon Bridge, Athlone Approach Embankment: Design, Construction and Long Term Performance. In E.R. Farrell (ed.), *Road Embankments on Soft Ground in Ireland*, Institution of Engineers of Ireland, Dublin: 81–89.
- Quigley, P., O’Malley, J. & Rodgers, M. 2003. Performance of a trial piled embankment constructed on soft compressible estuarine deposits at Shannon, Ireland. In Vermeer, Schweiger, Karstunen & Cudny (eds.), *Proc. Int. Workshop on Geotechnics of Soft Soils-Theory and Practice*. Rotterdam: Balkema: 619–624.

Recycled and secondary materials

The performance of an experimental road constructed from quarry waste

L.R.de Rezende

Federal University of Goiás, Goiania, Goiás, Brazil

J.C.de Carvalho

University of Brasilia, Brasilia, Federal District, Brazil

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ABSTRACT: Brazil presents some construction and maintenance problems in its road networks. One of these difficulties is the scarcity of granular materials permitted for the traditional specifications in the composition of the base layers of flexible pavements. When this type of material is available, its exploitation could also be problematic due to environmental constraints. Consequently, it is necessary to study alternative materials that could substitute for granular materials, and present the technical and economical viability of those substitutes. One example of these material substitutes is quarry waste, which has the additional advantage of eliminating the need for stockpiling. This paper illustrates the utility of quarry waste located in the city of Brasilia (the Federal District of Brazil), as the component of the base layer of a low volume traffic road. The main characteristics of this quarry waste were identified in laboratory tests, and for 4 years, field tests were done on this experimental road. From this data, it can be observed that the quarry waste base presented satisfactory initial results, but with time, as a function of the great fluctuation of the rainfall from the dry to the rainy months, the controlled parameters worsened.

1 INTRODUCTION

Brazil has been verifying the shortage of natural granular materials, such as lateritic gravel that traditionally were used in paving works. In that way, several studies have been undertaken with the objective of identifying other types of materials to be used as substitute for the granular materials. The tropical soil, that is abundantly found in Brazil (Rezende & Camapum de Carvalho 2003a), and several wastes types (Rezende & Camapum de Carvalho 2003b) have been so studied. All these alternative materials should be of good quality and economically viable.

Besides the substitution of granular materials, the use of several wastes in the layers of pavements minimizes environmental damage, avoiding having to acquire additional land for deposition of those wastes. The main wastes studied for highway and urban pavements are mostly used in mixtures with tropical soils found at the places of the works for composition of bases and subbases. Throughout the world, many types of materials can be recycled as presented by Schroeder (1994) and FHWA (2004).

2 METHODOLOGY

The selected material for the study is a waste generated by a cement quarry, called Pedreira Contagem, and located near Sobradinho, a satellite city of Brasília (the capital of Brazil). It is a result from limestone blasting process. This material was first tested in laboratory and then its field performance was evaluated by “in situ” tests. Characterization, compaction and California Bearing Ratio (CBR) properties were obtained and analysed by Rezende (1999) and resilient modulus was previously reported by Vale (1999).

During the field tests, an experimental road, 80 m in length, was built on a highway, close to the cement quarry (DF-205 West) in 1998. This highway is classified as a low-volume road, with the standard axle operation number equal 7.6×10^5 . After that, “in situ” tests such as Pencil pressuremeter, plate loading tests, Benkelman beam and Falling Weight Deflectometer (FWD) tests were performed to analyse the pavement structural characteristic for four consecutive years.

The pavement structure encompasses a 20 cm thick base layer and a double surface treatment, which is 3 cm thick of chip seal. During construction on the base using the quarry waste material, particles larger than 10 cm were removed before water mixing and compaction. The relative compaction ratio obtained was 100%. The natural subgrade is composed of a fine tropical soil. However, as this experimental road was already functioning as a local dirt road, gravel layers, which had been deposited on the natural subgrade, were not removed. Consequently a more superficial roadbed exists with 20 cm of thickness, and other materials constitute a deeper layer below. With the results obtained in the field tests, it was possible to execute a back-analysis procedure.

3 LABORATORY RESULTS

Figure 1 presents the grain size curve of the quarry waste, which indicates that it is made of 65.9% gravel, 12.0% sand and 22.1% fines (silt and clay). Table 1 presents the index properties and specific

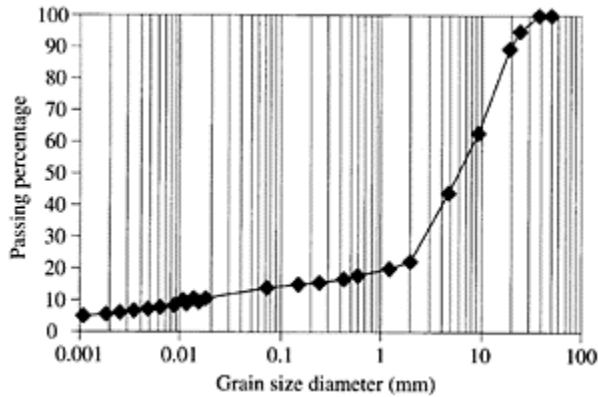


Figure 1. Grain size curve of the quarry waste material.

Table 1. Properties and classification of quarry waste material.

Properties	Value
w_L (%)	32.9
w_p (%)	24.5
PI (%)	8.5
ρ (g/cm ³)	3.0
TRB classification	A-2-4
USCS classification	GM
$\gamma_{d \max}$ (kN/m ³)	21.2
w_{opt} (%)	8.3
Swelling (%)	0.3
CBR (%)	27

gravity of the quarry waste material, as well its classification according to the Transportation Research Board (TRB) and to the Unified Soil Classification System (USCS). Table 1 also shows the properties related to compaction, swelling and the CBR for the quarry waste material compacted in the intermediate Proctor energy.

Vale (1999) carried out dynamic triaxial tests using the same material. Four samples were collected from different waste piles. Results are presented in Table 2, including values of resilient moduli. This table also presents properties of the natural granular material (lateritic gravel), commonly adopted in this region as base material. It can be noticed that the quarry waste material shows similar resilient modulus characteristics up to water content of 7.0%, although the CBR values are quite different. These results lead

to the conclusion that quarry waste material can successfully replace the natural material used for base layer in terms of deformation properties, because by increasing the stiffness the strain might be reduce and hence infer that deformation is reduced.

From the laboratory test results, one can notice the general trend of quarry waste material by analysing how the strength (CBR) and deformation (MR) parameters change due to material status, in terms of water content and dry unit weight. The trend and the relation between CBR and MR are presented in Figure 2. Consequently, for a CBR value of 27% (Table 1) the resilient modulus (MR) is equal 300 MPa. One can noticed that CBR is not necessarily correlated to MR and there are differences in testing procedures, such as in the load. Then, low CBR and high MR can mean that the material presents good quality, considering that MR is more consistent.

Table 2. Index properties and parameters of the quarry waste and lateritic gravel (Vale 1999).

Sample	W_{opt} (%)	$\gamma_{d\ max}$ (kN/m^3)	CBR (%)	$W_{MR\ test}$ (%)	MR (MPa) $\sigma_3=0.069$ MPa	Parameter k
Waste 1	6.0	22.0	59.1	5.0	471.6	$k_1=491.68$ $k_2=-0.0264$
Waste 2	8.4	20.2	33.8	7.4	351.0	$k_1=-256.59$ $k_2=-0.1885$
Waste 3	7.3	21.6	30.0	6.3	356.6	$k_1=397.73$ $k_2=-0.0081$
Waste 4	9.4	20.3	14.4	8.4	196.6	$k_1=116.43$ $k_2=-0.2744$
Lateritic gravel	12.5	20.7	103.0	11.5	386.6	$k_1=394.57$ $k_2=-0.0297$

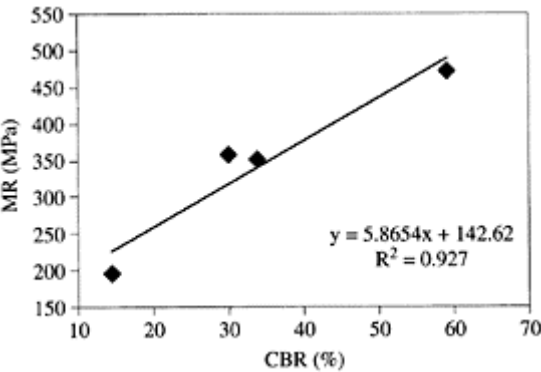


Figure 2. Relation between MR and CBR.

4 “IN SITU” TESTS RESULTS

4.1 Pencil pressuremeter tests

Pressuremeter tests can be carried out during and after pavement construction to act as control during execution or to evaluate the performance of the pavement and the need for reinforcement. It was found that the stress-strain hyperbolic model reasonably represents the loading and unloading cycles of the pressuremeter tests (Briaud et al. 1983). Then, moduli can be evaluated for any level of deformation from tests including only one loading and unloading cycle. This equipment has the advantage of low cost compared to others used for pavement evaluation. It also presents quick responses and is suitable for testing specific places or layers.

The used pressuremeter (Roctest), with a pressure capacity of 2500 kPa, is composed of a control unit, a cylindrical probe, a tube to saturate the system with water and a connecting tube between the probe and the control unit. The probe has 0.035 m of diameter and 0.230 m of length (fully expanded). The limit pressure (P_L) is defined by the applied pressure to double the initial volume of the cavity. The deformation modulus, one evaluated for the loading phase (E_p) and the other for the reloading phase (E_r), is calculated by the Equation 1:

$$E = \frac{2(1 + \mu)(V_o + V_m)}{1000V} \quad (1)$$

where E , deformation modulus in MPa; μ , Poisson ratio, taken as 0.33 for natural soil and as 0.45 for saturated soil; V_o , initial volume of the probe in cm^3 ; V_m , injected volume corresponding to the mid-range of the pseudo-elastic curve in cm^3 ; P , applied pressure changes in kPa; V , volume changes in cm^3 .

Table 3 presents the pressuremeter results, initially executed in July 98 and then subsequently in different seasons for the next four years. It is important to mention that the test holes were vertical, consequently the evaluated moduli refer to the horizontal properties of the material. One can notice that the quarry waste material presents the highest values of P_L just after construction (base compaction) and some decrease throughout the operation time. But in fact, in analysing the E_p and P_L values together, it is clear that this material performance is quite dominated by the water content, according to the rainy seasons (the higher the water content, the lower the deformation and strength parameters). For the quarry waste the horizontal modulus (E_p) changed from 36.9 to 19.9 MPa.

4.2 Plate loading tests

The plate-loading test was done using a metallic plate (25 cm of diameter and 25.4 mm thick), resting on a levelled and regularized surface of the layer tested. A jack ram and a load cell were placed between the plate and the reaction system (a truck axes). Four displacement dial gauges were evenly placed on the plate at every 90 degrees, using magnetic clamps linked to a 3 m long steel beam, which were taken out of the area affected by the tests. The capacity of the reaction system is 82 kN. The test procedure prescribes quick loading and unloading stages, in addition to displacement and load

readings 5 min after applying loads. Test results include a stress versus displacement curve,

Table 3. Pencil pressuremeter results throughout four years (1998–2002).

Date	Deep (m)	w (%)	E _p (MPa)	E _r (MPa)	P _L (kPa)
July (1998)	0.115	13.6	30.0	–	2200
September (1999)	0.115	6.9	36.9	73.9	1690
March (2000)	0.115	19.2	19.9	41.5	1200
December (2000)	0.115	11.8	27.2	74.5	1600
December (2001)	0.162	17.2	24.7	44.5	1520

from which the plate reaction modulus can be calculated (k_{PL}). This modulus is calculated by Equation 2 for the pressure of 0.56 MPa:

$$k_{PL} = \frac{P}{r} \quad (2)$$

where P , the pressure in MPa; r , displacement value corresponding to the adopted pressure in m.

Table 4 presents the main results obtained from the plate loading tests. Only the tests of July 98 were done laying the plate on the base layer just after prime coating, and all subsequent tests were done on the surface course. The highest modulus values were obtained right after the construction (base layer compaction and prime coating) and then, they decreased, as they became more sensitive to the higher material water contents with the advent of the rainy seasons.

4.3 Benkelman beam and FWD tests

The use of deflectometer equipment began in the 60s in Brazil, with the Benkelman beam. This kind of test has advantages related to test procedures as well as being non-destructive, that allows the deflectometer equipment to evaluate pavement structures, in measuring displacements caused by traffic loads. Initially, analysis took solely into account the maximum displacements. More recently, additional measurements were taken to determine the whole surface settlement basin, with the consequence that other desirable parameters were obtained.

The Benkelman beam test is standardized by the Brazilian Highway Department, which prescribes distance ratios of the beam equal to 2:1, 3:1 or 4:1 and a truck, with the back axis loaded to 82 kN and tire pressure set to 550 kPa (80 psi). The beam distance ratio is the relation between the distance from the rotating joint to the test probe and the distance from the displacement dial gauge to the rotating joint. Measurements are taken at the initial point and at intermediate distances. These values are used to calculate the displacements for each distance (commonly called deflections and expressed in hundreds of mm) and finally to obtain the deflection bowl and its curvature radius (R).

The Benkelman beam test was first carried out in July 98, just after its construction (base compaction). Since then, it has been executed on the surface treatment layer, in different seasons over the last four years. Tests were performed every 5 m along the experimental road segment. Benkelman beam results for the same places of other field tests are presented in Table 5 and the overall view of the average deflection bowls is presented in Figure 3a. Once more, the results present the same trend, with their best values just after base compaction and then some decrease, which is influenced by the water content of the material, caused by the rainy season.

The Falling Weight Deflectometer (FWD) is a non-destructive test and is also used to measure the pavement recoverable deflection. This equipment differs from the Benkelman beam in the way it applies loads, and its main advantage is related to quick and accurate measurements. This test is standardized by the Brazilian Highway Department, which also establishes the procedures to calculate

Table 4. Plate loading test results (1998–2002).

Date	w (%)	$r_{560 \text{ kPa}}$ (mm)	k_{PL} (MPa/m)
July (1998)	5.6	0.76	737
August (1998)	–	1.25	424
January (1999)	–	1.76	318
August (1999)	6.9	1.39	403
March (2000)	19.2	1.63	344
November (2000)	11.8	1.77	316
October (2001)	15.9	2.02	277
July (2002)	–	1.32	295

Table 5. Results of the Benkelman beam tests (1998–2002).

Date	D_0 (0.01 mm)	R (m)	k_B (MPa/m)
July (1998)	58.2	285.3	975
August (1998)	63.8	330.7	898
December (1998)	79.9	262.4	731
August (1999)	70.7	322.2	792
March (2000)	90.4	274.1	620
October (2000)	78.9	286.7	634
October (2001)	98.4	85.9	569
July (2002)	103.4	274.1	542

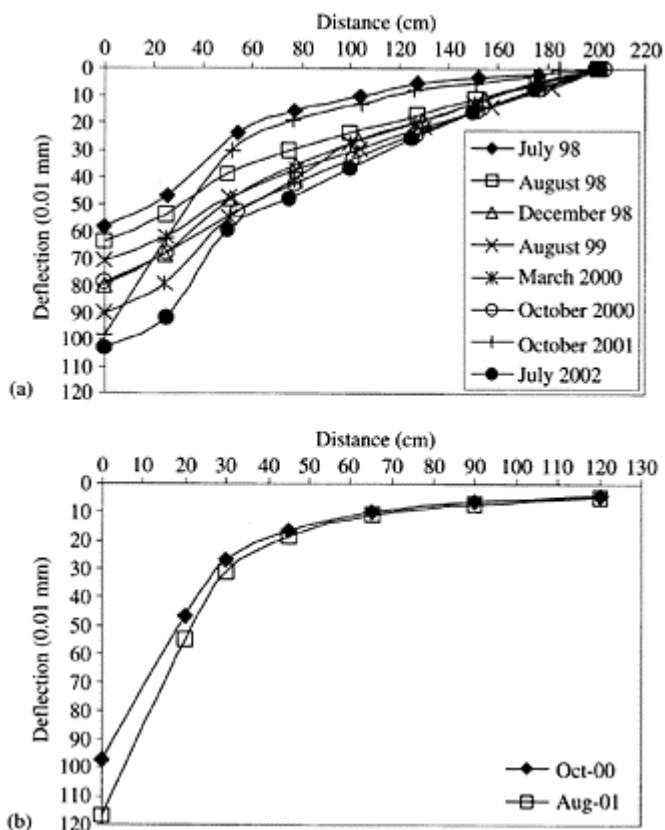


Figure 3. Average deflection bowls: (a) obtained by the Benkelman beam tests; (b) obtained by FWD tests.

FWD deflections. The FWD equipment simulates the traffic effects on the pavement, by dropping a set of weights from a certain height over a system of rubber shocker. This system is designed to approximate the load pulse by a sine curve.

Loads are transferred to the pavement by a metallic plate, of 15 cm diameter, and measured by a load cell. The duration of loading ranges from 25 to 30 ms, corresponding to a traffic speed between 60 and 80 km/h. Deflections are measured at up to seven points by velocity transducers or LVDTs (Linear Variable Differential Transducer). These measuring devices are placed in different positions and fixed to a reference beam, of 4.5 m length. Table 6 presents the parameters

Table 6. FWD test results (2000–2001).

Property	T _{pavement} (°C)	T _{air} (°C)	Stress (kPa)	D _o (0.01 mm)	k _{FWD} (MPa/m)
October (2000)	48	33	545	93.1	585.4
August (2001)	44	31	539	124.4	433.3

obtained from FWD tests, carried out in the same places of other field tests and Figure 3b depicts the average curves for the analysed period. One can notice a considerable increase in the maximum deflection values between 2000 and 2001. It can be explained for the higher water content observed in base material during 2001.

5 BACK-ANALYSIS

5.1 “*In situ*” tests

Using the programme Sigma/W (Geo-Slope 1995), a mesh of rectangular finite elements was defined of 3 m of width and 3 m of depth, composed of 1660 elements and 5147 nodes. In the plate loading back-analysis, the plate of steel was also represented and it was considered an elastic modulus of 200,000 MPa and a Poisson ratio of 0.27. A vertical pressure of 560 kPa was applied on the plate and for the analysis it was considered an axis-symmetric situation. For the subgrade a Poisson coefficient of 0.40 was adopted and for the quarry waste base the value 0.35 was adopted. For the beam Benkelman back-analysis the same mesh of finite elements was considered, but the traffic representation changed: in this case, the rigid plate was not considered and the pressure of 560 kPa was applied directly on the pavement. In both analyses the thickness of the surface course was incorporated to the base, because the surface course executed in double superficial treatment does not present structural function.

Table 7 presents the obtained results. It is verified that the modulus obtained by the back-analysis process of the two tests present different values. It can be explained by the differences existent during the tests execution. However, for the two field tests, it is observed that the values obtained for the base modulus are higher than the subgrade modulus. Through time, a reduction in the modulus values occurs: the modulus of the quarry waste base varied from 140 to 40 MPa in the plate loading test, and from 220 to 120 MPa in the Benkelman beam test. It is still possible to verify that all moduli values obtained from back-analysis for the quarry waste base were less than the value of resilient moduli defined in laboratory test.

Table 7. Back-analysis results for plate loading, Benkelman beam and FWD tests.

Date	Plate Loading		Benkelman beam		Falling Weight Deflectometer			
	E_{SG}^* (MPa)	E_B^* (MPa)	E_{SG} (MPa)	E_B (MPa)	E_{SG1} (MPa)	E_{SG2} (MPa)	EB (MPa)	Error (%)
July (1998)	85	140	160	220	–	–	–	–
August (1998)	70	70	160	200	–	–	–	–
January (1999)	40	55	130	150	–	–	–	–
August (1999)	40	90	160	170	–	–	–	–
March (2000)	40	60	120	130	–	–	–	–
November (2000)	50	50	140	160	106	163	217	11.6
October (2001)	40	40	100	120	81	143	202	12.8
July (2002)	40	90	100	120	–	–	–	–

* E_{SG} =subgrade modulus, E_B =base modulus.

Table 8. Variation between the relation of moduli values obtained from back-analyses.

Layer	E_{PLATE}/E_{BB}^*	E_{PLATE}/E_{FWD}^*	E_{BB}/E_{FWD}	E_{PLATE}/E_p^*	E_{BB}/E_p^*	E_{FWD}/E_p
Subgrade	0.2–0.9	0.2–0.5	0.4–1.3	–	–	–
Quarry waste base	0.3–0.9	0.2–0.3	0.6–0.9	1.6–3.0	4.9–7.0	6.7–9.1

* E_{PLATE} =moduli from plate loading test, E_{BB} =moduli from Benkelman beam test, E_{FWD} =moduli from Falling Weight Deflectometer test, E_p =moduli from Pencil pressuremeter test.

The programme Laymod4 (Rodrigues 2002) was used in the FWD tests back-analysis. It considers the pavement as a multilayer system. It is necessary to measure the following parameters: the basins data (local, load, displacement, temperature of the air and of the pavement), the number of pavement layers, the diameter of the plate that applies the load, the amount of displacement measures, the distances of the sensors and the characteristics of each material (type, thickness, Poisson ratio and modulus variation). In this experimental road, to reduce the value of programme error, the base layer thickness together with the surface course was considered. In addition, two different layers for the

subgrade, a superficial layer with 20 cm of thickness and a much deeper one, were considered. Table 7 also presents the main results for FWD back-analysis. It is verified that the quarry waste base presented moduli values higher than 200 MPa and that the resistance decreases when the two stages of tests are analysed. For the subgrade, it is observed that the superficial layer (E_{SG1}) presents smaller values of moduli than the deepest layer (E_{SG2}).

5.2 Relation between moduli values

With the obtained data it is possible to make a comparison between the values obtained for the modulus. It is verified that there is not a clear correlation between modules obtained from back-analysis processes executed with different field tests results. Table 8 shows the variation between the relations of moduli values.

It is also possible to compare values of the quarry waste base effective moduli obtained from back-analysis to the horizontal moduli determined from the Pencil pressuremeter test. As presented in the Table 8, it is observed that the largest values are verified for the relationship E_{FWD}/E_p and the smallest for the relationship E_{PLATE}/E_p . With those data it is verified that, for quarry waste, the effective modulus can be 1.5 to 9.0 times larger than the horizontal modulus, in function of the types of the executed field tests.

6 CONCLUSIONS

Analysing the data from laboratory and field tests, it is noticed that the performance of the quarry waste material can be evaluated adequately, since the main results are quite consistent. The main conclusions of this study are:

- The studied quarry waste material can be used as construction material for flexible pavements, replacing natural materials traditionally used, such as the lateritic gravel.
- Laboratory tests illustrated the enormous potential of the quarry waste material as pavement base (the resilient modulus can present values higher than 300 MPa).
- Results obtained from field tests performed on the experimental road and from back-analysis indicated that the performance of the quarry waste material is quite dependent on its water content, and consequently the rainy seasons. The wetter the material, the higher the displacements and the lower the moduli values for all tests. The results indicate the need of a correct drainage design for the road where the quarry waste will be used. It could be desirable to have some wet stability for base materials.
- Direct correlations between the identified parameters for different field tests do not exist.
- In analysing the moduli, the values for the relation of the quarry waste effective moduli obtained from back-analysis presents different values.
- As the field evaluation was performed on a low-volume road, quarry waste material is recommended for this kind of road and more investigation is suggested to widen its applicability.

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REFERENCES

- Briaud J.L., Lytton, R.L. and Hund, J.T., 1983. Obtaining module from cyclic pressuremeter test. *ASCE, Journal of Geotechnical Engineering*, 109(5), pp. 657–665.
- FHWA, 2004. User guidelines for waste byproduct material in pavement construction. *FHWA-RD-97-148*, <http://www.tfhrc.gov/hr20/recycle/waste/begin.htm>, (Jan 11, 2004).
- Geo-Slope, 1995. Sigma/W version 3. *User's Guide*, Geo-Slope International Ltd, Calgary Alberta, Canada. p. 390.
- Rezende, L.R., 1999. Alternative techniques to constructed highway base layers (in Portuguese). *MSc thesis*, publication number G.DM-055A/99, Department of Civil and Environmental Engineering, University of Brasilia, DF, Brazil, p. 169.
- Rezende, L.R. and Camapum de Carvalho, J., 2003a. Use of locally available soils on subbase and base layers of flexible pavements, *Proc. 8th Conf. on Low Volume Roads, Journal of the Transportation Research Board*, no 1819, Volume 2, Reno, Nevada, USA pp. 110–121.
- Rezende, L.R. and Camapum de Carvalho, J., 2003b. The use of quarry waste in pavement construction. *Resources, Conservation & Recycling*, Special Edition on Utilization of wastes, volume 39, issue 1, pp. 91–105.
- Rodrigues, R.M. 2002. Laymod 4 Program, Department of Geotechnical Engineering, *Technological Institute of Aeronautical*, Sao Jose dos Campos, Sao Paulo, SP, Brazil.
- Schroeder, R.L., 1994. The use of recycled materials in highway construction, *U. S Federal Highway Administration*, <http://iti.acns.nwu.edu/clear/infr/prau94.html>, (Jan 11, 2004).
- Vale, C.C.L., 1999. Geotechnical considerations on recovering degraded areas from limestone mining (in Portuguese). *MSc thesis*, publication number G.DM-062A/99, Department of Civil and Environmental Engineering, University of Brasilia, DF, Brazil, p. 106.

A laboratory study of the early life performance of a slag bound base

N.H.Thom & O.Wood

Nottingham Centre for Pavement Engineering, University of Nottingham
N.Ghazireh

Tarmac Limited

Pavements Unbound—Dawson (ed.)

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ABSTRACT: Whilst numerous studies have demonstrated the potential of slag bound material, the issue of early life trafficking has generally not been addressed. This paper presents the results of a series of tests in which simulated trafficking was applied to slab specimens of slag bound material overlaying a rubber support during the curing period. The severity of trafficking was varied from specimen to specimen. At every stage of the experiment, the effective stiffness of the slabs was measured. The resulting data clearly indicated that the number of load applications applied was critical in determining the eventual stiffness achieved by the material. The significant period during which repeated loading was found to damage the material commenced about 2 weeks after compaction. Prior to this, even though heavy construction traffic loading was simulated, there was no detectable long-term damaging effect. Beyond this point, the specimen which was subjected only to infrequent traffic gained stiffness appreciably, reaching levels typical of a cement bound material. The other specimens, subjected to much more frequent traffic, reached lower stiffnesses, stabilising after about 2 to 3 months. The paper concludes by suggesting an appropriate approach to design of this type of pavement and giving an example, based on the results obtained.

1 INTRODUCTION

The pressure to use “alternative materials” is a worldwide and growing phenomenon. Not only could slags be considered as “alternative”—they certainly fulfil the criteria of being a waste product—but they also have the benefit that many slags themselves have cementing properties. They therefore have the potential not only to act as an alternative aggregate source but also to act as an alternative binder.

This should mean that slags are seen as prime products for use in highway construction. Indeed, numerous studies (e.g. Sherwood, 2001; Nunes, 1997) have accepted the value of these materials. However, the situation regarding pavement design in the UK at present means that they are caught in a trap. There is often no problem using slags as direct replacement for unbound materials but there is no accepted means of taking account of their superiority due to their binding action. The first step toward solving this dilemma will shortly be taken with the publication (by the Transport Research Laboratory) of designs allowing different qualities of pavement foundation. High quality foundations will attract a thinner upper pavement design—although there remains the issue of establishing just which foundation category a pavement with a slag bound sub-base should be assigned to. However, the real aim of slag users is that slag bound materials are seen as more than simply superior sub-bases but that their potential as road bases is acknowledged—as is already the case in France for example. For this, it is necessary to develop real confidence in the long-term properties of a slag bound foundation, not simply in the laboratory but in a pavement.

In this regard, the key issue is the fact that the binding action of a slag is, like that of bitumen emulsion or foamed bitumen, a slow one. This means that the layer will only have begun to gain strength when the first heavy vehicle load is experienced. The unknown quantity is the degree to which curing of a slag bound layer is affected by traffic loading, and it is this matter which is the subject of the tests described in this paper.

2 MATERIAL TESTED

The material selected for this set of experiments was a slag bound aggregate. The mass proportions of the mixture used were 75% graded Greywacke aggregate (within the UK Type 1 grading envelope), 15% granulated blast furnace slag (GBS) and 10% basic oxygen slag (BOS), mixed with 9% water. The BOS is required since it contains free lime, which is used as an activator. The role of an activator is to increase the rate of hardening of the SBM and it can be an alkaline and/or sulfatic material. This includes hydrated lime, calcareous fly ash, cement, mixtures of gypsum and lime, sodium and potassium salts and similar products.

The Greywacke was not selected for any particular reason other than it is representative of aggregates with no self-cementing action. In reality, economic use of slag binders, as with emulsion or foamed bitumen, is often expected to require the inclusion of secondary aggregates rather than primary crushed rock.

3 EXPERIMENTAL ARRANGEMENT

Since the principal aim of the test series was to investigate the effect of trafficking during the curing process, it was necessary to devise a system which applied repeated load to the specimens in as realistic a way as possible. This necessitated a supported slab structure, mimicking the situation in a pavement.

The constraints on the test arrangement used were that it had to accommodate slabs of material compacted in the Nottingham roller compactor (Gibb, 1996), with plan dimensions 305 mm× 305 mm. In fact, it was decided to utilise the slab moulds to provide lateral restraint to the slab during simulated trafficking. Practicalities also dictated a limit to the slab thickness possible, and it therefore had to be accepted that the structure tested would have to represent a scaled down version of a pavement. The set-up in Figure 1 was therefore chosen, with a 50 mm thick slab of slag bound material overlying a rubber support.

The whole equipment was placed in a Mand universal testing machine and seated on a rigid base when loading was required. The rubber support was of approximate stiffness modulus 3 MPa and thickness 10mm and was designed to simulate the stiffness of a typical pavement foundation, very approximately of 100 MPa equivalent stiffness. The diameter of the loading plate was 100 mm. The arrangement was therefore approximately one-third scale in comparison to a real pavement, where a 300 mm diameter contact is commonly assumed, and the thickness of the layer is likely to be 150 mm or more.

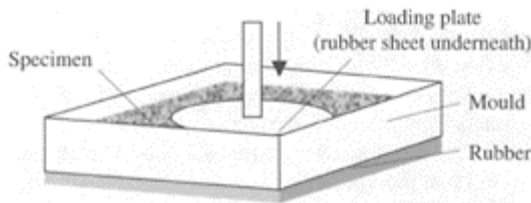


Figure 1. Diagram of test arrangement.

In order to evaluate stiffness, vertical deflection at the surface of the slab was measured (under a standard load) at three positions outside the loading plate.

4 APPLIED LOADING

Three levels of load were applied, as follows:

- 5.5 kN; this gave a contact pressure of 700 kPa and represented direct trafficking by construction vehicles prior to the application of a surfacing.
- 1.8 kN; the contact pressure reduced to 230 kPa and was intended to represent the stress level expected at the surface of a base layer beneath about 150 mm of surfacing, i.e. the in-service load condition. During this type of loading an overburden was applied to the surface around the loading platen to simulate the dead weight of the surfacing. However, the dead weight simulated was in fact only of the order of 50 mm and this, together with the low stiffness of the underlying rubber, meant that the loading case was a severe one.
- 1.0 kN; this was the load at which surface deflection measurements were taken for stiffness calculation purposes.

For stiffness measurement (load type (iii)) a slow loading rate of 0.1 Hz was used in order to facilitate reading of instruments. In other cases, 5 Hz was used, it having been established that the equipment was capable of applying the specified load at that rate.

5 TEST PLAN

Three slabs of material (from the same batch) were compacted in the Nottingham roller compactor, each seated on a 10 mm rubber layer over the base plate of the mould. The compaction level used was the same in each case. A small amount of surface cracking appeared during the rolling operation and this was eliminated by applying the final compaction through a stiff wooden board.

The following simulated traffic was then applied to each specimen:

Specimen 1:	50 simulated construction traffic loads after 24 hours 50 further construction traffic loads after 10 days 5000 simulated in-service truck loads per week thereafter
Specimen 2:	10 simulated construction traffic loads after 10 days 5000 simulated in-service truck loads per week thereafter
Specimen 3:	10 simulated construction traffic loads after 10 days 100 simulated in-service truck loads per week thereafter

The simulated in-service loads were applied in concentrated bursts, generally once a week although there was some slight variation in the periods between loadings. In one particular week, for Specimen 2, the 5000 loads were applied in 5 separate bursts on different days. Each time a specimen was loaded, its stiffness was measured (see next section) both before and after loading.

Between test days, the specimens were covered and stored at room temperature (20°C). Moisture was added to the surface whenever it appeared that a specimen was drying out beyond what would be likely beneath an asphalt layer.

6 STIFFNESS CALCULATION

Stiffness modulus has been deduced from the bending of the slab under load, as measured by surface deflections. The relationship between surface deflection (actually the slope of the deflected surface) and stiffness modulus has been computed assuming a “dense liquid” foundation. This is by no means an accurate description of the rubber support used, but the resulting stiffnesses were found to be of the anticipated order of magnitude. This was in contrast to two other techniques using computer analysis. The first, using the multi-layer linear elastic analysis program BISAR, gave very low stiffnesses indeed for the deflections measured. This is undoubtedly largely due to the inaccuracy involved in assuming the slab to be infinite in horizontal extent. The second program tried, ILLISLAB (Ioannides et al., 1985), was written to model discrete slabs in bending

(usually jointed concrete) and it assumes a “Winkler” foundation, analogous to a set of springs, which takes no account of the horizontal straining of the rubber. The results from ILLISLAB appeared better, but still clearly failed to match the form of the measured deflections.

The equation derived from the “dense liquid” foundation assumption was as follows:

$$\text{Stiffness (MPa)} = 13 / (\delta_{75} - \delta_{25}) \quad (1)$$

where δ_{75} and δ_{25} are deflections measured 75 mm and 25 mm from the edge of the slab (in microns) under a 1 kN load.

7 TEST RESULTS

Figure 2 shows all the stiffness data for all three test specimens. Day 10 represents the application of an asphalt surfacing. The “zig-zag” pattern thereafter is due to the way in which the simulated trafficking was carried out in bursts.

The following are seen as the main points to emerge:

- The level of in-service traffic is clearly of great importance. With only 100 trucks per week, the eventual stiffness for Specimen 3 settled down between 3000 and 4000 MPa, which is a value typical of weak lean concrete.
- The zig-zag pattern reveals that the material would certainly have been even stiffer if no trafficking had occurred at all. In contrast, the stiffnesses from the more heavily trafficked specimens have levelled out at around 400 MPa, which compares with the value expected for a UK Type 1 sub-base of around 150 MPa.
- The very large drop in stiffness on Day 10 for Specimens 2 and 3 following 10 construction vehicle loads, representing the plant required to lay the surfacing layer, reveals that considerable

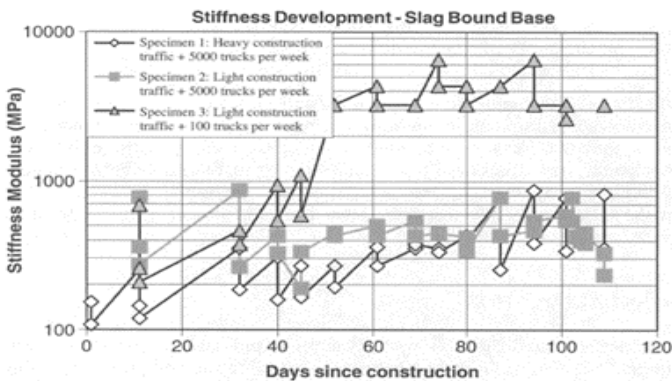


Figure 2. Development of stiffness throughout the test period.

damage occurred. Clearly, however, the recovery of Specimen 3 means that the damage was reversible as curing continued.

- The relatively minor difference between the long-term stiffnesses of Specimens 1 and 2 implies that the heavy early construction vehicle loading experienced by Specimen 1 had little longterm significance.
- The cluster of data between Days 100 and 105 is due to the carrying out of loading on Specimen 2 each day rather once in the week, but with the same number of load applications in all. This clearly resulted in a decrease in stiffness for that specimen in comparison with Specimen 1, which continued to receive its weekly traffic burst.
- 90 days would appear to represent the period required for a long-term stiffness value to become established for this particular material.

The only other testing carried out was determination of cube compressive strengths. This was conducted at 7 and 35 days, three cubes being crushed at each stage. The results were 1.1 to 1.5 MPa (average 1.3 MPa) at 7 days and 4.2 to 4.9 MPa (average 4.7 MPa) at 35 days. This suggests that the material would probably have reached the strength of a weak to medium lean concrete (UK category CBM2) in the long term. The compressive strength at 90 days was not determined.

8 DISCUSSION

The key issue here is the degree to which early-life trafficking can reduce the long-term stiffness of a slag bound layer. It is worth noting that stiffness values found for laboratory cured specimens are usually significantly higher than those found for the above specimens. Indirect Tensile Stiffness Modulus testing recently commissioned by one of the authors on a similar strength material found moduli mainly in the range 8000 to 15 000 MPa (at an age of 90 days), sometimes greater depending on the mixture design and in particular the nature of the aggregates and the activator type. One could also refer to experience from in-situ evaluation, although validated long-term results are not readily available since such materials have not traditionally been used. However, the authors have encountered examples of road base layers containing slag which, though originally intended to be purely granular, have in reality self cemented. An example from South Wales yielded a material with a compressive strength of about 4 MPa and an in-situ stiffness typically in the range 1000 to 2000 MPa. In this case, it underlay 175 mm of asphalt. Cement bound materials of similar strength have routinely been found to give similar or only slightly greater long-term in-situ stiffness.

The critical factors would appear to be the amount of protection offered by the asphalt and the severity of early trafficking. It seems clear that very early trafficking, within about the first two weeks of construction, is not an issue. Any damage which occurred during this period was found to be reversible. However, as soon as serious strength gain commenced, damage from traffic had a significant effect on long-term stiffness. Basically, the data available from these tests consists of just two values for long-term stiffness, about 3000 MPa under 100 heavy goods vehicle passes per week and 400 MPa under 5000 passes per week. One could begin to extrapolate from this limited data, first suggesting that a heavy goods vehicle might typically apply the equivalent of about 2 standard 80 kN axles and then by multiplying the traffic up over a 20 year design life,

giving about 0.25 and 13 million standard axles (msa) respectively—making an allowance for growth. One could then postulate a power law relationship between traffic and long-term base stiffness modulus, and in this case the resulting relationship would be approximately as follows:

$$E_{\text{base}} = 1500 N^{-0.5} \quad (\text{where } N \text{ is 20 year traffic in msa}) \quad (2)$$

The next step is to compute the stress in the base during early-life trafficking under different thicknesses of asphalt. It is suggested that maximum tensile stress is probably the most significant parameter in relation to possible base degradation. By way of example, this has been done assuming an asphalt stiffness of 4000 MPa and a base stiffness of 1000 MPa, bearing in mind this is for early-life conditions.

To continue, it is next necessary to postulate a relationship between tensile stress in the slag bound base under traffic and rate of damage. Making the highly speculative assumption that a ‘fourth power law’ applies, the difference in stress level (due to variation of asphalt thickness) can be equated to an equivalent difference in traffic level, at the stress level used in the experiments.

This adjusted traffic can then be input into the above equation to derive a value for long-term base stiffness (assuming that trafficking commences early in the life of the pavement). This would imply, for example, that use of 100 mm of asphalt in a 1 msa design would leave a long-term base stiffness of 4500 MPa. If only 50 mm of asphalt was used, the long-term stiffness becomes a little over 2000 MPa, whereas with 200 mm of asphalt it becomes well over 10 000 MPa.

This is instructive, but it is only really of use if the design traffic is genuinely compatible with the asphalt thickness chosen and the resulting long-term base stiffness. This can be checked using analytical design principles, assessing the strain in the asphalt layer. For purposes of illustration, this has been carried out using the Nottingham University program OLCRACK (Thom, 2000), which takes account of both top-down and bottom-up cracking through the asphalt layer. A longterm stiffness of 4000 MPa has been assumed for the asphalt, together with fatigue characteristics typical of a dense graded mixture, although these variables would clearly depend on the material type used. Additionally, it has been assumed that the slag bound base layer is 150 mm thick and that the underlying foundation is of stiffness 150 MPa.

By insisting that the design traffic, asphalt thickness and long-term base stiffness are compatible in this way, a single set of optimised designs can be derived. These are illustrated in Figure 3. The figure also shows the effect of the assumed damage law on design, the 6th power assumption (thought likely to be the more realistic) leading to much lower predicted damage, higher long-term base stiffness and therefore longer life. Typically, the long-term base stiffness is predicted to lie between 2000 and 6000 MPa for optimised designs.

The designs in Figure 3 are merely presented as examples of a sensible way forward in design of such pavements. As stated above there are numerous variables for which values have been assumed and, most importantly, the relationship between asphalt thickness, traffic intensity and long-term slag bound base stiffness requires significantly more data to be obtained to give confidence in the predictions made. It will then be necessary to

evaluate base mixtures, having different strength gain characteristics and different long-term strengths.

It is noted that designs which the UK Highways Agency is expected to issue during 2004 are not dissimilar in thickness to those shown in Figure 3 with a 6th power damage assumption.

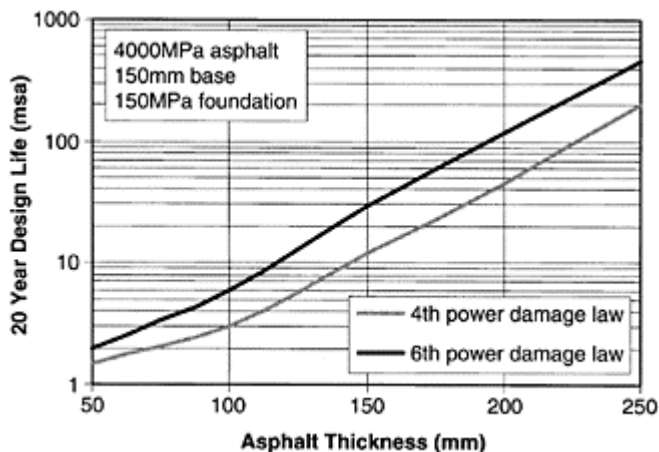


Figure 3. Examples of optimised designs for slag bound base pavements.

9 CONCLUSION

Three tests are not enough for any final judgements to be made. However, the trends revealed are clear enough and require more detailed study. Firstly, it is clear that the long-term stiffness applicable to such materials cannot be simply derived in advance from laboratory-cured specimens. The difference between the results for the two quite different levels of in-service traffic reveals the importance of this factor. Whilst the two most heavily trafficked specimens have been treated in an unusually severe manner and the long-term stiffnesses measured may therefore have been unrealistically low, it is also probable the third specimen was treated excessively lightly for many applications. It is absolutely essential for further data to be generated if designers are to be able to use such materials with confidence.

The fact that daily traffic application was, in the one case where it was tried, more damaging than weekly means that experiment design in any future work will be a critical factor. Ideally, the trafficking should be applied throughout the period, although compromises will almost certainly have to be made for practical reasons.

However, despite the relatively small data set obtained in this work, a method has been proposed which takes account of the likely long-term stiffness of a slag bound base

and which is clearly capable of generating meaningful designs. It is suggested that this methodology is the most appropriate for this type of pavement.

ACKNOWLEDGEMENTS

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REFERENCES

- Gibb, J.M., 1996, "*Evaluation of resistance to permanent deformation in the design of bituminous flexible pavement mixtures*", PhD Thesis, University of Nottingham,
- Ioannides, A.M., Thompson, M.R. and Barenberg, E.J., 1985, "Westergaard solutions reconsidered", *Transportation Research Record* 1043, Transportation Research Board, Washington, pp 13–23.
- Nunes, M.C.M., 1997, "*Enabling the use of alternative materials*", PhD Thesis, University of Nottingham.
- Sherwood, P., 2001, "*Alternative materials in road construction*", 2nd Edition, Thomas Telford Publishing.
- Thom, N.H., 2000, "A simplified computer model for grid reinforced asphalt overlays" *Proceedings of the 4th International RILEM Conference on Reflective Cracking in Pavements*, Ottawa, pp 37–46.

The use of recycled aggregates in slag bound mixtures

N.Ghazireh & H.L.Robinson

Tarmac Limited, Technical Centre, Wolverhampton, England

Pavements Unbound—Dawson (ed.)

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ABSTRACT: In the UK, bound sub-base and/or base usually comprise virgin aggregates with bitumen or Portland Cement as a binder. Production of these binders is energy intensive and use of alternative “secondary” binders could realise considerable cost and energy savings during road construction. Alternative hydraulic binders exist but their use in the UK is currently limited. Granulated blast furnace slag when activated with lime, or lime based material, demonstrates cementitious properties and can act as a binder for any suitable aggregate to form “slag bound material (SBM)”. SBM is widely used throughout Europe for sub-base and base layers in the construction of all categories of road. SBM is a cold mix/cold lay material which develops a high level of stiffness and strength compared to conventional bituminous mixtures and similar level of performance to cement bound materials, but at a slower rate. Whilst numerous studies have demonstrated the potential of SBM using natural or slag aggregates, there appears to be little if any previous research looking at the use of recycled fine aggregates generated by highway and utility arisings. This paper presents the results of a series of laboratory tests obtained on selected mixtures of SBM utilising recycled aggregates. Effects of mix design on the early life strength and longer term strength development have been investigated and the results are compared to a conventional mixture utilising natural aggregates. The paper concludes by suggesting optimised mix design for recycled aggregates with high levels of performance.

1 INTRODUCTION

Slag bound mixtures (known as SBM) were developed by LCPC (Laboratoire Central des Ponts et Chaussées) and Laitier Nord in the late 1960's as a response to the need to provide lower cost options for strengthening the existing National French road network. Between 1972 and 1992 approximately 10,000 km of the French network was

strengthened using hydraulically bound materials of which 7,000 km incorporated granulated blastfurnace slag (GBS) as the binder. In 1994, 3.4 million tonnes of SBM (or Grave-Laitier) was produced mainly in the North and West of France and used in both new and re-build constructions.

SBM can be thought of as having an aggregate grading similar to a DBM but with the bitumen being replaced with GBS present at around 15%. The French sometimes pre-grind the GBS to improve its reactivity, however this tends to be done within 24 hours of use to avoid the ground GBS “setting off”. Any aggregate type can be used in SBM providing it fulfils all the usual requirements for pavement bases (Refs 1 & 2). In France the preferred activator for triggering the binding action of the GBS is “Gypsonat” which is a blend of gypsum and lime. The general development of the hydration is envisaged to occur in two stages: one associated with water absorption, in which free lime may be an effective activator, and the second with water evolution, which is enhanced by the presence of gypsum. The Gypsonat activator is found to give the best long term strength development when compared to other activators (Ref. 3).

SBM comprise typically of 0/20 mm continuously graded aggregate with 15% GBS and a lime based catalyst of 1% and water. The water content depends highly on the water absorption of the aggregate's. This could vary from 3% to 8%. In these mixtures natural, blast furnace and steel slag aggregates could be produced to the required grading, but what about recycled aggregates? This is a growing area recognising government have set targets to encourage recycling in accordance with the principles of sustainable development.

The production of Type 1 and 2 granular sub-base based on recycled aggregates is usually achieved by crushing bricks, concrete and asphalt planings. This crushing process generates byproduct materials which are a mixture of fines and coarse aggregates with limited control over grading. Large quantities of these materials are currently available in growing stockpiles across the UK. They have a relatively high fines content which limits reuse into low value bulk fills, ground raising schemes or as bund materials. SBM represents a higher value end use for recycled materials as bound sub-bases and bases.

The results presented in this paper represent the initial findings of a longer term testing programme. Although the results presented herein are obtained on one source of recycled aggregates the study is currently investigating several sources and the effect of the constituents and nature of the recycled aggregates on the overall performance of the SBM will also be investigated.

2 MATERIAL DESCRIPTION

The materials used in this study were sampled from the Tarmac Recycling Ltd site in Ettingshall, Wolverhampton. The estimated size of the material stockpile in that unit alone is 12,000 tonnes. The material is graded 0/40 mm, is mixed with fines generated from crushing, some silts and clays and other physical contaminants. Thus it fails to meet the regulations and specifications for use in construction applications and as a result sits in a stockpile and is difficult to use. Previously all such material was sent to landfill but increases in the Landfill tax increased the economic burden of these materials and their disposal. Consequently, identifying alternative uses for this material could transform an

economic burden into a value added product. The material has been used as a general bulk fill and for filling underground voids, however modest amounts were used in these applications.

It was important to gain an understanding of the nature of this stockpiled material prior to commencing work on potential applications. Due to its variability it was crucial to gain representative samples from the large stockpile. This was achieved by excavating longitudinal trenches in the four directions of the stockpile (i.e., north, south, west and east). Numerous samples were then

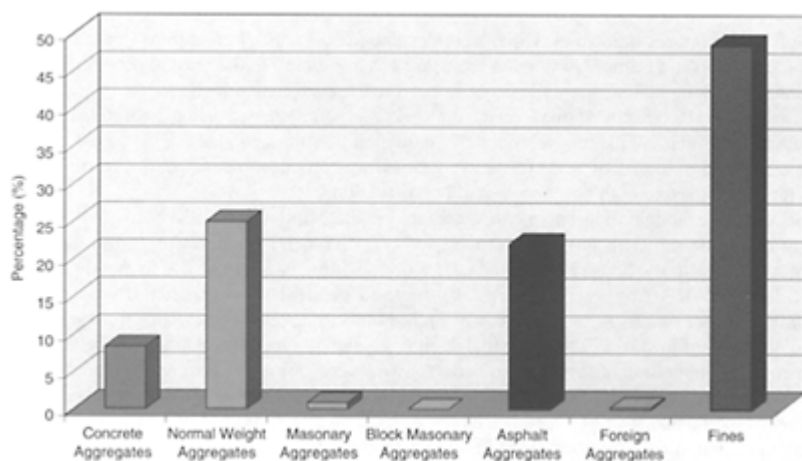


Figure 1. Typical composition of the material studied in this paper.

extracted from each trench at various locations, mixed dry in a cement mixer before being coned and quartered into representative lots. Two opposing quarters were further mixed in the cement mixer and manipulated to form another cone, quartering providing four representative samples. The method was repeated for the remaining two quarters and thus, eight representative large samples were formed.

The material consisted mainly of: 23% crushed natural aggregates; 6% concrete aggregates; 20% crushed asphalt planings; 46% fines, which is a blend of different aggregate dusts (generated by the crushing operation), some silts and clays (i.e., passing 5 mm). The remaining 5% consisted of masonry, bricks, glass and plastic fine aggregates (see Figure 1). The net result of the constituents analysis showed the wide variability in composition dependent on many aspects such as geographical location and type of collected highway arisings. The material composition described above is only typical of the source studied and the composition will vary from location to location.

3 LABORATORY TESTING PROGRAMME

3.1 Characterisation testing

The characterisation testing consisted of: Particle Size Distribution, Sub-Sieve Size analysis using the Warman Cyclosizer, Density, Porosity, constituents analysis, pH, Loss on Ignition, Carbonate Content, X-Ray Diffraction, Scanning Electron Microscopy and leaching tests. The main conclusion from the characterisation testing is reported in Section 4.

3.2 Slag Bound Mixture (SBM) testing

The purpose of this testing programme is to study the behaviour of SBM incorporating the recycled aggregates described above. Several mixtures were specifically designed in order to study the following characteristics:

- Measure the strength and stiffness development over 365 days.
- Evaluate the effect of the GBS content on the Compressive Strength and Indirect Tensile Stiffness Modulus (ITSM).
- Determine the shelf life of the mixtures.
- Determine the effect of shelf life/workability on the strength and stiffness profiles.

To achieve the above targets the following mixtures were adopted (Table 1).

Blend C is adopted as a “control mix” to act as a benchmark. The limestone aggregates used were obtained from Tarmac’s Cauldon Low quarry in Derbyshire.

Due to the variability of the recycled aggregate’s it was anticipated that the water absorption for each mix could vary. Therefore the optimum moisture content (omc) was determined for each mix using a vibrating hammer and taking 5 measurement points. The moisture content adopted for each mixture was (omc+1%).

Over 400 kg of material was mixed for each blend. This was carried out in the laboratory using a very large cement mixer with a capacity of 200 kg of materials per mix. For each mix, the aggregates

Table 1. Mixture details for SBM using recycled aggregates.

	Limestone aggregates (%)	Recycled aggregates (%)	GBS (%)	Steel Slag (BOS) (%)
Blend C	75	–	15	10
Blend 1	–	70	15	15
Blend 2	–	75	15	10
Blend 3	–	80	10	10

Blend 4	–	90	5	5

were mixed dry until a uniform mixture was achieved. Water was then added to achieve the target moisture content. The mixture was then stockpiled in the storeroom. 12 number cylinders (150 mm diameter×70 mm height) and 12 number cubes (150 mm) were then cast, sealed in plastic bags and cured at 20°C. One week later, the same stockpile material was utilised to prepare another set of specimens consisting of 12 number cylinders and 12 number cubes. The same procedure was repeated after 2 and 4 weeks. Over 250 number cylinders and 250 number cubes were prepared.

The prepared specimens were all cured at 20°C and tested at 1 month, 3 months, 6 months and 12 months for compressive strength and ITSM. The results of these tests are presented and discussed in Section 4.

4 RESULTS AND DISCUSSION

4.1 Control mix

Figure 2a has some data scatter, however the trends show that a four week old SBM loose mix when compacted still achieves about 8 GPa at 3 months and about 10 GPa at one year. The SBM

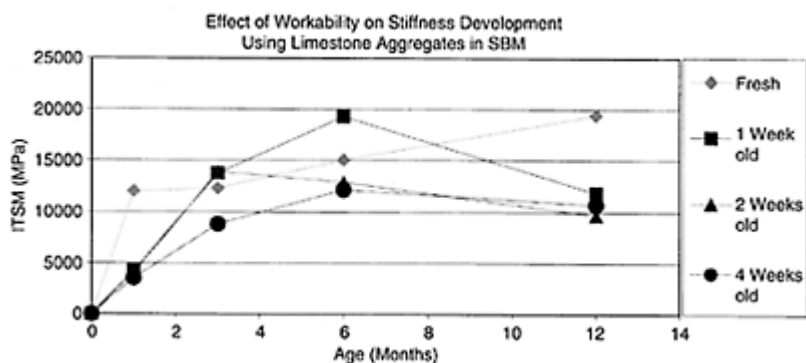


Figure 2a. Control mix stiffness development.



Figure 2b. Development of compressive strength for the control mix.

fresh mix i.e. compacted immediately after mixing, would appear to come through and perform the best at one year. This may be due to some moisture being lost through evaporation in the stored mixtures which may then limit the GBS hydration and hinder long term stiffness development.

Figure 2b shows that storing the mixes for between 2 and 4 weeks prior to compaction has some detrimental effect on the strengths at one year.

4.2 Blend 1

Figures 3a and 3b show that the mix stored for 4 weeks prior to compaction delivers the best performance at all ages for both stiffness and compressive strength.

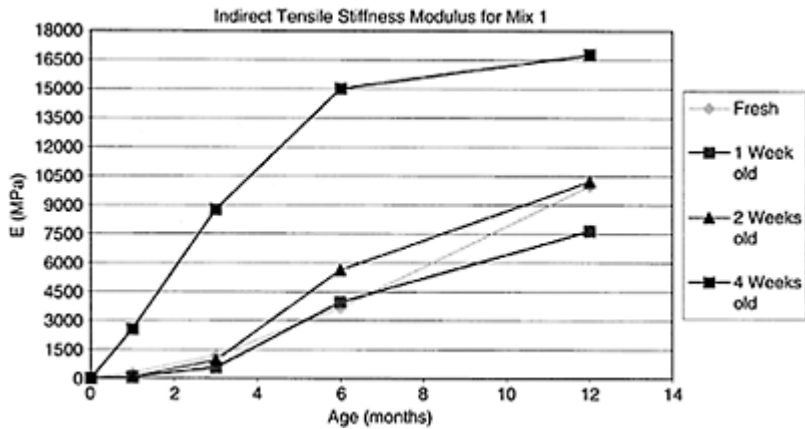


Figure 3a. Development of stiffness for blend 1.



Figure 3b. Development of compressive strength for blend 1.

4.3 Blend 2

Blend 2 (Figures 4a and 4b) shows similar trends to blend 1, however the overall stiffness and strength profiles are lower due to less steel slag activator being present.

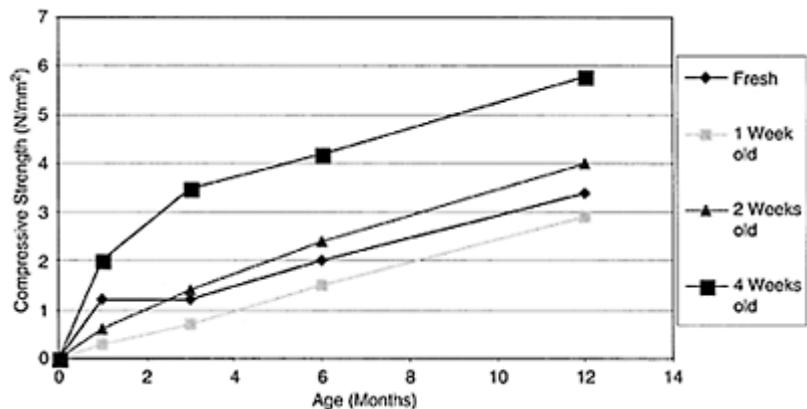


Figure 4a. Development of stiffness for blend 2.

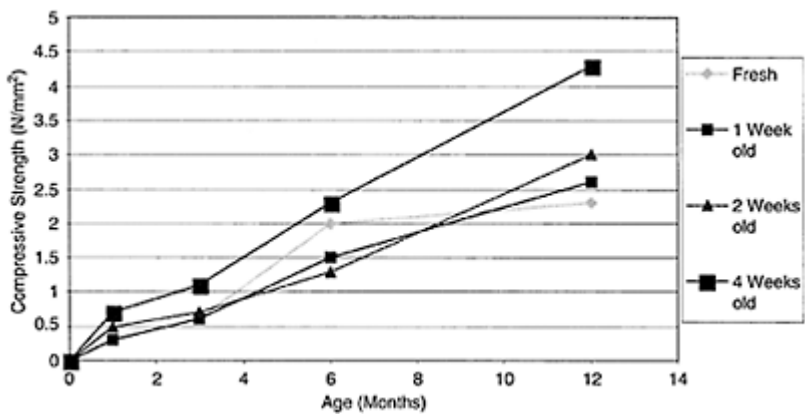


Figure 4b. Development of compressive strength for blend 2.

4.4 Blends 3 and 4

Blend 3 (Figures 5a and 5b) shows similar trends to blends 1 and 2 but the overall stiffness and strength results are again lower due to less slag binder (10%) being present. The 4 week old mix is again giving the best result.

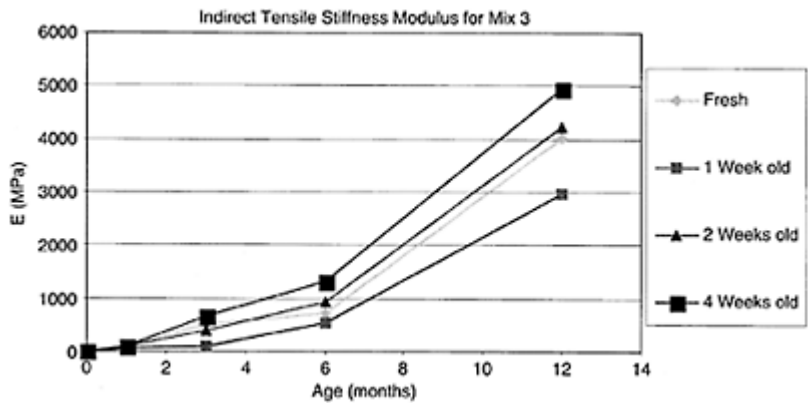


Figure 5a. Development of stiffness for blend 3.

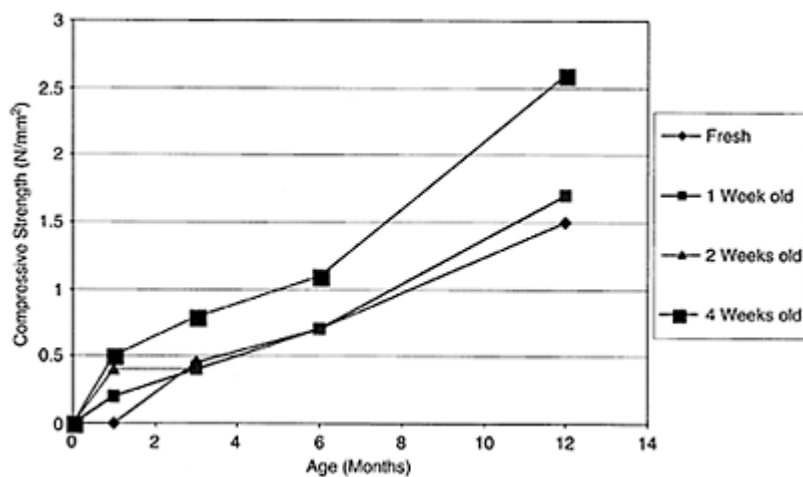


Figure 5b. Development of compressive strength for blend 3.

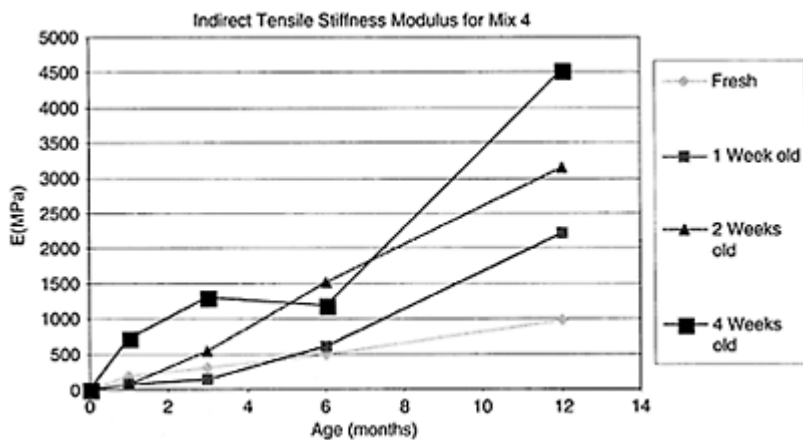


Figure 6a. Development of stiffness for blend 4.

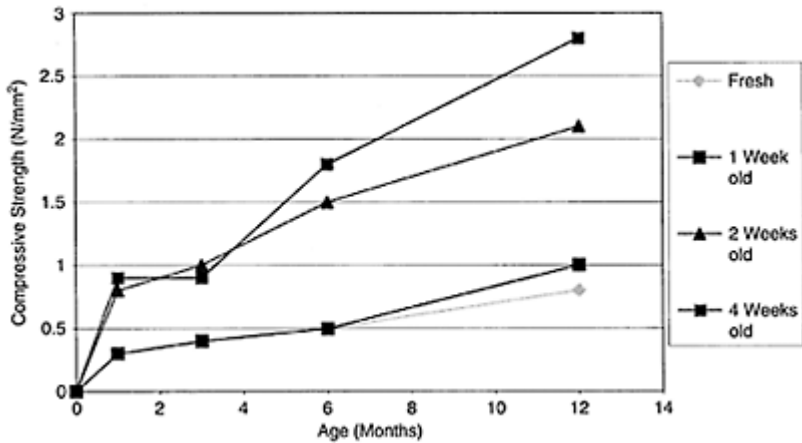


Figure 6b. Development of compressive strength for blend 4.

Blend 4 (Figures 6a and 6b) again shows similar trends to the previous blends with a further reduction in overall stiffness due to less slag binder (5%) and activator (5%), however the compressive strengths are similar to those of blend 3.

5 CONCLUSIONS

This work provides further evidence that slag bound materials can deliver levels of performance that should encourage their use in pavement bases.

The results indicate that the “control mix” using limestone aggregates performed in accordance with expectations with the added benefit that storing the material for up to four weeks prior to compaction still achieves good strength and stiffness development.

When all of the limestone is replaced with recycled material (blend 1) the compressive strength falls by about 50% whereas the stiffness remains broadly in the same range as for the Limestone control mix.

For reasons not yet fully understood there is compelling evidence that storing SBM containing recycled material for up to 4 weeks prior to compaction actually improves performance. This phenomenon is not witnessed in the Limestone SBM mixes.

Blends 2, 3 and 4 show the effect of reducing the binder and activator content on performance. As the binder content decreases the stiffness and strength decreases with the 4 week old mix consistently out performing the remaining mixtures.

REFERENCES

Ghazireh N. and Robinson H.L., 2001, “The Echline Experimental Road Pavement Trial—Environmental Assessment.” *Third BGA International Geo-environmental Engineering Conference*, Edinburgh.

- Robinson H.L., "Slag Bound Material (SBM)—For Flexible Composite Pavement Design" (1999). *Proc.3rd European Symposium, Performance and Durability of Bituminous Materials and Hydraulic Stabilised Composites*, Leeds, April, pp. 303–310.
- Juckes, L.M. and Thomas, G.H. (1988). "Self-Binding Composites of Blast Furnace and BOS Slag for Road Construction" (Unpublished paper).

Load-deformation behavior of fly-ash and bottom-ash capping and fill layers based on FWD deflection measurements

M.S.Hoffman

YONA Engineering Consulting & Management Ltd., Haifa, Israel

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ABSTRACT: Both Fly-Ash (FA) and Bottom-Ash (BA) obtained as a by-product from coal combustion for electricity generation have been used in Israel for road construction since 1997. Fill and capping layers using FA and BA have been implemented in several road projects amounting to over 1,000,000 tons by the end of 2003. Falling Weight Deflectometer (FWD) deflection basin measurements were made at different stages of Road 57 construction using FA, BA, and “conventional” granular materials to: a) Detect any weak spots and assess the sections homogeneity; b) Calculate load-deformation parameters (moduli, stiffness, etc.); c) Evaluate the relative density of the fill materials; and d) Compare between conventional and FA/BA pavement structures at different stages of the road construction, up to the AC layer level. The major findings of the study are presented, together with a discussion on the limitations of linear elasticity and continuum mechanics to correctly characterize the load-deformation behavior of particulate media. The need for harmonization of testing equipment and methods for the structural control and monitoring of the foundation, capping and granular layers is emphasized.

1 INTRODUCTION

The use of deflection measurements for the structural evaluation, monitoring and quality control of subgrade soils (foundation level) and granular (unbound) layers has become a popular scheme in the last 10 years or so [Cost Action 336 (2000), Stubstad (2002)]. This trend is rather new compared to the well-established and popular backcalculation methods used for the structural evaluation of flexible and rigid pavements at the “finished level”, which are being used for over 25 years [SPT 1375 (2000)].

The nondestructive structural evaluation of a road foundation (subgrade), capping layers and road bases in-situ, as the road construction progresses, is quite appealing as it

provides more meaningful information compared to the simple index tests normally used for QC/QA [Fleming et al (2000)]. It is common practice to model the foundation and the granular layers using the linear elastic homogeneous and isotropic half-space known as the Boussinesq model. Measured deflections are used to backcalculate stiffness or surface modulus [Cost Action 336 (2000), Groenendijk et al (2000)], or an equivalent k-value [Kamiura et al (2000)]. Attempts have been made to interpret deflections to evaluate degree of compaction [Cost Action 336 (2000), Van Gurp et al (2000)], or density [Rogers et al (2000)]. French (1997) and German [RStO 86 (1989)] pavement design and QC/QA practices require the execution of a field load-deformation test to verify that a minimum bearing capacity or elastic modulus value at the foundation and/or capping layer level is obtained before additional layers can be laid down. Despite the growing use of these schemes, there seems to be little or no theoretical support for most of the load-deformation criteria suggested, and this is due to the complex mechanical behavior of soils and granular materials.

Deflections are normally measured with static and preferably impulse type devices. The Falling Weight or Heavy Weight Deflectometer (FWD/HWD), the TRL Foundation Tester (TFT), the German Dynamic Plate Test (GDP), the Prima 100 handheld or mini-FWD, the French Dynaplaque are among the most popular [Fleming et al (2000), Kamiura et al (2000)]. These devices use circular loading plates with radii varying from 90 to 450 mm, and different positioning and number of geophones to measure surface deflections. Unfortunately, different testing equipment and loading modes lead to different results that are difficult to compare [Hoffman (1983)]. Nevertheless, these nondestructive evaluation techniques are particularly useful when new materials are used, and their structural behavior over time is not yet known.

This paper presents the major findings of a study designed to evaluate the load-deformation behavior of locally unknown Fly Ash and Bottom Ash fill and capping layers, and compare it to conventional materials. The study comprised FWD deflection basins measurements and interpretation at 3 stages of the road construction: a) at foundation or capping layer level, b) at the subbase level, and c) at the finished AC level. The data collected in this study were useful to explore some of the limitations of continuum mechanics when applied to particulate or discrete media.

2 STUDY GOALS AND DESCRIPTION

The specific goals of the study were:

- a) To assess the homogeneity of the non-conventional materials sections and detect any weak spots or irregularities.
- b) To backcalculate stiffness and moduli of the different materials at different construction stages as construction progressed.
- c) To analyze the compaction or relative density patterns of the conventional and non-conventional fills using up to 8 consecutive FWD mass drops at the same testing point.
- d) To compare the overall structural behavior and estimate the future performance of the conventional and non-conventional sections at the AC level.

2.1 Study description

The study was done during the reconstruction and widening of Road 57, east to the city of Natanya. It comprised 3 construction stages at the same road sections as the project progressed: a) Stage 1, on October 2000, at the FA, BA and compacted subgrade levels, b) Stage 2, On November 2000, after the FA and BA layers were covered with sub-base or capping layers, and at the sub-base level of a “conventional” section, and c) Stage 3, On November 2001, at the top of the AC layer.

The sections ranged from 120 to 240 meter long. FWD measurements were made at 20-m intervals and at several lateral lanes across the sections. The study also evaluated load effects in stages 1 and 2 using 2 levels of loading.

2.2 Study sections and stages

Table 1 describes the study cross-sections and stages.

2.3 Soil and materials

2.3.1 Subgrade soil

The subgrade soil alternated between a lean (A-6) to a heavy clay (A-7-6) with a liquid limit of 40% to 55%, a plasticity index of 15% to 30%, and 45% to 95% passing the 75 μ m sieve. The laboratory California Bearing Ratio (CBR) ranged between 6% and 8%.

2.3.2 Fly Ash

Fly ash (FA) was hauled from a stockpile at the electric power plant containing ashes from different coals normally burnt in Israel. The very fine, grayish powder-like material was non-plastic, with 85% to 95% passing the 75 μ m sieve. Maximum dry densities (Modified Proctor energy) of

Table 1. Outline of the study stages and cross sections.

Stage No.	Date	Section ID	Thickness of indicated layer, mm			
			Coal ash fill	Other fill/ Capping	Sub-base	Base Asphalt
1		A-1	Compacted Clayey Subgrade			
Oct. 2000		B-1	1200 BA	–	–	–
		C-1	1600 FA	thin cover	–	–
2		A-2	–	450	180	–
Nov. 2000		B-2	1500 BA	–	180	–
		C-2	1600 FA	400	–	–
3		A-3	–	450	380	170 200

Nov. 2001	B-3	1500 BA	–	180	170	150
	C-3	1600 FA	1000	180	170	150

1050 to 1200 kg/m³, and optimum water content of 28% to 38%. Laboratory CBR ranged from 12% to 20%. No major restrictions were imposed upon the FA other than limiting the loss on ignition (LOI) to 8%. When major changes in the FA characteristics were noted, a new set of laboratory density-moisture curves was performed for field quality control (QC).

2.3.3 Bottom ash

Bottom ash (BA) was also hauled from a stockpile at the power plant. The grayish coarse sand-like material was non-plastic, with 15% to 35% passing the 75 µm sieve. Maximum dry densities (Modified Proctor energy) of 1250 to 1350 kg/m³, and optimum water content of 15% to 25%. Laboratory CBR ranged from 20% to 35%. The LOI restrictions and the requirement of moisture-density curves were the same as for the FA.

2.3.4 Fill material and capping layers

Fill material and capping layers were limited to 100% passing the 75 mm size, 80% to 25% passing the 4.75 mm sieve, 0% to 25% passing the 75 µm sieve, liquid limit lower than 35%, plasticity index lower than 10%, and CBR greater than 20% within a 3% moisture range.

2.3.5 Other materials

Sub-base, crushed base and Hot Mixed Dense Asphalt Concrete met the requirements of the Israeli PWD Specifications.

3 RESULTS

3.1 HWD equipment

Deflections were measured with a Dynatest 8081 Heavy Weight Deflectometer. A 450 mm diameter plate was used for testing on unbound materials (stages 1 and 2) and a 300 mm diameter plate was used for testing at the AC level (stage 3), unless otherwise indicated. Seven sensors were used in all testing positioned at the center of the loading plate, and at 300, 600, 900, 1200, 1500 and 1800 mm away from the plate axis.

3.2 Maximum deflection and impact stiffness modulus (ISM)

The maximum deflection, D_0 , was measured with a single geophone located at the center of the loading plate. The values reported correspond to the third drop of the HWD loading mass, unless otherwise indicated. The Impact Stiffness Modulus (ISM) was

computed as the ratio of the applied HWD load to the maximum deflection D_0 and it is expressed in metric ton/cm. Since the differences

Table 2. Impact stiffness modulus (ISM), in ton/cm.

Statistic	Section A-1		Section B-1		Section C-1	
	2 to 3 ton load level	4 to 5 ton load level	2 to 3 ton load level	4 to 5 ton load level	2 to 3 ton load level	4 to 5 ton load level
Average ISM	30.7	28.9	28.2	31.7	52.8	52.8
Std. Dev.	6.3	4.4	2.2	2.1	9.4	8.1
CV (%)	20.5	15.2	7.8	6.6	17.8	15.3
Statistic	Section A-2		Section B-2		Section C-2	
	2 to 3 ton load level	4 to 5 ton load level	2 to 3 ton load level	4 to 5 ton load level	2 to 3 ton load level	4 to 5 ton load level
Average ISM	74.3	74.9	44.0	46.6	61.4	64.4
Std. Dev.	8.9	15.2	3.8	4.5	5.6	8.1
CV (%)	12.0	20.3	8.6	9.6	9.1	12.6
Statistic	Section A-3		Section B-3		Section C-3	
	7.5 ton level		7.5 ton level		7.5 ton level	
Average ISM	254.7		158.7		414.4	
Std. Dev.	17.3		11.9		24.0	
CV (%)	6.8		7.5		5.6	

in the load level are normalized within ISM, it is possible to make direct comparisons of the maximum deflection results, and detect non-linear behavior. Table 2 shows the average, the standard deviation and the coefficient of variation of ISM values measured at the study sections as construction progressed.

The following points are observed from Table 2:

- The compacted local soil (Section A-1) exhibits a moderate “softening” behavior (ISM decreases with increasing load) while the BA fill (Section B-1) exhibits a moderate “hardening” (ISM increases with increasing load). These two sections show a similar ISM value of about 30 ton/cm, which is almost 75% lower than the ISM obtained at the FA fill (Section C-1) of 53 ton/cm.
- As expected, stage 2 sections show higher ISM values than stage 1 sections. Since the reinforcement layers were not uniform among the sections with respect to stage 1, no direct comparisons can be made. It is worth noting that the BA fill was “reinforced” by just 180 mm of granular subbase and showed an ISM increase of almost 50%

(Section B-2). The FA fill, that was “reinforced” with a thin cover of capping material showed an ISM increase of about 20% (Section C-2).

- Stage 3 ISM values were determined at the completed AC level. Again, since the pavement cross sections were not uniform, no direct comparisons can be made. All ISM values are high, however, and well within the norm for flexible pavements in Israel. It is worth noting the very high ISM value obtained at the FA fill section (C-3) of over 400 ton/cm. It is suspected that the FA, that contained about 10% CaO (lime), has undergone a self-cementing pozzolanic reaction, and this is the reason for the high ISM. This point is the subject of a follow-up study under preparation.

3.3. Deflection basin AREA

The deflection basin AREA was computed according to the following expression [Hoffman and Thompson (1982)]:

$$AREA = 6(1 + 2D_1/D_0 + 2D_2/D_0 + D_3/D_0) \quad (1)$$

Table 3. AREA of Deflection Basin, in inches.

	Section A-1		Section B-1		Section C-1	
Statistic	2 to 3 ton load level	4 to 5 ton load level	2 to 3 ton load level	4 to 5 ton load level	2 to 3 ton load level	4 to 5 ton load level
Average AREA	11.0	11.0	12.2	13.0	10.7	10.9
Std. Dev.	0.9	0.7	0.5	0.6	1.0	0.7
CV (%)	8.2	6.4	4.0	4.6	9.3	6.4
	Section A-2		Section B-2		Section C-2	
Statistic	2 to 3 ton load level	4 to 5 ton load level	2 to 3 ton load level	4 to 5 ton load level	2 to 3 ton load level	4 to 5 ton load level
Average AREA	13.1	13.9	15.0	15.1	11.9	12.2
Std. Dev.	0.6	1.1	0.9	0.8	0.9	1.2
CV (%)	4.6	7.9	6.0	5.3	7.6	9.8
	Section A-3		Section B-3		Section C-3	
Statistic	7.5 ton level		7.5 ton level		7.5 ton level	
Average AREA	22.8		22.4		17.3	
Std. Dev.	0.8		0.6		0.9	
CV (%)	3.5		2.7		5.2	

where AREA=deflection basin AREA expressed in inches; and D_0 , D_1 , D_2 , and D_3 =HWD deflections at 0, 300, 600, and 900 mm away from the plate axis, respectively.

By definition, the AREA has a maximum possible value of 36" (when all sensors show equal deflections). Higher AREA values correspond to "stiffer" pavement structures with respect to their subgrade (higher modular ratio E_p/E_{subgrade}).

Table 3 shows the deflection basin AREA (average, standard deviation and coefficient of variation).

The following points are noteworthy from Table 3:

- Stage 1 AREA values are similar for the 3 sections and vary from 11 to 13 inches. There seems to be little influence of the load level on the AREA values.
- Stage 2 AREA values increase by 10% to 20% for all sections compared to Stage 1. The BA fill (Section B-2) shows the highest AREA.
- Stage 3 AREA values are significantly higher. Based on the two-layer linear elastic model (known as the Burmister model), "loaded" with the FWD 300 mm circular plate, it can be shown that an approximate relationship can be established between the AREA and the modular ratio, E_p/E_{sg} for pavement thicknesses of 250 to 750 mm and modular ratios between 2 and 50. This relationship is of the form:

$$E_p/E_{\text{sg}} = 0.1256e^{0.2095 \text{ AREA}} \quad (2)$$

where E_p =Equivalent modulus of elasticity of the pavement structure; E_{sg} =Equivalent modulus of elasticity of the subgrade; and AREA=Deflection basin AREA expressed in inches.

Using equation (2) and the AREA values obtained at stage 3, it is seen that sections A-3 and B-3 show a modular ratio of 14, while section C-3 shows a modular ratio of about 5. On the other hand, it is apparent that section C-3 has a much higher " E_{sg} ", as suggested by the high ISM obtained (see Table 2). Therefore, the lower modular ratio in Section C-3 is due to the high subgrade support and not due to a pavement structure deficiency, as further discussed later.

3.4 Sections homogeneity

It was hypothesized that the use of stockpiled FA and BA from different sources could result in heterogeneous sections. Heterogeneity could also result from the fact that these materials were placed and compacted using normal equipment, but they may require other methods of construction or equipment.

"Homogeneity" was analyzed in two ways: a) by observing the coefficients of variation (CV) of the deflection basin parameters within each section for all testing lanes. It was assumed that a CV greater than 25% would indicate a heterogeneous behavior, and b) by performing an Analysis of Variance (ANOVA) to test the hypothesis that all lateral adjacent lanes showed identical deflection basin parameters at a confidence level of 95%. This place is too short to show all the results, but the major findings of the homogeneity analyses are listed below:

- *Stage 1 sections:* The ISM-CV values never exceeded 21%, and the AREA CV values were all below 10% for all 3 sections. The B-1 section showed the highest

homogeneity with the lowest CV values. ANOVA showed differences in ISM averages among lateral lanes in some areas of B-1 and C-1. It was believed these differences were due to the fact that parts of the fills were trafficked during construction and others not.

- *Stage 2 sections:* Again, the ISM-CV values were below 21% for all sections, and the AREA CV values were below 9%. The B-2 section showed the highest homogeneity with the lowest CV values. The same ANOVA differences were detected at trafficked and non-trafficked areas of C-2.
- *Stage 3 sections:* The ISM and AREA CV values never exceeded 9% for all sections, showing a high level of homogeneity. ANOVA did not detect any differences among the deflection basin parameters across the sections.

It may be concluded that the FA and BA materials resulted in pavement structures (Sections B and C) as homogeneous as Section A made with conventional materials. The use of normal construction procedures and equipment for FA and BA layers produced acceptable results.

3.5 Seating factor

A seating factor (SF) was calculated at selected points of stage 2 sections according to the approach proposed by van Gurp et al. (2000). The goal was to investigate the feasibility of relating degree of compaction to deflection parameters. HWD deflections were measured for 8 consecutive drops at the testing point (instead of 10 in the reported study), at 2 load levels, and the SF was calculated according to the following expression:

$$SF = 1 - \frac{d_7 + d_8}{d_1 + d_2} \quad (3)$$

where SF=Seating factor; and d_i =maximum (center plate) HWD deflection at drop i .

The hypothesis is that a well compacted foundation would exhibit close to zero SF values, as all d_i values would be identical, and the right hand factor of equation (3) would be close to 1. On the other hand, an increasing SF value would indicate a “soft” foundation that is further compacted with every successive drop of the HWD mass [low (d_7+d_8) compared to (d_1+d_2)].

Table 4 shows the SF average, standard deviation and coefficient of variation determined at 4 points of each of Stage 2 sections at 2 load levels. The table also shows the total percentage decrease in D_0 between drop 1 and 8, and the relative change between drop 1 to 3 and drop 3 to 8.

The SF analysis suggests the following observations:

- Section B-2 exhibits significantly lower SF values than the other 2 sections and a much higher homogeneity as expressed by its lower CV. Based on the SF hypothesis, section B-2 should have a higher level of compaction compared to the other 2 sections. However, conventional in-situ compaction data do not support this observation as no major differences in degree of compaction were found among the sections. As noted by van Gurp et al (2000), the SF is not a

Table 4. Seating factor (SF) on stage 2 sections.

Statistic	Section A-2		Section B-2		Section C-2	
	2 to 3 ton load level	4 to 5 ton load level	2 to 3 ton load level	4 to 5 ton load level	2 to 3 ton load level	4 to 5 ton load level
SF Average	0.27	0.27	0.11	0.13	0.32	0.19
Std. Dev.	0.075	0.130	0.008	0.019	0.080	0.050
CV (%)	27.3	48.7	7.3	14.9	24.9	28.9
Total D_0 change 1÷8	34.5%	35.3%	15.1%	18.2%	39.7%	25.9%
Relative change 1÷3	68%	75%	75%	81%	75%	77%
Relative change 3÷8	32%	25%	25%	19%	25%	23%

satisfactory indicator for prediction of degree of compaction. The extensive LTPP data analysis study (Stubstad [2002])) also showed poor correlations between density and deflection basin parameters.

- Sections A-2 and C-2 show a higher SF heterogeneity reflected by CV values of over 25%. Other deflection basin parameters had not shown such high levels of variability.
- No load level effect on SF is evident from the data. Similar SF values are obtained at the 2 load levels of testing.
- The total change in D_0 between HWD drop 1 and 8 is quite significant reaching over 30% in sections A-2 and C-2. About 75% of the D_0 decrease occurs between drop 1 and 3 and only about 25% between drops 3 and 8. These changes contrast with the local experience on dozens of pavements at the finished AC level where the change in D_0 between HWD drop 1 and 3 never exceeds 10% with most cases below 5%.
- Not shown in Table 4, the deflection basin AREA increased with increasing number of drops by 15% in sections A-2 and C-2 and by less than 10% in section B-2.
- Also not shown in Table 4, deflections at 300 mm decrease between 2% and 15% between drop 1 and 8, while deflections at 900 mm show an increase of up to 10% with increasing number of drops. No major conclusions are suggested as data are limited and deflection values are low and close to the geophones accuracy range.

The SF data illustrate the complex mechanical behavior of unbound materials. It is believed that factors like the material's cohesion, internal friction, moisture content, gradation etc. also affect the SF values. This is probably why a single and meaningful correlation between SF and degree of compaction may not be feasible. The time that elapses between the conventional compaction monitoring and the HWD testing, and whether the section is trafficked or not, also seem to affect the SF values. Harmonization of equipment and testing procedures could help reducing the many variables involved, and thus improve the significance of the information being analyzed and compared.

3.6 Modulus of elasticity

Equivalent moduli of elasticity or “surface modulus” of the unbound stage 1 and 2 sections were backcalculated from surface deflections on the homogeneous elastic half-space model. The moduli E_r were backcalculated using HWD deflections measured at 0, 300, 600 and 900 mm from the load axis to get an insight into the load-deformation behavior at points farther away from the load plate center. Theoretical deflections under a uniform circular load at radial distances different from zero were calculated based on the tables developed by Ahlvin and Ulery (1962). A Poisson’s ratio of 0.5 was assumed for all materials. The general expression for the computation of vertical deflections at a radial distance “r” is:

$$D_r = \frac{1.5pa}{E_r} \left(\frac{H}{2} \right) \quad (4)$$

where D_r =Deflection at radial distance “r”; p =uniform loading pressure; a =radius of loading plate; and H =Ahlvin and Ulery deflection coefficient as a function of “a” and “r”.

Figure 1 shows the variation of the backcalculated E_r/E_0 as a function of the distance “r” for one typical testing lane at the Stage 1 sections. The modulus E_0 obtained at $r=0$ were: a) $E_0=58$ MPa at section A-1; b) $E_0=61$ MPa at section B-1; and c) $E_0=96$ MPa at section C-1.

Figure 1 shows that the modulus of elasticity is not constant with “r”. It increases by a factor of 2 to 3 at $r=900$ mm at all sections, i.e., these sections do not behave as a homogeneous linear elastic half-space. While this conclusion is not new, it emphasizes the need to search for better models to characterize the load-deformation behavior of soils and granular materials. The fact the equivalent linear elastic “surface” modulus increases with increasing “r” could be explained on the basis of stress dependency, lack of shear transfer, lack of continuity, etc. The E_0 values obtained, all above 50 MPa, are within the expected values at the foundation level. The C-1 section has a much higher E_0 than the other 2 sections. Based also on the E_0 parameter, it is concluded that the FA and BA sections behave equally or better than the conventional section.

Figure 2 shows the variation of E_r/E_0 as a function of the distance “r” for one typical testing lane at Stage 2 sections. The modulus E_0 obtained at $r=0$ were: a) $E_0=163$ MPa at section A-2; b) $E_0=91$ MPa at section B-2; and c) $E_0=137$ MPa at section C-2. It is seen that the moduli increased with respect to stage 1 by a factor of 1.4 to 2.8. It is also seen that the deviation from the $E_r/E_0=1$ line has moderated in sections A-2 and B-2 to less than $\pm 20\%$. Only C-2 still shows a marked departure from the half-space homogeneous model with E_r values up to 2 times E_0 at $r=900$ mm. The FA and BA sections continue to show a structural behavior comparable to the conventional section.

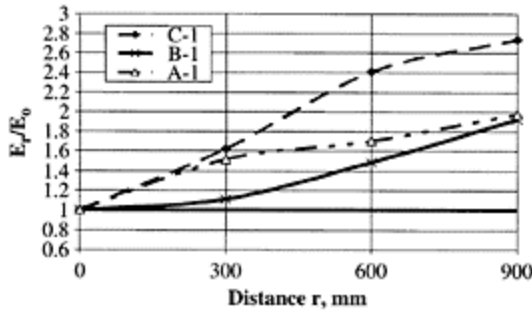


Figure 1. Variation of backcalculated surface modulus of elasticity with radial distance from the HWD plate axis at selected Stage 1 sections.

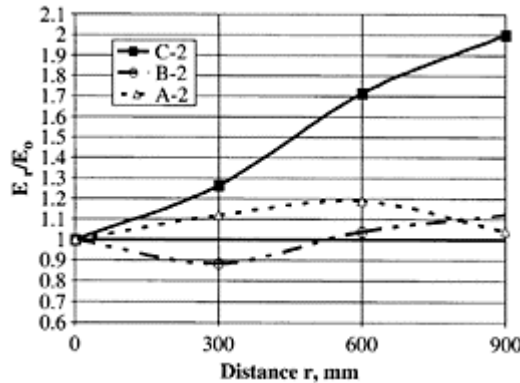


Figure 2. Variation of backcalculated surface modulus of elasticity with radial distance from the HWD plate axis—Stage 2 sections.

It may be concluded that the HWD seating effects, which in turn seem to be dependent on the unbound material density, cohesion, gradation, friction, moisture content, etc., and the non-compliance of these materials to the basic assumptions of continuum mechanics and linear elasticity, raise a question concerning the significance of a reported “surface” modulus of elasticity. Again, the lack of harmonization of equipment and testing procedures further reduces the usefulness of this approach.

It has been suggested that a model dealing with the forces and displacements (instead of stresses and strains) on the individual grains in a particulate medium could improve the characterization of unbound materials [Ullidtz (2002)]. These models, known as Distinct or Discrete Element Models (DEM) incorporate into the analysis the grain size

distribution of the particles, the shape of the grains, and the degree of compaction. These models require large amounts of computer memory to run and are still at the initial implementation stages. They could provide a better insight into the load-deformation behavior of unbound materials, in general, and of FA and BA in particular.

3.7 Structural evaluation of the completed sections

The structural evaluation of the completed sections at Stage 3 consisted of 2 parts: a) Backcalculation of pavement layers and subgrade moduli using the well-known MODULUS program [Michalak and Scullion (1995)], and b) Evaluation of the effective Structural Number and Subgrade Modulus using the YONAPAVE method [Hoffman (2003)]. Table 5 shows the results of these analyses.

From Table 5 it is noted that:

- The 3 sections exhibit different effective subgrade support based on the two schemes. While the comparison between the two models of analysis is beyond the scope of this paper; it is worthwhile noting that the simple YONAPAVE method and the more rigorous MODULUS model give similar subgrade moduli. In the MODULUS analysis, the “subgrade” modulus represents the support below the total pavement thickness of 750 mm in Section A3, and 500 mm in Sections B3 and C3. Pavement or layer thicknesses are not needed for YONAPAVE back-calculations.
- The unusually high subgrade modulus obtained at section C3 is believed to be due to the self-cementation of the Fly Ash platform below the capping layer. Because of the suspected cementation of the FA, it was decided to incorporate a 1000-mm thick “buffer” layer of capping material to avoid cracking at the pavement surface due to cementation of the FA fill.
- All moduli values are within normal expectations. The AC moduli reflect the AC temperatures at the time of testing that ranged from 27°C to 38°C, with the higher AC moduli generally corresponding to the lower temperatures.
- The A3 section, with the thickest pavement structure, shows the highest effective structural number (SN). The B3 and C3 sections show lower effective SN values, concomitant with their thinner pavement structures.

Table 5. Structural Evaluation of the Completed Sections (Stage 3).

Section	YONAPAVE		MODULUS				
	30th percentile Effective E_{SG} , MPa	30th percentile Effective SN	AC layer		Granular layer		Subgrade
			h_{AC} , mm	Average E_{AC} , MPa	h_{GR} , mm	Average E_{AC} , MPa	Average E_{SG} , MPa
A-3	177	6.2	200	4400	550	430	165
B-3	114	4.6	150	5220	350	210	116
C-3	641	4.1	150	5700	350	750	601

- Using the evaluated effective SN and E_{SG} , and based on the AASHTO (1993) method of pavement design, it is found that all 3 sections are expected to withstand well over the 20 million 80 kN ES AL's originally designed.
- After 2 years of service since November 2001, all sections are showing no signs of premature distress. The FA and BA fill and capping layers seem to be performing as well as the conventional sections.

4 CONCLUSIONS

- The HWD based Road 57 study shows that both Fly Ash (FA) and Bottom Ash (BA) fill and capping layers provide adequate support and similar to conventional fill and granular capping materials.
- Using normal construction equipment and procedures, FA and BA layers produce pavement sections as homogeneous as conventional materials.
- FA, BA, and unbound materials are highly sensitive to increasing number of HWD drops, and there seems to be no correlation between a seating factor and the relative density. The deflection under the HWD plate depends not only on density but also on cohesion, internal friction, degree of compaction, moisture content, gradation, etc.
- FA, BA, and unbound materials do not comply with the basic assumptions of a homogeneous linear elastic continuum that can be characterized by a single Modulus of Elasticity. Distinct or Discrete Element Models (DEM) seem to be better suited to characterize the load-deformation behavior of particulate media.
- Without harmonization of testing equipment and procedures, there is little significance to a single, equivalent, surface modulus of elasticity or stiffness value of the soil or unbound layer.

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REFERENCES

- AASHTO Guide for Design of Pavement Structures* 1993. Published by the American Association of State Highway and Transportation Officials, Washington DC.
- Ahlvin, R.G., and Ulery, H.H. 1962. Tabulated Values for Determining the Complete Pattern of Stresses, Strains, and Deflections Beneath a Uniform Circular Load on a Homogeneous Half Space. *Highway Research Board Bulletin* 342, Publication 1025, Washington D.C., 1962, pp 1–13.
- Cost Action 336. 2000. *Use of Falling Weight Deflectometers in Pavement Evaluation*. Annex H. FWD Foundation Test.

- Fleming, P.R., Frost, M.W., and Rogers, C.D. F. 2000. A Comparison of Devices for Measuring Stiffness In-Situ. *Unbound Aggregates in Road Construction*. Edited by A.R.Dawson, A.A.Balkema Publishers, pp 193–200.
- French Design Manual for Pavement Structures. 1994. Translation of the December 1994 French Version of the Technical Guide “*Conception et Dimensionnement des Structures de Chaussee—Guide Technique*”, Published by LCPC and SETRA in May 1997.
- Groenendijk, J. van Haasteren, C.R. and van Nierek, A.A. 2000. Comparison of Stiffness Moduli of Secondary Road Base Materials Under Laboratory and In-Situ Conditions. *Unbound Aggregates in Road Construction*, Edited by A.R. Dawson, A.A. Balkema Publishers, pp 201–208.
- Hoffman, M.S., and Thompson, M.R. 1982. Comparative Study of Selected Nondestructive Testing Devices, *Transportation Research Record 852*, TRB, Washington, DC.
- Hoffman, M.S. 1983. Loading Mode Effects on Pavement Deflections. *ASCE Journal of Transportation Engineering*, Vol. 109, 5, pp 651–668.
- Hoffman, M.S. 2003. Direct Method for Evaluating the Structural Needs of Flexible Pavements Based on FWD Deflections. Accepted for Publication in the *Transportation Research Record (TRR)*, *Journal of the TRB*, Washington DC.
- Kamiura, M., Sekine, E., Abe, N., and Maruyama, T. 2000. Stiffness Evaluation of the Subgrade and Granular Aggregates Using the Portable FWD, *Unbound Aggregates in Road Construction*, Edited by A.R. Dawson, A.A.Balkema Publishers, pp 217–223.
- Michalak, C.H., and Scullion, T. 1995. *Modulus 5.0: User s Manual*. Texas Transportation Institute, College Station, Texas.
- Rogers, C.D. F., Fleming, P.R., and Frost, M.W. 2000. Stiffness Behaviour of Trial Road Foundations. *Unbound Aggregates in Road Construction*, Edited by A.R.Dawson, A.A.Balkema Publishers, pp 231–238.
- RStO 86. 1989. *Richtlinien fur die Standardisierung des Oberbaues von Verkehrsflächen*, Ausgabe 1986, Erganzte Fassung 1989.
- STP 1375. 2000. *Nondestructive Testing of Pavements and Backcalculation of Moduli*: Third Volume. S.Tayabji and E.Lukanen Editors, ASTM.
- Stubstad, R.N. 2002. *LTTP Data Analysis: Feasibility of Using FWD Deflection Data to Characterize Pavement Construction Quality*. NCHRP Web Document 52 (Project 20–50[9]).
- Ullidtz, P. 2002. Analytical Tools for Design of Flexible Pavements. Keynote address at the ISAP, Copenhagen.
- Van Gurp, C., Groenendijk, J., and Beuving, E. 2000. Experience with Various Types of Foundation Tests. *Unbound Aggregates in Road Construction*, Edited by A.R.Dawson, A.A.Balkema Publishers, pp 239–246.

Stabilisation

Laboratory and in situ evaluation of stabilisation of limestone aggregates using lime

P.Hornych

Laboratoire Central des Ponts et Chaussées, Nantes, France

O.Hameury

Laboratoire Regional des Ponts et Chaussées d'Aix en Provence, France

Michel Kergoët

*Laboratoire Regional des Ponts et Chaussées de l'Est Parisien, Melun,
France*

Daniel Puiatti

Lhoist Group, France

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ABSTRACT: A study was conducted in France, in the Region of Seine et Marne, to evaluate the potential for improvement of soft limestone granular materials, widely available in this region, by lime treatment. Three experimental sections of low traffic pavements were built, with 3 different limestone base course materials. Each section was divided in two parts, one treated by addition of 1% of quicklime and one untreated. These sections were followed up by deflection measurements and FWD tests. Laboratory tensile strength and modulus measurements were performed on the same materials, after different curing times. For all three limestones, the study indicated that the addition of 1% of quicklime leads to a very significant increase in performance, due to several mechanisms, including: lime hydration, leading to moisture reduction and precipitation of calcite; carbonate cementation due to reaction of lime with carbon dioxide to form calcium carbonate that coats the aggregates. Values of elastic modulus and tensile strength of the materials increased considerably with time, leading to elastic moduli ranging from 4000 to 8000 MPa after 1 year. These improvements were verified on site, by FWD tests and deflection measurements: after 2 years, the modulus increased to 3000 to 5500 MPa for the lime-treated layers, compared with 600 to 700 MPa for the non-treated layers.

1 INTRODUCTION

During the late 1980s, the companies responsible for the construction of the Channel Tunnel on the French side were confronted with the following problem: how to construct, in a very short space of time, during the off-season, a dam for the spoil extracted during the excavation of the tunnel, to a height of 40 m, using 1.4 million m³ of saturated unworkable chalk. The problem was solved by an original solution: the systematic treatment of the chalk with quicklime at the rate of 1 to 3%, according to water content. The originality resides in the fact that if the use of quicklime in the treatment of soils is common practice in France (SETRA/LCPC, 2000) it had so far been limited to the case of argillaceous or clayey soils the water content and/or plasticity of which required reduction. The performance obtained at the Tunnel construction site raised questions. Studies were conducted to investigate the phenomenon, by the Laboratoire des Ponts et Chaussées of Lille (1992) and the University of Lille.

The explanation was given by Professor Paquet (1993) from the University of Lille, in the early 1990's: when hydrating, quicklime instantly reduces the water content of the chalk (thereby also causing a substantial increase of its load-bearing capacity) which, with the increase in the pH, leads to the precipitation of part of the calcium carbonate naturally present in the water. The precipitated carbonate and the hydrated lime cover the surface of the chalk aggregates. The hydrated lime gradually converts into carbonate upon contact with CO₂ (Chloup-Bondant & Evrard). The newly-formed crystals make a crust reinforcing the aggregates and then interlock, providing cohesion to the treated material. Treatment with lime leads to a permanent improvement of the properties of the soft limestone material, even in cases of prolonged immersion in water, through the low solubility of calcium carbonate in a basic medium (Chloup-Bondant & Evrard). Similar studies performed by different researchers in the USA led to the same conclusion and confirmed this interpretation (Bhuijan et al. 1995, Graves et al. 199).

Since the construction of the Channel Tunnel, millions of tons of chalk have been treated with lime, among other things for the construction of highways, the TGV Nord high speed train track (Terrassements & Carrieres 2002) and, more recently, the TGV Est between Reims and Chalons in the Champagne region (del Piero 2003). In all these cases, the main objective was a rapid and permanent increase of the-bearing capacity, for the construction of embankments.

2 PRESENTATION OF THE PROJECT

2.1 Objectives

During the 90s, an ambitious co-ordinated research programme called Materloc (Matériaux Locaux) was launched in France in order to make more rational use of natural resources in the field of construction and civil engineering. This programme included Materloc Calcaires, a programme specifically devoted to the upgrading of limestone materials, in particular the soft limestone materials producing aggregates whose properties hardly satisfy the requirements of road construction standards. The recent

experience with chalks treated with lime gave rise to the idea of having this programme include tests on lime-treatment of limestones.

2.2 Selected materials and experimental programme

The Paris region is penalised by the lack of good quality aggregates. Its resources come mainly from alluvial gravels and sands and soft limestone deposits. The porosity of these limestone materials, the occasional presence of clay and the solubility of the carbonate in an acid medium explain the unfavourable assessments of these materials (brittleness, sensitivity to water and freezing). Three of these limestone materials, originating from the Seine et Marne region, south-east of Paris, appeared as good candidates to evaluate the potential for lime stabilisation.

Like the chalks, the porous limestone materials from the quarries are often saturated with water containing dissolved calcium carbonate. Therefore, similar effects as for the chalk were expected: an instant modification of certain geotechnical properties, due to the formation of a crust closing the pores of the elementary granulate, and then the development of an internal cohesion of the granular mix, resulting in an increase of the service life of the construction.

As the phenomenon was not yet fully understood, its reproduction in the laboratory was uncertain; it was therefore decided to begin with the realisation of a trial section, on a local road. The limestone granular materials were used in the road base, under a thin bituminous wearing course. The granular materials were prepared in a mixing plant, including incorporation of the quicklime to a level of 1%. In a second phase, drawing on a better understanding of the phenomenon, this programme was completed by laboratory tests designed to study the influence of time and lime percentage on the mechanical properties of the treated materials.

3 PRESENTATION OF THE EXPERIMENTAL PAVEMENT SECTIONS

3.1 Characteristics of the experimental road

The experimental sections were built in 1992 on the RD 218 county road, running in a rural zone of the Seine et Marne department, south-east of Paris. This local road carries a traffic of 1600 vehicles per day, including some 40 heavy vehicles per day in each direction.

The experiment was conducted during the widening of the existing road. The works included the construction of lateral shoulders by laying 30 cm of granular material in order to widen the existing road, then the reinforcing of the whole road using the same granular material to a thickness varying between 20 cm on the pavement edge and 12 cm on the axis. A wearing course of bituminous concrete (0/10 mm) was then laid to a thickness of 6 cm. (Fig. 1). The end-to-end length of the 6 experimental sections was 1500 metres. Due to the variable cross section, all the measurements presented in the paper were made near the edge of the pavement, where the thickness of the granular materials is at a maximum (50 cm).

3.2 Characteristics of the materials

The three materials selected for this experiment were obtained from freshwater limestone material deposits from the tertiary era. They were issued from the quarries of Souppes-sur-Loing (ChâteauLandon freshwater limestone horizon), Ecuelles and Jouy-le-Châtel (Champigny freshwater limestone horizon). The aggregates were prepared by crushing after screening to remove the polluted fraction of the deposit. The main characteristics of the produced aggregates are set out in Table 1.

Laboratory tests on these limestones treated with 1% of lime, conducted during the preparation of the road, showed that this treatment improves the intrinsic properties of the aggregates (see table 1).

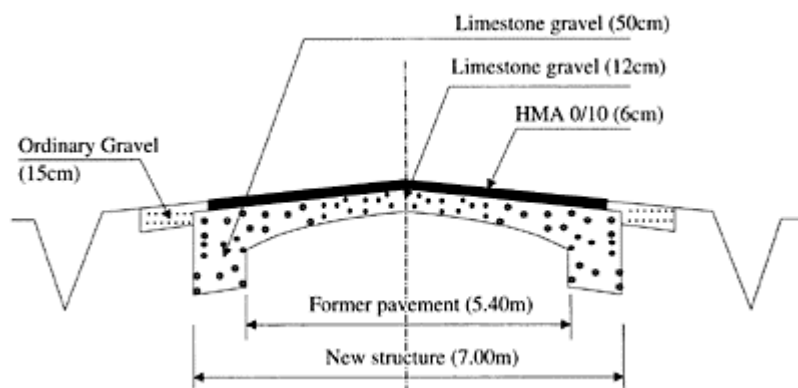


Figure 1. Structure of the experimental pavement sections on road RD 218.

Table 1. Characteristics of the aggregates of the three studied granular materials.

Quarries	Resistance of aggregates	Resistance of aggregates	Category of cleanliness of fines
	Untreated	Treated with 1% of lime	
Souppes-sur-Loing	LA: 28 to 29	LA: 28.5	a
	MDE: 31 to 35	MDE: 29	
Ecuelles	LA: 48	LA: 41	c
	MDE: 64 to 80	MDE: 5 1.5	
Jouy-le-Châtel	LA: 29 to 30	LA: 30	b/c limit
	MDE: 24 to 25	MDE: 24.5	

Section 1 Jouy-le-Châtel		Section 2 Ecuelles		Section 3 Souppes-sur-Loing	
1% lime	Untreated	1% lime	Untreated	1% lime	Untreated
0	250	500	750	1000	1250
1500 m					

Figure 2. Arrangement of the 6 experimental sections.

The main improvement concerns the resistance to abrasion (micro-Deval test) with a gain of 20 points in average for the Ecuelles material and 4 points for the Souppes material.

The positive effect of lime-treatment on the intrinsic properties of limestone aggregates supported the idea of adopting the lime-treatment parameter for the experiment. Six experimental sections were therefore prepared, corresponding to the three origins of the granular materials, untreated and treated with 1% lime, arranged as shown in Figure 2.

The 3 gravels were continuously graded 0/20 mm granular materials, in accordance with French standard NF P 98–129, obtained by the recomposition of at least two granular fractions, wetted and mixed in a plant (with the addition of lime in particular cases). The lime used was a cast quicklime in accordance with French standard NF P 98–101 (0/2 mm; $\text{CaO} > 80\%$; reactivity $\text{T60}^\circ\text{C} < 25 \text{ mn}$).

The limestone granular materials from Souppes and Ecuelles have high fines contents, so the formulation of these materials included the addition of 10% to 15% of clean alluvial sand in order to limit the quantity of fines and, consequently, the sensitivity of these gravels to water.

3.3 Construction of the experimental sections

The road was constructed in October 1992 during wet, rather cold weather. Dynamic plate load tests indicated a modulus for the in situ soil (fine argillaceous wet soils) of between 30 and 50 MPa. Areas of insufficient load-bearing capacity were cleared before laying the gravel.

The controls on the granular materials recomposed in the mixing machine revealed:

- grain sizes within specification range and with low dispersion;
- an average water content equal to Modified Proctor (target value) water content for the material from Ecuelles, and approximately 1% less than the optimum water content for the materials from Souppes and Jouy;
- a lime content very close to the target value (1% to 1.1%).

The in situ density was somewhat low (97% of the optimum of the Modified Proctor test in average, compared with a target value of 100%) with a rather high degree of dispersion.

4 FOLLOW-UP OF THE EXPERIMENTAL SECTIONS

The survey of the pavement sections was mainly based on deflection measurements and FWD tests, in order to determine the moduli of the granular layers. These tests were completed in 2001, i.e. after 9 years of service, with a distress survey and rut depth measurements.

4.1 FWD tests

FWD tests were conducted on the 6 experimental sections on 3 different dates: in December 1992 (2 months after construction), in February 1994 and in February 1995. The test results were analysed with the ELMOD back calculation software, to estimate the moduli for the 2 granular layers (base and subbase) and the subgrade soil. Table 2 shows the average modulus values obtained for the 6 sections at an applied load of 53 kN. Figure 3 shows the changes over time of the moduli of the

Table 2. Moduli of the granular layers (E_G) and subgrade soil (E_{soil}) determined from FWD tests.

Section	December 1992			February 1994			February 1995		
	EG		E_{soil}	EG		E_{soil}	EG		E_{soil}
	Base	Sub		Base	Sub		Base	Sub	
	MPa	MPa	MPa	MPa	MPa	MPa	MPa	MPa	MPa
Jouy+CaO	360	209	98	1399	812	183	2710	1572	220
Jouy	431	250	119	540	318	134	552	320	141
Ecuelles+CaO	453	263	173	2951	1712	307	5305	3077	363
Ecuelles	332	193	168	438	254	211	513	297	204
Souppes+CaO	466	270	152	1611	934	295	3166	1836	353
Souppes	472	274	113	608	352	116	661	383	136

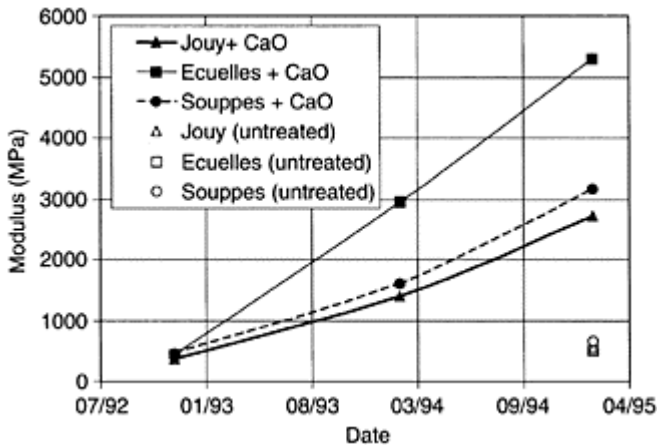


Figure 3. Evolution over time of elastic moduli of lime-treated base courses (FWD tests).

treated base courses. In December 1992, more or less identical results were obtained on the treated and untreated sections, ranging between 330 MPa and 470 MPa. However, 16 months later (February 1994) there was a very considerable increase of the moduli of the lime-treated granular materials (values of the order of 1500 MPa for the materials from Souppes and Jouy-Le-Châtel, and as much as 2950 MPa for the material from Ecuelles in base course), while the moduli for the non-treated sections had hardly changed. In February 1995, the moduli of the treated courses had practically doubled again, reaching values of between 2780 MPa and 5430 MPa in base course.

4.2 Deflection measurements

The deflection measurements were performed using a deflectograph with a reference load of 130 kN. Six sets of measurements were taken between 1992 and 2002. Figure 4 shows the changes of deflections on the 6 sections over this period. The results are in agreement with those of the FWD tests and demonstrate the highly favourable effect of lime-treatment, with a very significant reduction of deflection on the treated courses. However, the change of deflection over time is different for the three materials. For the gravel from Ecuelles, the deflections are practically stabilised after one year, while the gravel from Jouy-Le-Châtel takes 4 years to reach its maximum stiffness. The effect of lime-treatment seems to be permanent for all materials, since the deflections remain at the same low levels after 9 years.

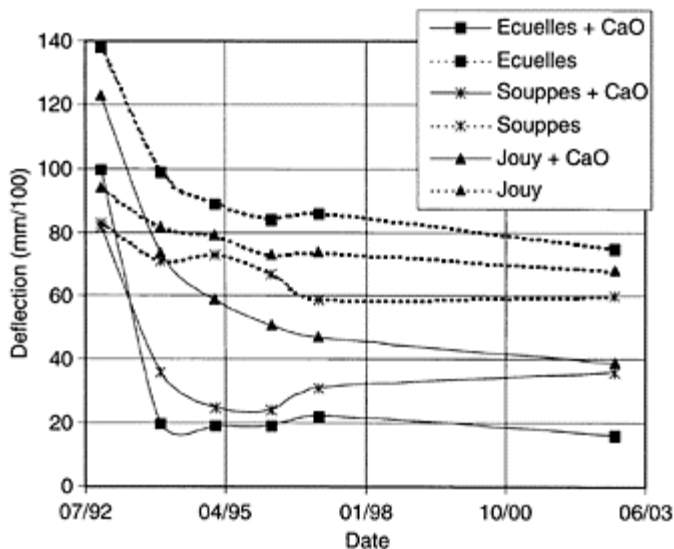


Figure 4. Changes of average deflection on the six experimental sections between 1992 and 2002.

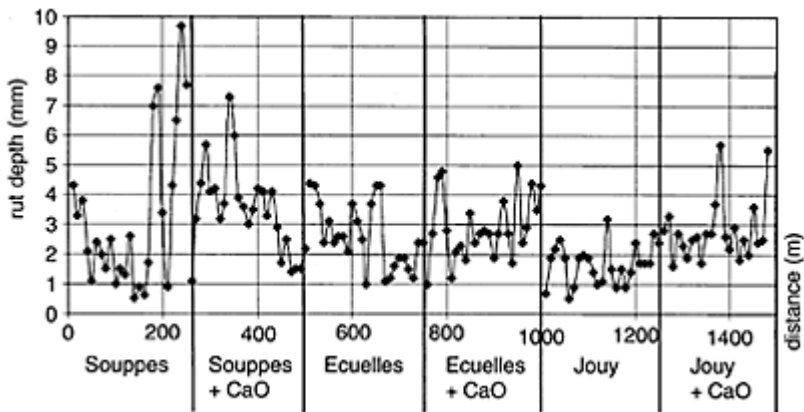


Figure 5. Results of rut depth measurements on the experimental sections.

4.3 Surface distress survey

In 2001, after 9 years of traffic, a detailed survey of the experimental pavement sections, including distress analysis and rut depth measurements, was performed. The results of the

rut depth measurements on the six experimental sections are shown on Figure 5. The rut depths are low (less than 5 mm on all sections, except for a length of about 70 m on the section built with the Souppes material, where rut depths attain 7 to 10mm), and there is no significant difference between the treated and untreated sections.

The distress analysis indicates only a few isolated longitudinal cracks, except for the sections built with the Jouy material, where longitudinal cracking is more important, covering approximately 10–15% of the length of the two sections (untreated and treated). Additional investigations are under way to determine the cause of this cracking. One explanation could be a lack of bonding between the bituminous wearing course and the base course. Again, no clear difference is observed between treated and untreated sections.

These results indicate that after 9 years, the pavement sections present no severe damage. This satisfactory performance, and the lack of difference between the treated and untreated sections are probably due to the low level of traffic. After 9 years, the cumulative traffic represents about 130000 standard French (130 kN) axle loads, which corresponds roughly to 20% of the expected design life for this type of pavement structure, according to the French pavement design method (SETRA-LCPC, 1994).

5 LABORATORY TESTS

5.1 *Test programme*

Following the very good in situ results obtained on the treated layers, a test programme was started at the Laboratoire Regional de Saint-Brieuc to evaluate the laboratory performances of these materials. Taking account of the level of moduli obtained in situ (up to 3 000 MPa after 1 year), it was decided to characterise the materials using measurements of elastic modulus and of indirect tensile strength. The tests were conducted on the three limestone materials, treated with different percentages of lime (0.5%, 1% and 1.5%) and after different sample storage times (2 months, 6 months and 1 year) in order to study the influence of these parameters.

The same procedure was used for the preparation of all the laboratory specimens: each material was divided into several fractions, and the mixes were recomposed in the laboratory in order to obtain the same grading curves as in situ. The lime used was identical to that used on the road. The specimens (diameter 160 mm, height 320 mm) were compacted using the vibro-compression method (standard EN 13286–52). They were all compacted at Modified Proctor reference water content and density. After preparation, the specimens were stored in water tight moulds, in a room with a temperature close to 20°C and a degree of humidity higher than 80%.

For each material, three identical specimens were prepared according to each method (lime dosage, curing time); these were first subjected to a simple compression test to measure the modulus, and after that to a diametrical compression test to determine the indirect tensile strength.

5.2 Simple compression tests

Given the particular behaviour of these lightly treated materials (low strength and stiffness at early age, non-linear behaviour similar to that of unbound granular materials), a special test procedure was used for measuring the moduli in compression. This consisted of performing three successive loading cycles from 0 to 100 kPa, at a loading rate of 0.01 MPa/s. The three cycles were necessary to stabilise the deformations of the specimen (during the first loading, greater deformations were observed, partly due to the irregularities of the end surfaces of the specimen). The elastic modulus was determined during the third loading, for an axial stress of 100 kPa.

The modulus values obtained for the three materials are given in Table 3. Each value represents the average of measurements on 3 specimens). The results show a considerable increase of the moduli over time, with values after 360 days varying between 3000 MPa and 9000 MPa, depending on the material and the percentage of lime. These values are very much higher than those of unbound granular materials, which only rarely exceed 1000 MPa. As in the pavement, the Ecuelles material was found to perform better than the other two materials.

5.3 Diametrical compression tests

Given the low tensile strength of the specimens, especially after 2 months in storage, the diametrical compression tests were conducted under deformation controlled loading, at a loading speed of 1 mm/min. The tensile strength values obtained for the various test conditions are given in Table 3. Each value corresponds to the average for 3 tests. Like the moduli, the tensile strengths increase significantly over time. However, the values after 360 days remain rather low, ranging approximately

Table 3. Result of modulus (E) and indirect tensile strength (Rt) tests on the three lime-treated granular materials.

Material	Lime dose %	60 days		180 days		360 days	
		E MPa	Rt MPa	E MPa	Rt MPa	E MPa	Rt MPa
Ecuelles	0.5	324	0.040	4700	0.087	7590	0.140
	1.0	428	0.078	5950	0.126	8227	0.187
	1.5	987	0.095	7627	0.159	9193	0.220
Souppes	0.5	897	0.078	2150	0.078	3033	0.109
	1.0	2013	0.105	3933	0.132	5457	0.142
	1.5	2373	0.123	4310	0.147	7033	0.170

Jouy-Le-Châtel	0.5	160	0.043	731	0.039	3180	0.093
	1.0	1810	0.071	1612	0.083	5300	0.128
	1.5	2607	0.103	3470	0.107	8700	0.172

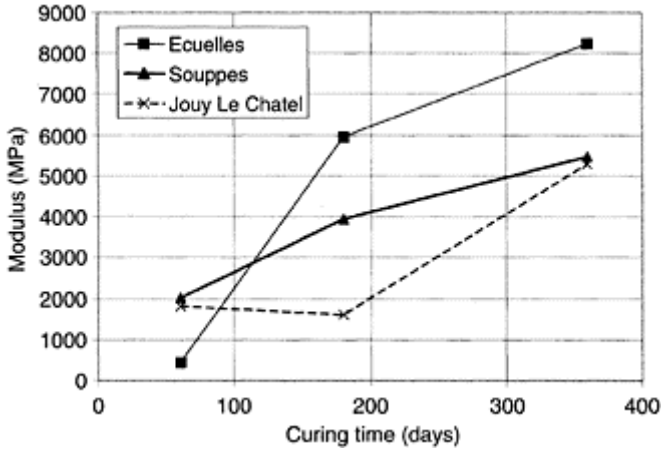


Figure 6. Influence of curing time on elastic moduli of the three treated materials (lime content 1%).

between 0.1 MPa and 0.22 MPa. These values are lower than the minimum values required for materials treated with hydraulic binders, in the French pavement design method (SETRA-LCPC, 1994).

5.4 Influence of lime content and curing time on mechanical performances

Figure 6 shows the evolution of elastic moduli measured on the three materials with curing time, for a lime content of 1% (identical to that used in situ). Differences appear between the three materials. The granular material from Ecuelles presents the highest moduli (8 200 MPa after 1 year) and is also the quickest-setting; the other two materials reach a modulus of approximately 5000 MPa after 1 year, with a slower rate of change. For the material from Jouy-Le-Châtel, the low values obtained after 180 days seem to be due to the longitudinal fracturing observed in some specimens, which reduced their mechanical performance.

The evolution of the indirect tensile strengths (R_t) of the specimens with curing time followed the same trends. Again, the Ecuelles material presented the highest rate of increase of R_t , and the best values after 1 year (0,19 MPa, for 1% of lime).

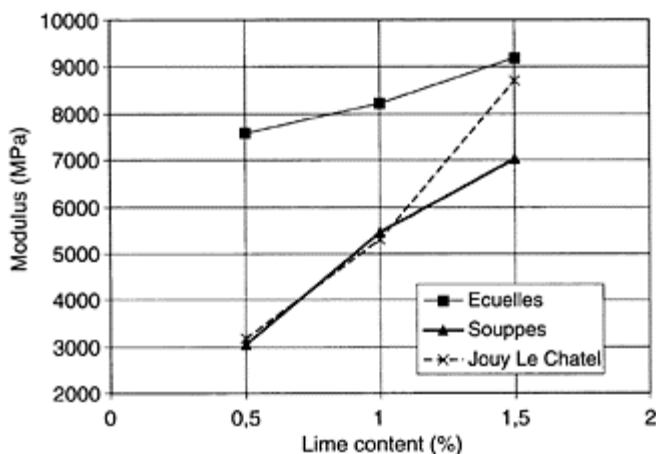


Figure 7. Influence of lime content on moduli measured after 360 days.

Figure 7 shows the influence of the lime content on the modulus values measured after 360 days. For the experimental range of lime contents (0.5% to 1.5%), the increase in modulus with the percentage of lime is important and quite linear. For the Souppes and Jouy-le-Châtel materials, between 0.5% lime and 1.5% lime, the modulus more than doubled. For the Ecuelles material, the increase was less marked (20%), although the absolute values of modulus were always higher than for the two other materials.

6 CONCLUSION

The objective of this study was to evaluate the effect of lime treatment on the mechanical properties of soft limestone granular materials from the region of Seine et Marne, southeast of Paris. Three materials, with medium to poor aggregate resistance (Los Angeles values ranging from 28 to 48) were evaluated both in the laboratory, and in situ, on an experimental pavement.

The laboratory tests showed that lime treatment improves significantly the mechanical properties of the limestone materials, due to carbonate cementation. After one year, the granular materials treated with 1% of lime all presented elastic moduli exceeding 5000 MPa, and significant tensile strengths, ranging from 0.13 to 0.19 MPa. The best properties were obtained on the softest material (Ecuelles). The tests also indicated that the modulus and tensile strength increase almost linearly with the quantity of lime, for the three percentages tested (0.5, 1 and 1.5%). It seems, therefore, possible to optimise the lime content, depending on the nature of the material and the required mechanical properties.

Aggregate resistance tests performed on the limestone aggregates (treated with 1% of lime) indicated a very significant reduction of the Micro Deval value (from 71.5 to 51.5) and of the Los Angeles value (from 48 to 41) for the softest limestone (Ecuelles). For the

other aggregates, the effect was less significant, probably because of their better initial (untreated) characteristics.

The experimental sections confirmed the good mechanical properties of the treated limestones. Globally, in terms of pavement performance, the survey of the experimental sections performed in 2001, after 9 years of traffic, indicated no significant distress (rut depths generally lower than 5 mm and only very limited longitudinal cracking); however, no clear difference in performance could be observed between the treated and untreated sections, due to the low volume of the traffic. The FWD and deflection measurements, however, indicated a substantial increase in stiffness of the treated sections, with values of elastic modulus of the base course attaining about 5500 MPa on the softest limestone (Ecuelles), and 3000 MPa on the two other materials, after two years. Compared with the laboratory tests, the in situ values are about 40% lower. This difference is probably due, in part, to the lower density obtained in situ (97% of optimum on average compared with 100% of optimum for the laboratory specimens), and to more severe environmental conditions in the pavement.

The study confirms that as for chalks, lime treatment is an efficient technique for improving the properties of soft limestone granular materials, at a relatively low cost (about 1 € per ton for a treatment with 1% of lime). However, additional work will be needed to define suitable rules for taking into account this type of “lightly treated” material in pavement design. In particular, their resistance to fatigue needs to be studied, to define an appropriate fatigue design criterion. The standard fatigue criterion used in the French design method for materials treated with hydraulic binders does not seem to be applicable, because their tensile strength is well below the minimum required for hydraulically bound materials.

REFERENCES

- Bhuiyan, J.U., Little, D.N., Graves, R.E. (1995). Evaluation of calcareous base course materials stabilized with low percentage of lime in South Texas, *74th TRB Meeting*, Washington.
- Chloup-Bondant, M., Evrard, O. Connaissance et propriétés physico-chimiques des couples matériaux-liants. *Materloc Calcaires Study Report*, Université de Nancy I.
- Chloup-Bondant, M., Evrard, O. Propriétés chimiques des matériaux. *Materloc Calcaires Study Report* Université de Nancy I:.
- del Piero, B. (2003). Ligne grande vitesse Est—Lot 23B tronçon C. *Travaux* n°798.
- Graves, R.E., Eades, J.L., Smith, L.L. Ca(OH)_2 treatment of crushed limestone base course materials for determination of self-cementation potential. *Transportation Research Record* 1250.
- Laboratoire Regional des Ponts et Chaussées de Lille (1991–1992). Etude des effets de l’apport de la chaux sur le comportement des craies humides en terrassements. *Special Lhoist Study Report*.
- Paquet, J. (1993). Mécanique et microstructure de la craie. Influence du traitement à la chaux. *Special Lhoist Study Report*, University of Lille.
- SETRA/LCPC, (1994). Conception et dimensionnement des structures de chaussées. *Guide Technique SETRA LCPC*, Paris.
- SETRA/LCPC, (2000). Traitement des sols à la chaux et/ou aux liants hydrauliques. Application à la réalisation des remblais et des couches de forme. *Guide Technique SETRA LCPC*, Paris.
- Terrassements & Carrières (2002). Stabilisation des sols : le traitement des craies humides à la chaux vive. *Terrassements & Carrières* n° 45.

Rehabilitation of Unbound Pavements using foamed bitumen stabilisation

J.D.Jones & J.M.Ramanujam

Queensland Department of Main Roads, Brisbane, Australia

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ABSTRACT: Most flexible pavements are expected to require some rehabilitation after about 20 years of trafficking. After that time they are typically rutted, contain excessive numbers of small repairs and are rough to drive on. They could also be a hazard to road users if the condition is really poor. According to the condition and cause of distress a number of rehabilitation alternatives are possible with many variations within each typically chosen on a whole of life, minimum net present cost basis. Pavement recycling or insitu stabilisation is a particularly attractive option. This paper provides a historical perspective of the performance of stabilised granular pavements in Queensland and discusses the details of design, construction and performance of foamed bitumen stabilisation technique to improve the performance of unbound granular pavement materials.

1 INTRODUCTION

The Department of Main Roads (Main Roads) is the steward of 34,000 km of Queensland's state controlled road network, which is 20 per cent of the state's total road network, carries 80 per cent of traffic and represents the state's largest single physical asset with a replacement value of approximately A\$30 billion.

Current expenditure allows for the construction of approximately 40 km of new pavement, 440 km of widening and road realignment and 330 km of existing pavement rehabilitation each year (QDMR, 2003). Experience in Queensland has shown that most flexible pavements are expected to require some form of rehabilitation after about 20 years of trafficking.

In recent years, pavement recycling or insitu stabilisation has become an extremely attractive alternative to traditional overlays for rehabilitation of existing pavements. The usefulness of this technique has expanded with the introduction of alternative additives and proprietary products.

Of particular interest is foamed bitumen stabilisation, which has been used in a range of situations from local roads right through to highways forming part of the national

highway network. This paper details the design, construction and performance of foamed bitumen stabilisation to improve the performance of unbound granular pavement materials.

2 PAVEMENT IN QUEENSLAND

Except for high traffic urban arterials connecting the major population centres in South East Queensland, the majority of pavements in Queensland consist of a spray sealed unbound granular construction. When such pavements are constructed from relatively low quality granular pavement materials and subjected to high axle loads, such as the case in Queensland, it is common for these pavements to require rehabilitation after about 20 years of trafficking, which mimics the design life cycle expected for these pavements. Queensland has a number of factors that influence the pavement design and rehabilitation options for existing pavement assets. These factors include the presence of expansive soils over approximately one third of the land area of Queensland, sources of high quality crushed rock are scarce and the allowance of vehicles with relatively high axle loads to use the state-controlled road network.

3 PAVEMENT REHABILITATION OPTIONS

3.1 *General*

For most pavements rehabilitation projects, where the pavement is significantly out of shape and showing signs of structural weakness, typically a number of options are considered for unbound granular pavements. These options would involve strengthening the pavement structure through stiffening the existing pavement (insitu stabilisation) or increasing the total depth of the pavement (overlay). Of the options available, only insitu stabilisation will be discussed further.

3.2 *Insitu stabilisation*

Insitu stabilisation is a process whereby the load-bearing capacity and/or stability of the existing material are improved to enhance the performance of the pavement and increase its useful life. Stabilisation comes in many forms but usually involves the inclusion of an additive to the existing pavement.

For rehabilitation of existing pavements the addition of a cementitious additive has typically been the preferred stabilisation method used by Main Roads. Its popularity reached its peak during the 1980's when it was used on a regular basis during the construction of many arterial roads in South East Queensland. Although these cement stabilised pavements were generally the lowest cost pavement option at the time of construction, their maintenance requirements were often significantly more than the full depth asphalt pavement alternative. Over recent years these pavements began to reach the end of their useful lives, several poor performance characteristics became apparent. The primary issues with these pavements were:

- The development of undesirable transverse shrinkage cracking at regular 3–6 m spacings.
- The development of undesirable, difficult to treat fatigue cracks towards the end of the pavement's design life.

The sensitivity of the layer thickness to the performance of the pavement. Inadequate pavement thickness, often only 10–20 mm less than that actually required, could result in rapid failure of the pavement.

4 FOAMED BITUMEN STABILISATION

4.1 General description

Foamed bitumen is a mixture of air, water and bitumen. Injecting a small quantity of cold water into hot bitumen produces an instantaneous expansion of the bitumen up to 15 times its original volume forming a foam. When the bitumen is in this foamed state, it is ideal for mixing with fine materials. The foam collapses very quickly and therefore rapid mixing is required to adequately disperse the bitumen throughout the material. During the mixing process, the foamed bitumen coats predominantly the finer particles, thus forming a mortar, which effectively binds the mixture together.

4.2 Bitumen foaming characterisation

The characteristics of the bitumen foam is characterised by two distinct properties; the expansion ratio and half-life. The expansion ratio is the ratio of the maximum volume of foamed bitumen to the volume of the unfoamed bitumen. The half-life is the time taken for the maximum foamed volume of the bitumen to settle to half this volume. The foaming characteristics of the bitumen are influenced by a number of properties including the bitumen class, the bitumen temperature, the foaming water content and the inclusion of a bitumen foaming agent to enhance the bitumen's foaming characteristics.

An expansion ratio of at least 10 times with a half-life of 30 seconds is desirable for adequate bitumen dispersion. Class 170 (equivalent to 80–100 penetration grade) bitumen is typically used for foamed bitumen stabilisation in Australia.

4.3 Hydrated lime as a secondary additive

Since the first foamed bitumen stabilisation project undertaken by Main Roads in 1997, hydrated lime (or quicklime) has been used as a secondary stabilising agent in the stabilisation process. It has the benefit of:

- stiffening the bitumen binder;
- reducing the potential for stripping to occur;
- assisting with the dispersion of foamed bitumen throughout the material;
- increasing the initial stiffness and early rut resistance of the stabilised material; and
- reducing the moisture sensitivity of the stabilised material.

4.4 Construction

4.4.1 Process

For insitu foamed bitumen stabilisation, the following construction process is typically used:

- Pulverise the existing pavement material using a reclaimer/stabilising machine.
- Identify any unsuitable material within the milled material and replace with fresh material.
- Spread hydrated lime and mix into the stabilised material. Additional water should be added to the material at this time for the purposes of binder dispersion and compaction.
- Inject and mix bitumen into the pavement material. This must be completed within one hour of the lime being incorporated into pavement material to avoid coagulation and agglomeration of the plastic fine particles before bitumen stabilization has been completed.
- Compact and trim the stabilised pavement material using standard construction practices.

4.4.2 Equipment

The Wirtgen 2500 or Wirtgen 2500 k series reclaimer/stabiliser machines have been used successfully for foamed bitumen stabilisation in Queensland. The Wirtgen 2500 k stabilising machines have the advantage of being able to inject hydrated lime directly into the mixing chamber. This system significantly reduces the creation of dust through separate spreading and mixing operations of hydrated lime. Therefore stabilisation using foamed bitumen can now be completed safely in relatively busy urban environments as it virtually eliminates safety concerns associated with the creation of airborne dust.

4.5 Application

4.5.1 Appropriate uses

Foamed bitumen stabilisation is generally suitable as a rehabilitation treatment for well graded granular pavement materials. Various researchers including Akeroyd et al (1988), Muthen (1999) and Maccarrone et al (1993) have investigated the suitability of materials for stabilisation with foamed bitumen. The grading suitability charts discussed by these authors all appear quite valid as a preliminary guide. Based on Main Roads' experience with mix design testing, foamed bitumen stabilisation is usually suitable for materials that meet the grading and plasticity limits indicated in Table 1.



Figure 1. Foamed bitumen stabilisation using a Wirtgen 2500 reclaimer/stabiliser.

Table 1. Suitability limits for foamed bitumen stabilisation (modified from Asphalt academy, 2002).

Sieve size (mm)	Percentage passing by mass (%)	
	Minimum	Maximum
26.5	80	100
2.36	25	60
0.075	5	20
Plasticity index	Maximum 12	

Based on Main Roads experience foamed bitumen stabilisation is a cost effective treatment in the following situations:

- Where there is insufficient suitable pavement material for cement stabilisation without the inclusion of additional pavement material. The greater fatigue resistance of foamed bitumen as compared to cement means that the stabilisation thickness can be reduced.
- Where the existing pavement material is of sufficient quality to meet the suitability limits indicated in Table 1. For very fines materials, the bitumen is unable to fully disperse throughout the pavement material.

- Where rapid strength gain is critical to allow early trafficking shortly after stabilisation has been completed. Foamed bitumen appears more tolerant of early trafficking than cement or lime/flyash modification.
- Where a stabilised pavement is the most cost effective option and it is highly undesirable for regular crack sealing treatments to be applied over the life of the pavement.

4.5.2 Inappropriate uses

Foamed bitumen stabilisation is generally not suitable where:

- Highly plastic fines are present in the parent pavement material.
- The grading of the parent pavement material and mix design test results indicate the treatment is unsuitable (as defined in Tables 1–2).

Table 2. Laboratory test requirements to determine suitability.

Design traffic (ESAL)	Minimum initial modulus (MPa)	Minimum cured modulus (MPa)	Minimum soaked modulus (MPa)	Minimum retained modulus ratio (%)
$<1 \times 10^6$	500	2500	1500	40
$1 \times 10^6 - 1 \times 10^7$	700	3000	1800	45
$>1 \times 10^7$	700	4000	2000	50

- Where the support directly below the stabilised layer is relatively low ($< \text{CBR5}$ as measured using a dynamic cone penetrometer).
- Bitumen can not be economically transported to the project site.

4.6 Design

4.6.1 Material design procedures

The strength (or more specifically the resilient response) of foamed bitumen stabilised material is determined using one of the following tests:

- Indirect tensile strength (ITS) test
- Indirect tensile resilient modulus (MATTA) test—Australian Standards AS2891.13.1.

In Australia, the MATTA test is the conventional method of testing foamed bitumen stabilised materials. During the early 1990's Emoleum (Australia) Ltd developed a test procedure for testing materials stabilised with foamed bitumen which is discussed in detail by Maccarrone et al (1994). Queensland Department of Main Roads has since adopted this basic test methodology, however the test procedure has since been simplified and modified to suit Queensland conditions. The Main Roads' test procedure involves mixing the material with bitumen and hydrated lime at a water content of 70% OMC (of

the unstabilised material) using standard compaction. The treated material is then compacted using the Marshall procedure. The specimens are compacted into 150 mm diameter $\times$ 65 mm high Marshall samples.

Samples are extracted from the moulds and cured for three hours at 25°C prior to initial modulus testing. This test is intended to give an indication of the early rut susceptibility of the material immediately after compaction. The applicability of the initial modulus requirement needs to be considered with respect to the immediate traffic loading anticipated during the 24 hours immediately after stabilisation. The specimens are then cured at 40°C for 3 days prior to retesting at 25°C. The specimens are tested in a dry state and then soaked in water for 10 minutes under a vacuum of 95 kPa less than atmospheric air pressure. The soaked modulus gives an indication of the material's susceptibility to weakening due to water penetration if the pavement becomes inundated. Guideline limits for minimum acceptable test results are provided in Table 2.

Testing for foamed bitumen stabilised material requires specialist test equipment. This equipment includes the laboratory bitumen foaming apparatus (WLB10) and MATTA test machine shown in Figure 2.

4.6.2 Pavement design

Over the last twenty years pavement design in Australia has moved away from empirical design methods towards mechanistic design procedures. These procedures are based on elastic theory of the pavement's response to wheel loads and the results of reported field and laboratory investigations in which material properties and behaviour have been characterised. The failure mechanisms assumed include:

- fatigue of bitumen-bound and cemented layers due to repetition of horizontal tensile strains at the bottom of such layers; and

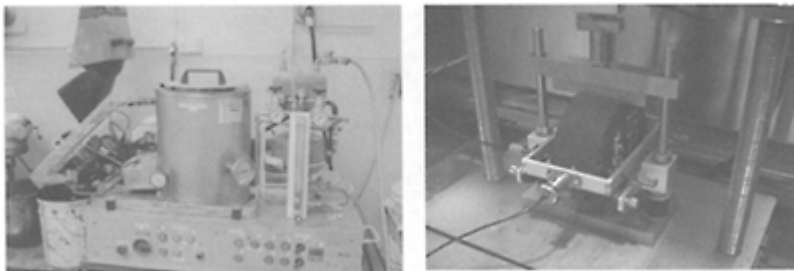


Figure 2. Wirtgen WLB10 laboratory bitumen foaming apparatus and indirect tensile resilient modulus (MATTA) test machine.

- permanent deformation of the subgrade due to repetition of vertical compressive strains induced in the subgrade.

Although field data in Queensland is scant at this stage, there is sufficient data to indicate the primary distress mechanism of foamed bitumen stabilised pavements is fatigue failure of the stabilised layer. Therefore an appropriate design criterion that reflects the fatigue performance of foamed bitumen stabilised pavements requires development. Currently the provisional methodology adopted by Main Roads is based on the Shell asphalt fatigue criterion used for asphalt design, which is discussed in Angel (1988). The limited field data available suggests that this criterion provides a reasonable estimate of the fatigue performance of foamed bitumen stabilised material provided the shift factor to relate laboratory to field performance does not exceed 1.0, the assumed volumetric percentage of binder does not exceed 8% and the resilient modulus of the stabilised layer does not exceed 2,500 MPa. The design modulus is based on the soaked indirect tensile resilient modulus results at the nominated design binder content for the project. Therefore:

$$N = \frac{6918(1.08 + 0.856V_b)}{S_{mix}^{0.36} \times \mu\epsilon}$$

where: N=design repetition to failure; V_b =volumetric binder content (6–8%); S_{mix} =Stiffness of foamed bitumen mix ($\leq 2,500$ Mpa); and $\mu\epsilon$ =induced horizontal tensile strain at bottom of stabilised layer.

A snapshot of the field performance of foamed bitumen stabilised pavements observed in Queensland is outlined in Section 4.7.

4.7 Observed performance

4.7.1 General

Since 1997, more than 30 km of pavement throughout the state of Queensland has been stabilised using foamed bitumen. Regular monitoring of the condition of several of these pavements has been undertaken as a means of validating the pavement design method developed. The location of these sites is shown in Figure 3. Details of the traffic loading and treatment applied to each section are provided in Table 6.

Pavement sections being monitored include:

- 1.2 km section of the Cunningham Highway, Gladfield
- 1.6 km section of the Cunningham Highway, Inglewood
- 0.6 km section of Rainbow Beach Road, Rainbow Beach
- 17 km section of the Cunningham Highway, Allora.

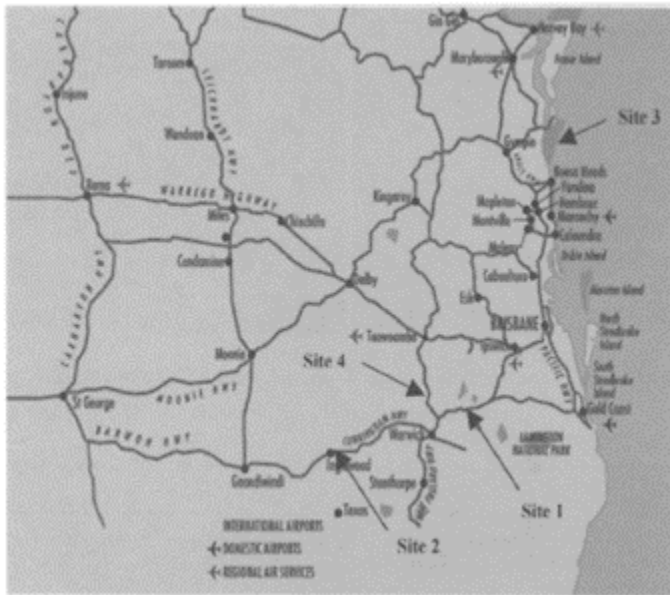


Figure 3. Location of foamed bitumen stabilisation sites.

4.7.2 Site 1: Gladfield—Cunningham Highway

The first foamed bitumen stabilisation trial was undertaken in May 1997 on the Cunningham Highway between Brisbane and Warwick. This section of pavement was overlaid in 2000 with 160 mm of unbound granular pavement material as part of a overarching rehabilitation strategy for the Cunningham Highway between Cunningham's Gap and Freestone Creek. Shortly before the granular overlay was constructed, approximately 10% of the pavement was exhibiting signs of minor fatigue distress as reported by Kendall et al (2001). These fatigued areas correlated relatively consistently with pavement failures that were stabilised with cement during previous maintenance treatments.

4.7.3 Site 2: Inglewood—Cunningham Highway

As part of the maintenance strategy for the Cunningham Highway, a 1.6 km section of unbound granular pavement near Inglewood, which has historically performed poorly, was stabilised with foamed bitumen as a repair treatment for this under-performing pavement section. This underperformance was primarily related to poor drainage conditions and irrigation methods used on adjacent farmland. Pavement stabilisation was undertaken in June 1998 and performed satisfactorily for approximately 3 years before deterioration of the pavement began to occur in isolated areas. An investigation to determine the failure mechanism was subsequently undertaken in November 2001. This

investigation revealed that inadequate subgrade support was the primary contributor to the observed distress. Fatigue cracking was not observed in situations where reasonable subgrade support was present. This highlights the influence subgrade support plays in performance of stabilised pavements. The distress in the stabilised layer is illustrated in Figure 4. The results of indirect tensile resilient modulus testing of six cores extracted from the pavement are provided in Table 3.

The reasons for relatively poor performance of this project include:

- wet and inadequate subgrade conditions;
- possibly a lack of compaction at the lower part of the stabilised layer;
- poor drainage conditions as described earlier; and
- the stabilisation thickness being inadequate to cope with the traffic loading applied.

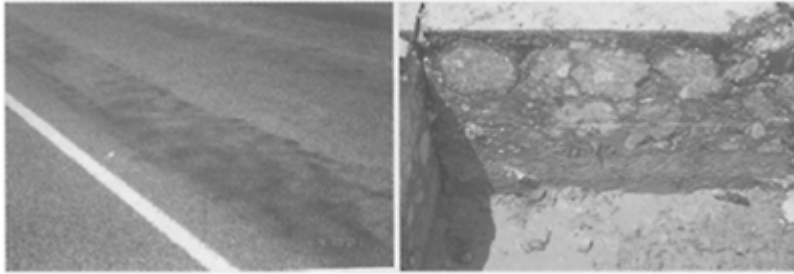


Figure 4. Fatigue cracking of the Inglewood foamed bitumen stabilised pavement, November 2001.

Table 3. Average indirect testing resilient modulus for cores extracted from Inglewood foamed bitumen stabilised pavement, November 2001.

	Dry modulus (MPa)	Soaked modulus (MPa)	Retained modulus ratio
Top	4,871	2,241	0.46
Middle	2,530	423	0.17
Bottom	2,329	321	0.14

Table 4. Average indirect tensile resilient modulus of extracted cores from Rainbow Beach Road.

	Dry modulus (MPa)		
	Top	Middle	Bottom
3% Bitumen & 1.75% Quicklime	4,400	3,761	3,336
4% Bitumen & 1.75% Quicklime	5,850	5,114	3,448

5% Bitumen & 1.75% Quicklime	5,612	4,855	1,246

4.7.4 Site 3: Rainbow Beach Road, Rainbow Beach

A trial aimed at comparing the performance foamed bitumen and bitumen emulsion stabilisation was completed in June 1998 on Rainbow Beach Road. The trial indicated that the early performance of the foamed bitumen sections were significantly better than the bitumen emulsion sections. However, the longer term performance of both sections has been similar. Indirect tensile resilient modulus test results of cores extracted from the three foamed bitumen sections approximately six months after construction are provided in Table 4.

4.7.5 Site 4: Allora—New England Highway

During April—May 1999 a 17 km section of the New England Highway was stabilised with foamed bitumen. This remains the largest foamed bitumen stabilisation project carried out in Queensland so far. Due to the length of the project, the condition of the pavement has been monitored on a regular basis. The monitoring has included:

- Visual assessment
- Deflection testing
- Coring of the pavement.



Figure 5. Condition of the Rainbow Beach in March 2003.



Figure 6 isolated fatigue cracking in foamed bitumen stabilised pavement, Allora.

Table 5. Average indirect testing resilient modulus for cores extracted from New England Highway foamed bitumen stabilised pavement.

	Dry modulus (MPa)	Soaked modulus (MPa)	Retained modulus ratio
Top	7,708	6,263	0.81
Middle	6,628	6,570	0.99
Bottom	3,759	2,764	0.74

After five years of trafficking, the pavement appears to be performing to a satisfactory standard with only isolated load associated failures covering less than 1% of the project length. An example of these isolated fatigue cracks is shown in Figure 6.

Four cores were also extracted from this pavement approximately one year after construction. The average indirect tensile resilient modulus results obtained from these cores are provided in Table 5. These results confirm the increase in pavement stiffness resulting from foamed bitumen stabilisation.

4.8 Correlation between pavement design and field performance

The development of an appropriate design procedure for foamed bitumen stabilised pavements is dependent on the gathering of substantial accurate field performance data. Although only limited data is currently available from Queensland projects, an analysis of the field performance in combination with the original mix designs and pavement investigations was undertaken to determine the appropriateness of the proposed design model. The results of this analysis were provided in Table 6. Although the performance of these pavements varies significantly, it can be seen that the model proposed provides a reasonable (but conservative) estimate of the expected life.

5 SUMMARY

The Queensland Department of Main Roads has used foamed bitumen stabilisation successfully as a rehabilitation technique for existing unbound granular pavements. Deflection testing and testing of extracted cores from these pavements indicates that significant improvement to the stiffness and load bearing capacity of the pavement can be achieved using the insitu foamed bitumen stabilisation technique. Observed performance of several projects has indicated that pavement stabilisation using foamed bitumen has superior fatigue resistance than material stabilised with cement. However, as

Table 6. Correlation between design and field performance.

	Freestone	Inglewood	Rainbow Beach	Allora
Date stabilised	May 1997	June 1998	June 1998	March—May 1999
Depth stabilised (mm)	250	200	200	250 OWP & 200 IWP
Additive content	4% bitumen & 2% cement	4% bitumen & 1.5% quicklime	3–5% bit & 1.75% quicklime	3.5% bitumen & 1.5% quicklime
Design modulus (MPa)	1,250	1,500	2,000	2,500 Mpa
Subgrade support (CBR)	7	5–20	10	4–10 (4 represents fatigue areas)
Expected traffic to failure (ESAL)	1×10^6	3×10^5 – 1.3×10^6	6×10^5	1×10^6 – 3×10^6
Actual traffic to failure (ESAL)*	2×10^6	8×10^5 (CBR5 only)	N/A	1×10^6 (CBR4 only)
Ratio actual life to expected life	2	2.7	N/A	1

* Failure means appearance of fatigue cracking in some sections but not failure of the entire job.

with all stabilisation treatments, determination of the correct stabilisation thickness is critical in achieving acceptable performance from these pavements. A provisional pavement design system has been developed, based on limited performance data, to determine the appropriate stabilisation thickness required to achieve acceptable pavement performance in the longer term.

REFERENCES

- Akeroyd, F & Hicks, B. 1988. Foamed Bitumen Road Recycling. *Highways*, Volume 56 (Number 1933): 42–45.
- Asphalt Academy, 2002. *Interim Technical Guideline: The Design and Use of Foamed Bitumen Treated Materials*, Pretoria: Asphalt Academy.
- Muthen, M. 1999. Foamed Asphalt Mixes—Mix Design Procedure. Contract Report CR-98/077. Pretoria: CSIR Transportek
- Kendall, M. Evans, P. Baker, B. & Ramanujam, J. 2001. Foamed Bitumen Stabilisation—The Queensland Experience, *Proceeding of the 20th ARRB Conference*, Melbourne: ARRB.
- Maccarrone, S. Holleran, G. Leonard, D. & Hey, S. 1994. Pavement Recycling Using Foamed Bitumen. *Proceeding of the 17th ARRB Conference*, Surfers Paradise: ARRB.
- Angel, D. 1988. *Technical Basis for the Pavement Design Manual*, Pavements Branch Report No RP1265, Brisbane: Main Roads Department Queensland.
- QDMR. 2003. Roads Implementation Program—2003/04 to 2007/08, Queensland Department of Main Roads, Brisbane, 2003.

Strength and swelling properties of Oxford Clay stabilized with wastepaper sludge ash

J.M.Kinuthia, R.M.Nidzam, S.Wild & R.B.Robinson

University of Glamorgan, Pontypridd, United Kingdom

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ABSTRACT: The potential of Wastepaper Sludge Ash (WSA), an industrial by-product of the recycling of paper, as a soil stabilizer is reported. Lower Oxford Clay (LOC), a sulfate-bearing clay, was stabilized with quick-lime ((CaO) at typical dosages of 2, 4 and 6%) and with various blended stabilizers incorporating WSA (at 10, 15 and 25% stabilizer dosages). Compacted cylinder specimens were made with and without allowing for a 3-day mellowing period prior to compaction. They were then cured for up to 365 days and then tested for compressive strength development. Linear expansion measurements were also taken for a period of up to 100 days. The results indicate that the strength values of systems incorporating WSA are generally higher than those utilizing traditional quick-lime. For all the stabilizers in the current investigation, strength values of the unmellowed specimens are also higher than for the mellowed specimens at all curing periods.

1 INTRODUCTION

As the environmental crises deepen and valuable resources continue to be depleted, it is important that resources be utilized at rates approximately equal to the natural rate of regeneration. This will involve optimization of recycling and use of renewable resources, subject to advances in technology. In the paper industry, the environmental impact of paper manufacturing may be reduced by increasing the quantities of paper recycled (Frederick et al. 1996), and by utilizing ash from combusted wastepaper sludge.

The composition of wastepaper sludge is a function of the type, grade and quality of the recycled paper, and also of its thermal history. The sludge comprises approximately equal amounts of organic and inorganic components, the latter consisting principally of limestone and kaolin. Péra & Amrouz (1998) have shown that combusted wastepaper sludge used as a mineral admixture in high strength concrete is effective as a pozzolan. In the UK, one of the principal wastepaper recycling companies, Aylesford Newsprint Ltd. (ANL), combusts wastepaper sludge in a fluidized bed, resulting in ash that is currently dumped to landfill (~700 tonnes/week). Research work by Kinuthia et al. (2001) and by Bai et al. (2003) has established the principal crystalline components in ANL's WSA as

typically calcium oxide (about 5 wt.% of which is free quick-lime with traces of calcium hydroxide), together with non-hydraulic, latently hydraulic and hydraulic components (gehlenite, bredigite, α -C₂S, small amounts of anorthite, ~5 wt.% calcium carbonate, and quartz). The ash is highly alkaline (pH 11–12) probably as a result of the residual free CaO. The potential of the utilization of this ash in concrete has been investigated as reported by Kinuthia et al. (2001), Bai et al. (2003), and by Veerapan et al. (2003). The objective of the current investigation is to establish WSA's potential for soil-stabilized pavement material, with or without blending it with CaO, Portland Cement (PC), or with ground granulated blastfurnace slag (GGBS).

2 EXPERIMENTAL PROCEDURE

2.1 *Materials*

Lower Oxford Clay was the target soil for stabilization. It was supplied by Hanson Brick Ltd. from their brick works at Stewartby, Bedford. Mineralogical studies by Hanson Brick Ltd. have established the LOC to contain illite (23%), kaolinite (10%), chlorite (7%), calcite (10%), quartz (29%), gypsum (2%), pyrite (4%), feldspar (8%) and organics (7%). Pyrites and sulfates in clay soils form expansive minerals such as ettringite and thaumasite when such soils are stabilized using CaO, making the stabilized layer volumetrically unstable (Snedker 1990, 1996, Higgins et al. 1998, 2002, Wild et al. 1999). LOC is therefore an excellent challenge for investigative work on soil stabilization.

WSA was supplied by Aylesford Newsprint Ltd. in the form of a dry coarse to fine powder with a small percentage (less than 10%) of sand-sized particles. Quick-lime (CaO) was supplied by Buxton Lime Industries Ltd. in the form of a fine white powder of cement size fineness, while Portland Cement (PC) was supplied by Blue Circle Ltd. Ground granulated blastfurnace slag (GGBS) was supplied by Civil and Marine Slag Cement Ltd., Llanwern, Newport. The oxide composition of LOC and these stabilizers are given in Table 1.

WSA was the key stabilizer used, with and without blending with CaO, PC or GGBS. The control mixes were LOC stabilized with 2, 4 and 6% CaO. The dosages of WSA, and WSA blends were 10, 15 and 20%. These dosages had been established in a previous unpublished research study as the levels likely to achieve the minimum CBR value of 15% stipulated by Department for Transport (DfT) for a CaO-stabilized capping layer (Highway Agency (HA) 2000, MCHW 1). For the blended binders, two mix proportions were investigated (90:10 and 80:20 WSA: CaO, PC or GGBS).

2.2 *Specimen preparation*

It was necessary to establish a common dry density and moisture content for specimen preparation. Therefore, several BS (BS1377, 1990) Proctor compaction tests were conducted in order to establish mean values of the density and moisture content to be adopted for the preparation of test specimens. In all the stabilized systems, the maximum dry density (MDD) ranged from 1.20–1.36 Mg/m³ and a mean dry density value of 1.30 Mg/m³ was adopted. The optimum moisture content (OMC) range was wide, from 27 to

33%, and two mean OMC values of 28% and 31% were adopted. Using the mean dry density and the nearest mean moisture content value, dry material (blended LOC+ stabilizer) intended for mellowing was thoroughly mixed with the water required to achieve OMC and stored in polythene bags to mellow for 3 days at $20\pm1^\circ\text{C}$ and 100% relative humidity. During subsequent compaction, an extra amount of water (equivalent to 20% of OMC) was added so as compact the material wet of OMC as in common practice. For the unmellowed material, all the water required to achieve this condition was added during compaction, which was achieved using a cylindrical steel mould and a hydraulic jack. The specimens were then wrapped in several runs of cling film and several layers of polythene bags, before being placed on a perforated Perspex platform below which water was always maintained to ensure high relative humidity. This is referred to as moist

Table 1. Oxide composition of LOC, CaO, WSA, PC and GGBS.

	CaO	SiO ₂	Al ₂ O ₃	MgO	Fe ₂ O ₃	FeO	CaCO ₃	MnO	TiO ₂	K ₂ O	P ₂ O ₅	Na ₂ O
LOC ¹	6.15	46.73	18.51	1.13	6.21	0.80	—	0.07	1.13	4.06	0.17	0.52
Lime ²	95.9	0.9	0.15	0.46	0.07	—	2.2	—	—	—	—	—
WSA ³	37.0	34.0	18.39	5.04	1.77	—	—	—	—	—	—	—
PC	63.0	20.0	6.0	4.0	3.0	—	—	0.03– 1.11	—	—	—	—
GGBS ⁴	42.0	35.5	12.0	8.0	0.4	—	—	0.4	—	—	—	—

Note: Data supplied by ¹Hanson Brick Ltd. ²Buxton Lime Industries Ltd. UK; ³Southern Water Services Ltd. for Aylesford Newsprint Ltd.; and ⁴Civil and Marine Slag Cement Ltd. UK.

curing, and was carried out for 7, 28, 90, 180 and 365 days in a temperature controlled chamber at $20\pm1^\circ\text{C}$, before testing for Unconfined Compressive Strength (UCS). Three specimens were used for each curing period and for each mix composition, and the average strength value determined.

For specimens used to monitor linear expansion, approximately 10 mm of the bottom of the samples was exposed immediately after specimen fabrication, by cutting and removing the cling film. The specimens were placed on porous discs and then placed on a Perspex platform which was in turn placed in Perspex containers. The lids to the containers were fitted with dial gauges. A layer of water was always maintained below the Perspex platforms to provide a high humidity thus minimizing evaporation from the samples. After moist curing for 7 days, the samples were partially immersed in water to a depth of 10 mm by increasing the water level using a siphon. This ensured minimal disturbance of the specimens. The containers were kept in a temperature controlled chamber, maintaining temperatures at $20\pm1^\circ\text{C}$ and to $65\pm5\%$ relative humidity. Within the container the r.h. would be expected to be approaching 100%. Linear expansion during moist curing and subsequent soaking was monitored daily for about 100 days.

3 RESULTS

3.1 Unconfined Compressive Strength (UCS)

Figures 1(a) and (b) show the unconfined Compressive strength of the CaO- and WSA-stabilized LOC for both mellowed and unmellowed conditions. In both systems, the performance of the unmellowed stabilized material is superior to that mellowed for 3 days, not only at 7 days but also after prolonged moist curing. At the low CaO level of 2%, no significant improvement in the strength with increasing curing time was observed for either mellowed or unmellowed conditions. The unmellowed material shows marginally higher strength values, relative to the mellowed material, throughout the one year of moist curing. At higher CaO levels of 4% and 6%, there is some improvement in strength upon prolonged moist curing, for both mellowed and unmellowed systems. After moist curing beyond 180 days, the unmellowed CaO-stabilized material shows superior strength development relative to the mellowed material. At all WSA levels (10%, 15%, and 20%), there is no noticeable improvement in the strength of the mellowed material with increasing curing time. This pattern is identical to the CaO-stabilized LOC at 2% CaO.

For the unmellowed WSA-stabilized material, there is also significant increase in strength throughout the period of moist curing with WSA levels above 10% resulting in a rapid strength increase after 28 days. In both CaO-LOC and WSA-LOC systems, it is apparent that the mellowing stage has a profound and long-term impact on both increase and rate of increase in strength. It is also apparent that in the unmellowed condition, WSA performs better than CaO, at the dosage levels investigated.

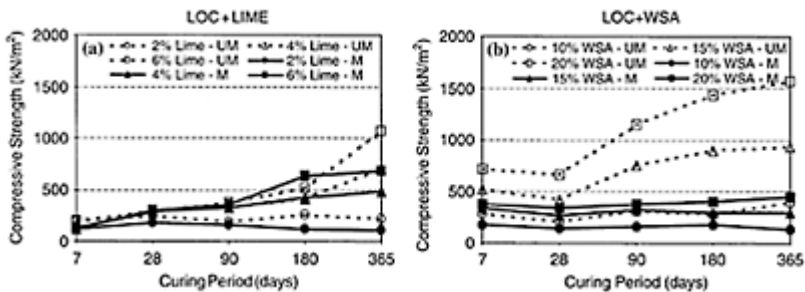


Figure 1. Compressive strength of LOC stabilized with (a) CaO, and (b) WSA, for mellowed and unmellowed conditions.

Figures 2(a) and (b) illustrate the strength development when LOC is stabilized with two WSA-CaO blends (90:10 and 80:20 WSA:CaO). As was the case for CaO-LOC and WSA-CaO-LOC systems, the unmellowed stabilized material performs better than the mellowed one. In contrast to the case when WSA is used on its own, as seen in Fig. 1 (b), by blending WSA with CaO, the mellowed material shows some slight strength increase

with curing time at all dosage levels of the blended stabilizer. Better strength improvement is achieved with the unmellowed material stabilized with the blend richer in WSA (90:10). Therefore, there is no apparent advantage of increasing the CaO content beyond that present in the 90:10 (WSA:CaO) ratio.

The effects of blending WSA with PC, rather than with CaO, are illustrated in Figures 3(a) and (b). The blending ratios were identical to those used in the WSA-CaO blends. The benefits to strength enhancement of blending WSA with PC are very similar to, but marginally greater than, those of blending WSA with CaO. There is however a noticeable enhancement of strength development of the mellowed material when the level of PC is increased in the WSA-PC blends from 90:10 to 80:20

Figures 4(a) and (b) illustrate the effects of blending WSA with GGBS, a readily available material in the UK as a by-product from steel manufacture. In a previous research study by the authors on the use of WSA-GGBS blends as binder in concrete (Kinuthia et al. 2001, Veerapan 2003), a 50:50 (WSA-GGBS) blend was observed to show optimal strength and durability performance. This is the basis of the 50:50 (WSA-GGBS) ratio in the current investigation. In order to improve on the economics of this blend for soil stabilization, a blend with a higher proportion of the cheaper WSA was also investigated (70:30 (WSA-GGBS)). Dosage levels adopted were similar to those of the other WSA blends using CaO or PC (i.e. 10, 15 and 20%).

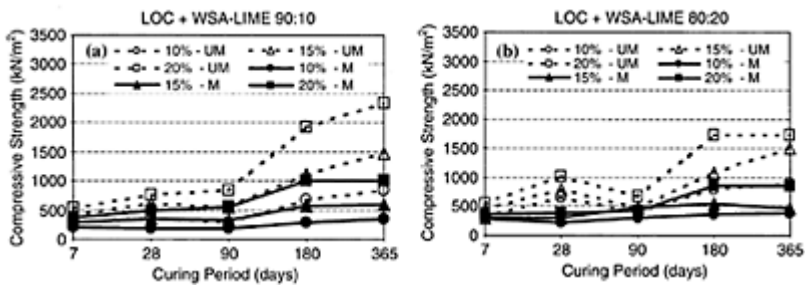


Figure 2. Compressive strength of LOC stabilized with two WSA-CaO blends (a) 90:10 WSA:CaO and (b) 80:20 WSA:CaO for mellowed and unmellowed conditions.

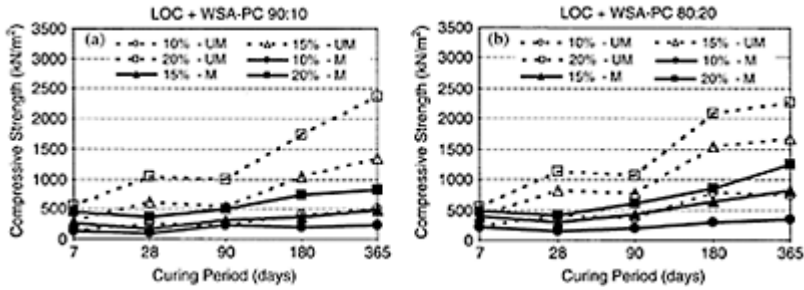


Figure 3. Compressive strength of LOC stabilized with two WSA-PC blends (a) 90:10 WSA:PC and (b) 80:20 WSA:PC for mellowed and unmellowed conditions.

As with the other stabilizers, WSA-GGBS blends also showed better strength development when the stabilized material was compacted without mellowing. The strength development was comparable with that achieved with WSA-CaO and WSA-PC blended stabilizers, with the 50:50 blends showing better performance in the case of the mellowed material, especially upon moist curing beyond 90 days (Fig 4(a)). Although the highest long-term strength in the entire LOC stabilization system investigated was observed on the unmellowed 50:50 WSA-GGBS system (2883 kN/m^2 , at 365 days), the performance of the 50:50 blend is still very close to that of the cheaper 70:30 blend.

3.2 Linear expansion

The linear expansion of all the stabilized LOC systems under investigation was monitored for a period of 100 days. Over this period all the systems either attained terminal linear expansion or continued to expand at a negligible rate of increase. Figure 5 shows a typical plot of linear expansion with increasing soaking time, for the most expansive—LOC-CaO—system. The Figure illustrates that the expansion reaches a stable level after about 40 days of soaking. For the lesser expansive systems, the stability was achieved much earlier and due to limitations in space on this paper, Table 2 shows only the terminal linear expansion after 100 days of soaking.

It is evident that at the stabilizer dosages investigated, LOC stabilized with WSA on its own or with WSA-blends recorded significantly lower expansion at 100 days, compared with the CaO-stabilized one, for both mellowed and unmellowed systems. With all the stabilizers, the mellowed

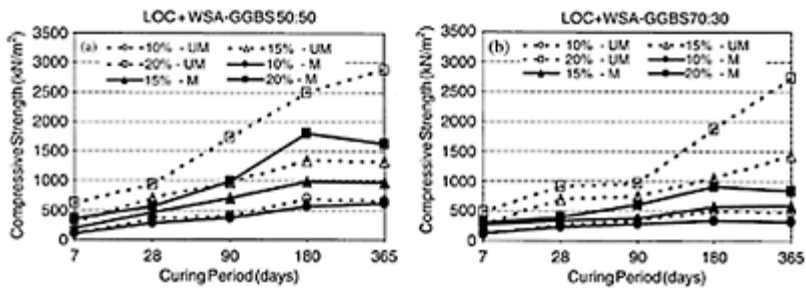


Figure 4. Compressive strength of LOC stabilized with two WSA-GGBS blends (a) 50:50 WSA:GGBS and (b) 70:30 WSA:GGBS for mellowed and unmellowed conditions.

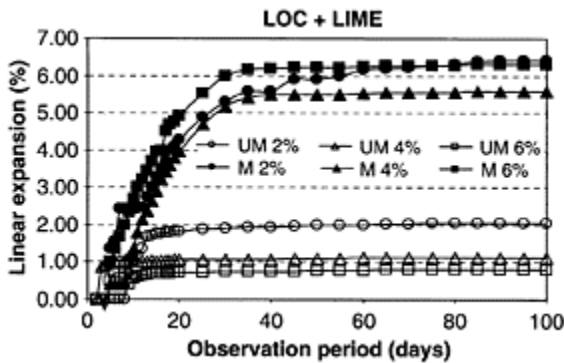


Figure 5. Linear expansion at 100 days of LOC stabilized with CaO and WSA (UM—Unmellowed; M—Mellowed for 3 days).

Table 2. Linear expansion at 100 days of LOC stabilized with two WSA-CaO blends, 90:10 and 80:20 (UM—Unmellowed; M—Mellowed for 3 days).

			WSA-CaO			WSA-PC				WSA-GGBS				
WSA			90:10		80:20	90:10		80:20		70:30		50:50		
M	UM	M	UM	M	UM	M	UM	M	UM	UM	M	UM	M	UM
10%	0.92	0.47	0.24	0.15	0.67	0.09	0.43	0.05	0.43	0.06	0.79	0.09	1.06	0.11

15%	0.62	0.81	0.32	0.28	0.32	0.14	0.18	0.21	0.45	0.05	1.46	0.09	1.51	0.07
20%	0.32	0.25	0.29	0.10	0.38	0.29	0.17	0.05	0.00	0.03	0.22	0.05	1.26	0.01

specimens generally expanded significantly more than the unmellowed ones. This contradicts previous observations by Thomas (2000), who observed that mellowing CaO-LOC mixtures reduced linear expansion. The current investigation appears to suggest that mellowing does not always result in reduction in expansion.

Table 2 shows that both WSA-GGBS blends showed remarkable volume stability in the unmellowed condition. This confirms previously observed GGBS-induced suppression of linear expansion in unmellowed CaO-stabilized soil system (Thomas 2000, Higgins et al. 1998, 2002, Wild et al. 1999). It also suggests that the amount of CaO consumed during mellowing (by GGBS and by cationic reactions) has a big impact on subsequent material improvement during curing. Considering that CaO at the dosage levels investigated also performed poorly on strength development compared to WSA/WSA-blends, it is further clear that there are technological, economic as well as environmental advantages with the systems containing WSA.

4 DISCUSSION

It is well-established that during the period of mellowing CaO is consumed in a soil modification process in which the soil properties are changed (Kinuthia et al. 1999). From the current work it is evident that the changes in soil properties also include long-term effects on the strength properties of the mellowed material. When the CaO level originally present in the stabilized target material is low, there is no significant pozzolanic activity after mellowing and hence no enhanced strength development upon prolonged curing time.

There are numerous mechanisms that would result in delayed strength enhancement that is witnessed in the current work. The availability and rate of consumption of CaO above the ICL value, could be controlled by other factors besides the mere presence of excess CaO, such that the conditions are not conducive for enhanced strength in the unmellowed system until after some period. If the unmellowed material has the capability to produce sulfate via oxidation, then the consumption of residual CaO will gradually increase resulting in enhanced strength development in a sulfate-induced ettringite formation and strength enhancement and/or expansion (Kinuthia & Wild 2001). However, strength development especially in the presence of sulfate is not simply related to CaO content. For the mellowed material, the oxidation of LOC takes place early during the mellowing period, when the material has an easy access to CaO (Thomas 2000). As more hydration products form, the porous nature of the mellowed material (as evidenced by its lower density relative to the unmellowed material (Thomas 2000)) becomes a disadvantage, due to the more porous structure. In contrast, the more compact system of the unmellowed system benefits from the increase in hydration products.

When LOC is stabilized with WSA, the lack of strength increase in the mellowed system even at high WSA dosage levels (20%) suggests that the amount of CaO initially available in the system is low. It also suggests that after the initial free CaO present in the

WSA is consumed, the WSA is incapable of further hydration during prolonged moist curing. In the unmellowed WSA-LOC system, the CaO initially present in the WSA is utilized in both modification and stabilization processes whereas in the mellowed system, the modification process takes place when the soil is not compacted. The system is therefore susceptible to carbonation of lime, although this was minimized by the sealing in a polythene bag. The result is that there is little or no more CaO to further the stabilization process upon compaction. The situation is exacerbated by the porous nature of the material as already mentioned. In the unmellowed system, the process of modification and stabilization takes place in a more compact material and without interference. This has beneficial consequences as evidenced by the strength results. Carbonation of CaO and stalled hydration of WSA due to reduction of pH all add to the complexity of the hydration systems investigated.

Blending WSA with CaO or PC results in good strength development with increasing curing time, and better performance in volume stability in both mellowed and unmellowed conditions. On the other hand, blending WSA with GGBS shows the best performance for the unmellowed system. Previous work on CaO-GGBS systems has established the consumption of CaO by GGBS during curing. It is also well-established that the hydration of slag is also enhanced by the presence of sulfate (Higgins et al. 1998, 2002, Kinuthia et al. 1999), and the volume stability of the unmellowed WSA-GGBS system is only comparable to that of the WSA-PC. The closely comparable performance of WSA blends suggests that a decision on the preferred material (CaO, PC or GGBS) for blending with WSA will also be determined by other considerations besides strength development, such as relative cost and availability of the materials, volume stability, durability and site or other considerations.

5 CONCLUSIONS

From the work carried out on strength and linear expansion properties of stabilized LOC, it may be concluded that it is possible to utilize WSA for the stabilization of a sulfate-bearing clay. In the current research, the strength and expansion properties of WSA-stabilized LOC were superior to those achieved using the traditional CaO-stabilization. Therefore there is potential for technological, economic as well as environmental advantages of utilizing WSA and similar waste in pavement construction.

By blending WSA with a controlled amount of small quantities of CaO, the performance of WSA is greatly enhanced, resulting in improvement of both strength and volume stability. Both PC and GGBS may also be used to blend WSA, leading to systems of comparable advantageous effects, particularly when the material is compacted without mellowing. However, the high dosages of GGBS required are likely to be uneconomical.

Mellowing was not beneficial in the systems investigated in the current study. Whether or not to mellow will depend primarily on stabilizer used, other variables including the period of mellowing, the target material and site condition, besides possibly other factors. It is possible that the disadvantages of mellowing, such as reduced long-term strength, may be mitigated by compensating effects such as those of improved volume stability. Thus, more research on a wider range of soils and conditions prior to

compaction is needed in order to fully establish the circumstances under which mellowing is likely to be beneficial.

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REFERENCES

- Bai, J., Chaipanich, A., Kinuthia, J.M., Lewis, M.H., O'Farrell, M., Sabir, B.B. & Wild, S. 2003. Compressive strength and hydration of wastepaper sludge ash (WSA)—ground granulated blastfurnace slag (WSA-GGBS) blended pastes, *Cement and Concrete Research*, 33, pp 1189–1202.
- British Standard (BS) 1377, 1990. *Methods of test for Soils for civil engineering purposes*.
- Frederick, W.J., Iisa, K., Lundy, J.R., O'Connor, W.K., Reis, K., Scott, A.T., Sinquefield, S.A., Sricharoenchaikul, V. & Van Nooren, C.A. 1996. Energy and materials recovery from recycled paper sludge, *Tappi Journal*, 79, No.6, pp 123–130.
- Higgins, D.D., Thomas, B. & Kinuthia, J. 2002. Pyrite oxidation, expansion of stabilized clay and the effect of ggbs, *Proc. 4th European Symposium, Bitmap4, on Performance of Bituminous and Hydraulic Materials in Pavements*, April 2002, 348 pp 11–12, Nottingham, UK.
- Higgins, D.D., Kinuthia, J.M. & Wild, S. 1998. Soil stabilization using CaO-activated GGBS, *Proc. 6th CANMET/ACI Int. Conf. on Fly ash, Silica Fume, Slag and Natural Pozzolans in Concrete*, May 31 st-June 5th, 1998, Bangkok, pp 1057–1074.
- Highway Agency (HA), 2000. *Design manual for roads and bridges (DMRB)* HMSO, Vol. 4—Geotechnics and drainage, Section 1—Earthworks, Part 6- HA74/2000-Design and construction of CaO stabilized capping.
- Kinuthia, J.M. & Wild, S. 2001. Effects of some metal sulfates on the strength and swelling properties of CaO-stabilized kaolinite, *International Journal of Pavement Engineering (IJPE)*. Vol. 2, No. 2, pp 103–120.
- Kinuthia, J.M., Wild, S. & Jones G.I. 1999. Effects of monovalent and divalent metal sulfates on consistency and compaction of CaO-stabilized kaolinite, *Applied Clay Science*, 14, pp. 27–45.
- Kinuthia, J.M., O'Farrell, M., Sabir, B.B. & Wild, S. 2001. A Preliminary Study of Cementitious Properties of Wastepaper Sludge Ash Ground Granulated Blast-Furnace Slag (WSA-GGBS) Blends, *Proc. International Symposium, Recovery and Recycling of Paper*, 19th March 2001, Dundee, pp 93–104.
- MCHW 1. *Manual of Contract Document for Highway Works* Volume 1—Specification for Highway Works.
- Péra, J. & Amrouz, A. 1998. Development of highly reactive metakaolin from paper sludge, *Advances in Cement Based Materials*, 7, pp 49–56.
- Snedker, E.A. & Temporal, J. 1990. M40 Motorway Banbury IV Contract- CaO Stabilization, *Highways and Transportation*, December 1990.

- Snedker, E.A. 1996. M40—*CaO Stabilization Experiences, CaO Stabilization*, Thomas Telford, London, pp 142–158.
- Thomas, B.I. 2000. *Stabilization of sulfide rich soil: Problems and solutions*, PhD Thesis, University of Glamorgan, Pontypridd, UK.
- Veerapan, G., Kinuthia, J.M., O'Farrell, M., Sabir, B.B. & Wild, S. 2003. Compressive strength of concrete block manufactured using wastepaper sludge ash, *International Symposium: Advances in Waste Management and Recycling, Symposium W2-Recycling and Reuse of Waste Materials*, 9–11th September 2003, Dundee, UK.
- Wild, S., Kinuthia, J.M., Jones, G.I. & Higgins, D.D. 1999. Suppression of swelling associated with ettringite formation in CaO-stabilized sulfate-bearing clay soils by partial substitution of CaO with ground granulated blastfurnace slag (GGBS), *Engineering Geology*, 51, pp 257–277.

Unsealed GeoCrete-road with high bearing capacity

C.van Gorp,

KOAC•NPC, Apeldoorn, The Netherlands

B.Kroesen

GeoCrete BV, Schiedam, The Netherlands

Pavements Unbound—Dawson (ed.)

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ABSTRACT: The monitoring and analysis of the structural characteristics, behaviour and performance of the stabilisation material GeoCrete® mixed with cement is presented. The mix was applied for upgrading the structural condition of an unpaved low-volume road in Laarbeek, The Netherlands. The objective of the research programme was to gather data that could be used as reference for future reconstruction projects of roads, industrial areas and even verges. Results of extensive laboratory testing and periodic Falling Weight Deflectometer surveys are presented. Special attention is given to documentation of the actual construction and weather conditions.

1 INTRODUCTION

This paper describes the research programme on the structural characteristics, behaviour and performance of the stabilisation material GeoCrete® mixed with cement as applied in a 700 m long section of the unpaved low-volume road Wilhelminaweg in Laarbeek, The Netherlands. The road is in use for residential traffic only. Traffic loading is low and consists of approximately 25 cars and three heavy goods vehicles (HGVs) per day. Enhancement of the bearing capacity was necessary because extra HGV traffic was expected due to construction works in the vicinity of the road. However, an absolute requisite was that the local rural character of the road should be preserved. The local soil was not removed for the upgrading of the road but stabilised in place. The nominal thickness of the stabilisation layer is 250 mm. It was expected that the pavement life would be less than 20 years.

The objective of the research programme was to gather data that could be used as reference for future reconstruction projects of roads, industrial areas and even verges. The programme mainly consists of laboratory experiments and field trials with coring and Falling Weight Deflectometer (FWD) testing. Laboratory experimentation was focussed on determination of the characteristics of the local soil, the composition of the premix to be applied, and the strength values actually accomplished in the various stages of

hardening. Additionally special attention was given to documentation of the actual construction activities and weather conditions. Detailed reporting of the project is given by Van Gorp et al. 2003.

2 PRODUCT DESCRIPTION

GeoCrete® is a whitish powder consisting of alkaline and alkaline earth elements or complex compounds. It promotes cement hydration processes and inhibits the action of fulvic acids and carbonic acids. The structural changes and the formation of minerals occurring during cement hydration greatly increase the compressive strength, the static and dynamic stiffness modulus, the tensile bending strength, and the frost resistance of the soil, and also stabilise humus-rich soils.

Apart from improving the above-mentioned parameters, GeoCrete® also promotes the immobilisation of pollutants, such as heavy metals and organic parameters, which get permanently embedded in the new crystal structures in the soil. Stabilised bases courses treated with cement only, usually combine high stiffness with a high risk on premature cracking. This undesirable combination was regarded as a major handicap for stabilising. One of the design objectives was that the pavement surface would remain unsealed and preferably uncracked for a long time. A mixture of 12% cement containing 1.5% GeoCrete® was applied because expectations were high that it would provide sufficient stiffness and integrity for the long term. Determination of the mixture depends on the properties of the local soil.

3 PREPARATION

Sieve analyses and other typical soil tests were used to characterise the local soil of the Wilhelminaweg. In summary, the soil can be labelled as slightly gravelly, slightly silty sand. The soil does not comply with the requirements set in The Netherlands to materials to be used as capping layer in a pavement structure (RAW Conditions 2000). The proportion of fines are slightly too high. The content of organic matter does not exceed the level of 1.5%. The fulvic acids in the soil appear to be of a nasty type capable of destroying the hardening process of the stabilisation. Fortunately, the content of fulvic acids was rather low. Maximum Proctor density was investigated to be equal to 1980 kg/m³ at a moisture content of 14% (Korotowski 2003).

Suitability testing in the laboratory showed that at best a premix of 12% cement with 1.5% GeoCrete® could be applied to the soil under investigation. Minimum compressive strength values of 2 MPa after 28 days and 3 MPa after 90 days were considered needed to meet the design life requirements according to Keppens (2002).

4 STABILISATION

4.1 Construction

Stabilisation was conducted in a period of two days on 27 and 28 February 2003. The month prior to the construction days was dry, sunny and cold with various weeks with frost at night. Temperature dropped below -5°C two days before commencement of the work. Therefore, the stabilised soil was very cold and inevitably this must have had its effect on the hardening process in the weeks following construction. These weeks were extremely sunny and dry with very mild weather.

All obstacles and rubble were removed from the road after which a grader was used to obtain the desired profile and cross-fall. The premix was applied in two batches. After each batch two milling passes were used to mix the premix as good as possible with the local soil. Extra water was added after each mill sequence. The quantity of water spread was needed to increase the moisture content to the desired optimum level. Unfortunately water was not spread uniformly over the width of the road leading to the risk of variation of quality of the structural behaviour and performance of the stabilisation layer over the width of the road. In total four milling passes were applied with two sequences of adding extra water. Milling started immediately after spreading of the premix. Depth of milling was set to roughly 25 cm. Analyses showed later that deeper milling should have been better. Width of the milling equipment was 2.20 m whereas road width was 4 m. Compelled by necessity, material around the centre line of the road was mixed more frequent than material closer to the shoulders.

The road section was compacted in two passes by a vibrating roller. A small grader was used to correct the profile trying to obtain a cross-fall of 1%. A last round of compaction was applied either by the same vibrating roller with vibration option switched off or by a rubber-tyred roller to remove all remaining irregularities. At the first day of compaction patches of stabilisation material got stuck to the rollers leading to unwanted irregularities in the pavement surface. These patches were pressed onto the pavement surface but formed no good bond with the underlying material anymore. Use of an extra seal coat of small sized granular aggregate prior to the last round of compaction, prevented that stabilisation material could stuck to the roller. This measure led to a better and more uniform compaction process. As a last action, extra water was spread over the surface in a number of passes to decrease the risk of quick evaporation and desiccation.

4.2 Laboratory tests

At first sight, the layer thickness of the large quantity of cores appeared to vary between 185 mm and 300 mm. Better inspection of the core holes revealed that a part of the dispersion could be explained from the fact that some cores were broken during coring leaving some of the stabilisation material remaining in the hole. Analysis of construction data and analyses for FWD test results pointed at a more uniform distribution of layer thickness.

Compaction degrees as determined according to the Dutch specifications varied between 93% and 100%. At some locations the individual samples from the pavement complied with the requirements, but on average the degree of compaction was slightly below the requested value of 100%. Nevertheless, compressive strength of the samples was (sometimes ways) above the requested values of minimum strength. Table 1 presents the compressive strengths of specimens mixed and prepared in the laboratory.

The test results have been analysed with reference to the requirements set to soil cement simply because the stabilisation material contained cement and no other specification was available for this product in The Netherlands (RAW Conditions 2000). The mean compressive strength of specimens prepared in the laboratory was requested to be 5 MPa after 28 days. In reality a value of 9.1 MPa was achieved.

Table 2 presents the compressive strength values of cylinders cored 7, 28 and 90 days after construction. The section delineation originates from the FWD trial. Table 2 shows that the differences between compressive strengths of the upper and lower halves of the core are small. Spatial variability is much larger. The Sections 1 and 3 provide better strength values than the other sections.

Because of the dispersion in test results, extra cylinders were cored after 7 days since construction to determine the compressive strength after 28 days once more. The test results are presented in Table 3. The “new” cores are not much better than the “old” cores from the Sections 1 and 4.

The compressive strength of cored specimens should be equal to or greater than 1.5 MPa according to the Dutch RAW standard conditions 2000. A quick perusal of the tables 2 and 3 exhibits that most cores comply easily with the requirements. Only in the case of the 7 days values, half of the cores cannot meet the requested value.

The coefficient of variation of compressive strength is quite large and varies between 40% and 70%. A tighter band of variation would have been better. However, from experience many bound foundation layers appear to show a similar pattern of spatial variability in strength and stiffness values.

Table 1. Compressive strength of laboratory specimens.

Code cylinders	Compressive strength after 28 days (MPa)
1 (400 m)	13.5
2 (350 m)	11.2
3 (300 m)	10.0
4 (200 m)	2.4
5 (100 m)	8.3
Average	9.1

Table 2. Compressive strength of cored cylinders.

Compressive strength (MPa)				

Section	After 7 days		After 28 days		After 90 days	
	Code cylinder	Comp. strength	Code cylinder	Comp. strength	Code cylinder	Comp. strength
1	1a top	5.8	1b top	8.1	1c top	10.7
	1a bottom	6.5	1b bottom	6.9	1c bottom	9.3
2	2a top	0.6	2b top	4.9	2c top	5.4
	2a bottom	0.8	2b bottom	3.9	2c bottom	5.0
3	3a top	6.8	3b top	13.6	3c top	21.3
	3a bottom	7.2	3b bottom	8.0	3c bottom	10.6
4	4a top	–	4b top	4.7	4c top	–
	4a bottom	1.4	4b bottom	–	4c bottom	7.5
5	5a top	1.2	5b top	4.6	5c top	5.3
	5a bottom	1.0	5b bottom	3.1	5c bottom	3.4
Average upper half		3.6		7.2		10.7
Average lower half		3.4		5.5		7.2
Grand average		3.5		6.4		8.7

Table 3. Compressive strength of extra cores.

Section	Code cylinder	Total core thickness (mm)	Location	Compressive strength after 28 days (MPa)
1	11a top	250	623 m Left (between wheel paths)	5.8
	11a bottom			8.0
	11b top	210	668 m Left (between wheel paths)	1.8
	11b bottom			2.3
4	14a top	250	190 m Right (between wheel paths)	2.9
	14b bottom	280	235 m Left (between wheel paths)	2.7

5 FIELD TRIALS USING FWD TESTING

FWD tests were performed to assess the bearing capacity of the road and to capture data for evaluation of the development of the stiffness modulus of the stabilisation layer with

time. The tests were conducted 7, 28 and 90 days since construction. The first set of deflection data was used to delineate the road into five homogeneous sections. Layer thickness down to a depth of 1 m was determined via coring at a representative position in each section. Deflection data and layer thicknesses were used for the back-analysis of stiffness moduli of the constituent layers of the pavement structure. Two drop heights were used to generate loads of 25 kN and 50 kN to investigate any effect of stress dependency. The subgrade was divided into an upper sub-layer of 0.5 m and a lower infinite half space, since the surface modulus showed an increasing value with depth. By splitting the subgrade for analytical reasons, more accurate layer stiffness moduli will be calculated when using a linear elastic approach. Table 4 presents the results of the back-analysis. This table clearly demonstrates that the stiffness moduli of the stabilisation layer vary extensively. The stiffness moduli of the stabilisation course are also displayed in Figure 1. This figure and Table 4 also shows that the stiffness modulus of that layer increases with time. The at first poor Section 2 develops an acceptable stiffness after 28 days. The initial low stiffness value can be due to the low stiffness of the subgrade at that location causing loss of the compaction energy due to absence of proper structural support.

Table 4. Layer stiffness moduli at two levels of FWD load.

Section	Test day	Stiffness modulus (MPa)					
		Stabilisation		Subgrade(upper part)		Subgrade (lower part)	
		25 kN	50 kN	25 kN	50 kN	25 kN	50 kN
1	7	1,050	840	121	109	124	110
	28	1,600	1,500	111	103	117	111
	90	1,750	1,650	126	118	121	115
2	7	390	240	80	77	115	105
	28	2,950	2,200	65	70	137	118
	90	4,950	4,400	65	65	143	131
3	7	2,350	1,900	117	113	140	124
	28	5,450	5,100	86	85	147	137
	90	7,150	7,050	89	72	147	144
4	7	740	560	114	101	136	118
	28	5,500	4,900	103	92	147	137
	90	7,550	6,850	102	90	156	149
5	7	5,000	3,850	78	92	139	119

28	8,800	8,100	100	100	137	129
90	8,150	8,050	80	60	141	145

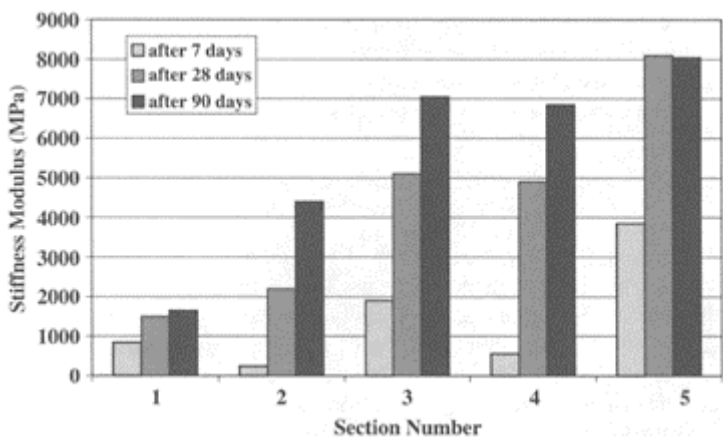


Figure 1. Stiffness modulus of stabilisation course per section over time.

The effect of stress dependency seems to be marginal. The stiffness modulus appears to decrease slightly with increasing loads imposed. This pattern is often found at pavement structures with traditional granular material in the base course. It is common practice in sandy regions that the upper part of the subgrade is less stiff than the lower part. Confinement is the principal driving factor behind this difference. The stiffness moduli backcalculated are normal values for the type of material in the subgrade.

No real design verification calculations were performed in the research project because of the simplicity of the design and traffic loading for comparing the bearing capacity of other pavement structures built up with traditional materials. Nevertheless in summary, may be concluded that the bearing capacity and layer stiffness moduli are pretty high for the type of road.

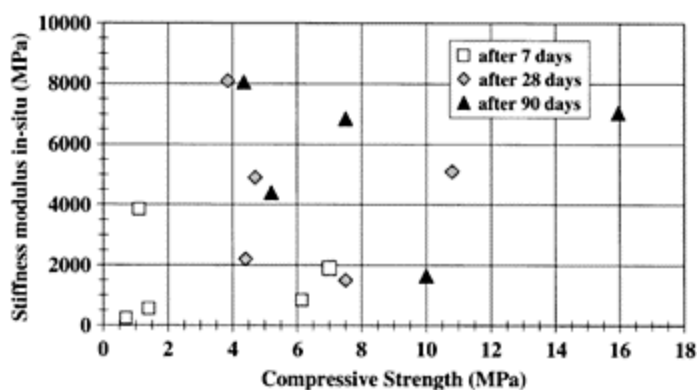


Figure 2. Relationship between compressive strength and stiffness modulus of the stabilisation course.

6 ANALYSIS

6.1 Relationship between stiffness modulus and compressive strength

Comparison of the stiffness moduli and compressive strengths (Fig. 2) of the stabilisation course shows that the Sections 1 and 3 exhibit similar behaviour. Both stiffness modulus and compressive strength of Section 2 are on the low side. Remarkably, Section 5 performs much better than the other sections although the compressive strength values are not that high. Incorrect layer thicknesses in the back-analysis may be the cause of this deviation of the pattern. A closer investigation into this issue did not corroborate use of incorrect values of thickness.

6.2 Relationship between test results and design parameters

6.2.1 Stiffness modulus

The stiffness modulus after 90 days varies for the five sections between 1,650 MPa and 8,050 MPa for the 50 kN drops. The backcalculated values are section means. For design purposes lower values are used that account for some degree of safety or reliability. Often a 85% reliability level is used assuming that 15% of the section will be of lesser quality. A design stiffness modulus of 3,000 MPa is recommended if a 85% reliability level is used. The analysis demonstrates that it will pay off to reduce the dispersion in strength and stiffness. If Section 1 is erased from the set of data, then the design stiffness modulus will jump to a 5,000 MPa level, a astonishing 2,000 MPa higher than found for the entire road.

6.2.2 Compressive strength and tensile bending strength

The tensile bending strength of the stabilisation course was set to 1 MPa in the design of the pavement structure. Cementitious materials usually allow determination of the compressive strength quite easily, but the relationship between this parameter and the tensile bending strength depends heavily on the type of material. Since the tensile bending strength is the principal design parameter, determination of the relationship between the tensile bending strength and the compressive strength is a prerequisite for verification of the design. In this particular case, only compressive strength testing was performed. Conversion equations may be used to convert one parameter into the other. The conversion equations developed by Packard and the VBC 1995 conversion equation were used. Packard's equation is developed for lean concrete with approximately 100 kg/m³ Portland cement.

$$\sigma_{bt} = 0.318 \cdot \sigma_d^{1.02} \quad (1)$$

Table 5. Prediction of tensile bending strength based on compressive strength.

Compressive strength (MPa)	Tensile bending strength (MPa)	
	Packard approach	VBC 1995 approach
1	1.7	0.1
3	1.8	0.4
5	1.9	0.7
7	2.0	1.0
9	2.1	1.3
11	2.3	1.6
13	2.4	1.9
15	2.5	2.2

where σ_{bt} =tensile bending strength (MPa); σ_d =compressive strength (MPa).

The VBC 1995 approach presents the following relationship between the design values for bending tensile strength and compressive strength.

$$\sigma_{bt} = \frac{(1.6 - h) \cdot (1.05 + 0.05 \cdot (\sigma_d + 8))}{m} \quad (2)$$

where h=thickness of stabilisation course (m); m=material factor.

The layer thickness varies around 0.25 m. A material factor of 1.2 should be used for the type of application. Table 5 displays which tensile bending strengths will be predicted on the basis of the compressive strength.

A tensile bending strength of 1 MPa was used in the initial design. Table 5 shows that the two predictive approaches result into quite different required values for the corresponding compressive strengths. The average strengths of 6.4 MPa and 8.7 MPa

were actually found after 28 days and 90 days (Table 2). Tensile bending strength values of 1 MPa to 2 MPa seem to correspond with this input. In conclusion can be stated that correct values were used in the design.

The high compressive strengths point at the risk of premature cracking in the stabilisation course. In a period of seven months after opening to traffic, hardly any distress could be observed at the pavement surface, although the surface was not covered by any seal coat or surface dressing. This underlines the capacity of the GeoCrete® application by combining high strength with low risk of cracking.

7 CONCLUSIONS AND RECOMMENDATIONS

In summary can be concluded that the application of 12% Portland cement containing 1.5% GeoCrete® in the stabilisation course of the unsealed pavement structure under investigation resulted into high values of bearing capacity, high stiffness moduli and high compressive strengths. The common pattern of premature cracking under these conditions could not be observed at the Wilhelminaweg. Hardly any growth in distress could be observed later. Consequently, the product is a good application to combine increase in bearing capacity with low risk of (premature) cracking. This combination allows construction of asphalt pavements that might be substantially thinner than the currently used 120 mm and 140 mm thick asphalt layers in structures with bound base courses. The thicknesses mentioned are only needed for limiting the propagation of cracking to the pavement surface. The propagation rate appears to be much smaller in the GeoCrete® case.

Another benefit of the product is that local soil does not have to be removed from the site but can be improved in place. The project showed that more than sufficient bearing capacity and quality was realised.

The local soil in the project under investigation contained a small but aggressive amount of fulvic acids. These acids deteriorate the hardening process of the stabilisation material. GeoCrete® has the ability to immobilise this detrimental effect. The absence of cracking and other signs of distress may be regarded as evidence for the well-performance of the product.

Careful and accurate realisation of a stabilisation project determines the success whether the potentially good properties of GeoCrete® will be fully deployed. Detailed protocols and construction procedures should be drafted to guarantee quality of future works.

Layer thickness of a stabilisation project in place course should be kept constant over the length of the road. Premix and water should be distributed and spread uniformly over the width of the pavement. Combination of both factors will reduce the dispersion in quality and structural properties of the stabilisation course.

REFERENCES

- Keppens, P. 2002. Berekening levensduur van een DOROPLAN behandelde zandweg (in Dutch). *Letter Report geo001-120*. Brussels: GEOROC Benelux BY

- Koritowski, A. 2003. Gutachterliche Empfehlung/Eignungstest Trag- und Frostschuttschichtersatz Landweg Laarburg-Bavaria unter Verwendung von Doroplan/GeoCrete (in German). *Report AZ 0103/669*. Schöffengrund, Germany: GeoConCept Gesellschaft für angewandte Geotechnik und Consulting mbH.
- RAW Standard Conditions 2000 (including addendum December 2002) Ede, The Netherlands: CROW.
- Van Gurp, C.A.P.M. & Van Drunen, J.C.M. 2003. Research programma GeoCrete stabilisation course Wilhelminaweg in Laarbeek (in Dutch). *Report e0300071*. Apeldoorn, The Netherlands: KOAC•WMD

Aggregate supply and specification

Aggregate supply and performance issues, Auckland, New Zealand

P.Black

Department of Geology, University of Auckland, New Zealand

Pavements Unbound—Dawson (ed.)

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ABSTRACT: Auckland, New Zealand's area of highest population density and growth, also has a backlog of infrastructure projects; thus requirements for quality rock aggregate to satisfy demands for housing, construction, and road building are increasing substantially. All prime local aggregate resources (young basalts) have either been exhausted or sterilised by urban expansion. The other locally available aggregate resource is greywacke, a collective term that includes argillite which the industry perceives to have a particularly high swelling clay content. Since swelling clays are a major source of poor aggregate performance, contract specifications for supply of roadbase aggregate for Auckland roads are often overwritten with the statement that it shall include no argillite, thus effectively ruling out greywacke quarries and forcing consumers to purchase from more distant sources. The cost of road transportation from source to market now averages close to 50% of the average delivered cost. The high plasticity swelling clays impart to roadbase is minimised by modification or chemical stabilisation with lime, steelwork slag and other substances. Current research is focused on determining the nature of the swelling clays in aggregate source rocks and the affect they have on the materials properties of aggregates.

1 INTRODUCTION

Greater Auckland is New Zealand's area of highest population density and growth and it is also among the highest in Australasia. In 2002 nearly 4 million tonnes of aggregate (13% of the total aggregate production in New Zealand) was consumed by Auckland in road building projects. Because of the urgent need for the Auckland region to redress a backlog of infrastructural projects, including several large motorways (some originally planned as far back as the 1960's), and taking into account projected population growth, the area's requirements for quality rock aggregate are expected to increase to about 20 million tonnes per annum by 2020 (Findlay, 2000). This paper reviews some of the issues

related to aggregate supply in the greater Auckland issue and outlines a research project designed to address them.

Auckland city is located on a narrow isthmus between two harbours, the Waitemata and Manukau Harbours, opening into the Pacific Ocean and Tasman Sea respectively (Figure 1). The greater Auckland urban area extends in a broad north-south trending belt and includes three other cities: Manukau to the south, Waitakere in the west and North Shore on the north side of the Waitemata Harbour. The present combined population of these cities is 1,250,000. These four cities are served by a north-south and a northwest-southeast motorway system linked by a common section in the Auckland isthmus. There is a single bridge across the Waitemata Harbour providing access between the rapidly expanding urban area of North Shore and the Auckland and Manukau cities. The particular geography of the region provides substantial constraints on access to, and the provision of, aggregate resources.

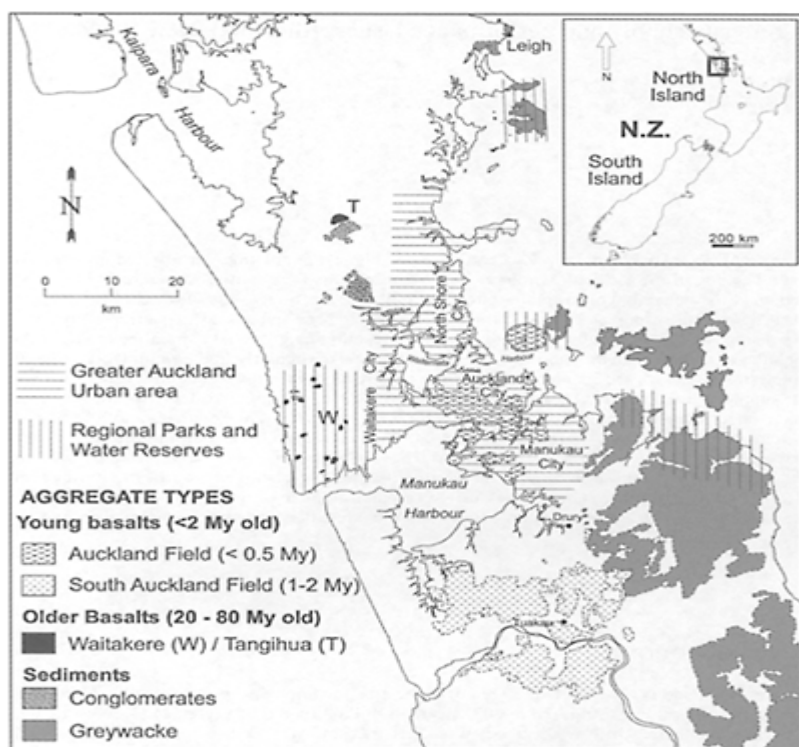


Figure 1. Potential sources of aggregates in the Auckland Region with indications of competing land use that limits the exploitation of the resource.

2 AGGREGATE RESOURCES

The Auckland region has four types of rock which are suitable for use as aggregate.

2.1 *Young basalts*

All less than 2 million years old and erupted subaerially in two discrete fields—the Auckland (isthmus) and South Auckland volcanic fields. Although these two basalt fields contain slightly different rocks, and the South Auckland basalts, being the older of the two fields, have a propensity to be weathered, basalt flows provide strong fine to medium grained rocks and high quality aggregates which are extensively used as pavement sealing chips. Theoretical reserves in excess of 500 million tonnes remain in the Auckland isthmus (Johnson and Happy, 1998) but the resource has been sterilised by urban expansion and the last major quarry in the Auckland isthmus closed in 2001. South Auckland basalts are now being exploited as a replacement supply of high quality aggregates but there are competing land uses for many areas where quarries in these rocks could be sited.

2.2 *Older basalts*

In the 20–80 million year age range, were all erupted in the submarine environment and have been affected by sea water alteration. They contain varying amounts of swelling clay minerals, which have formed mainly from the alteration of volcanic glass. Tangihua basalts, which are of very limited occurrence in the region, have also been intensively deformed and contain alteration minerals on sheer surfaces. Waitakere basalt outcrops of solid rock suitable for quarrying are very limited and most are located in regional reserve areas.

2.3 *Conglomerate*

Horizons are mapped over considerable areas north of Auckland but most are unsuitable for quarrying. Further, the pebbles in the conglomerates are enclosed in a clay-rich matrix which requires extensive processing to remove and the rounded nature and size range of the pebbles limit the grainsize of the aggregate that can be produced.

2.4 *Greywacke*

Is a collective term which includes argillite, siltstone and sandstone. These rocks have been weakly metamorphosed to temperatures in excess of 250°C, which is above the high temperature stability limit of smectite clays (Black et al. 1993). However post peak metamorphic alteration of chlorite in the greywackes has produced a complex interlayer mineral with a swelling component. The physical breakdown (as the result of abrasion) of the greywacke rock in service liberates the chlorite into the fine-grained fraction where in contact with water it is then modified by weathering to a montmorillonite clay-type mineral (Sameshima and Black, 1980). Clay minerals such as chlorite are most abundant in argillaceous rocks. The greywackes are not inherently weak rocks and the coarser

grained siltstones and sandstones have the potential to provide good aggregates but argillites are very common in the sequence and it is often difficult to selectively quarry to avoid them.

3 AGGREGATE PERFORMANCE ISSUES

Swelling clays have been identified by both aggregate producers and the consumer industries in the Auckland region as a major source of poor performance of some aggregates in road construction because of the high plasticity that swelling clays impart to roadbase fines. In New Zealand reading terminology the roadbase is called the basecourse.

Swelling clays in the Auckland region are usually associated with greywacke which is the major aggregate resource available. The swelling mineral in the greywackes is a type of interlayered chlorite—illite—smectite, tentatively called corrensite (Sameshima and Black, 1980) and it is present in unweathered rock and core. Swelling clays also occur in volcanic rocks used for aggregate but in these the swelling clay is a smectite (montmorillonite) and the product either of devitrification of volcanic glass (particularly in older basalts), or of hydrothermal alteration (associated with many andesites to the east of the Auckland region), or of weathering. The occurrence of swelling clays in altered and weathered rocks and the consequent reduction in rock performance that results is not unique to the Auckland region but is exacerbated by the high demand and difficulties in supply of aggregates.

4 AGGREGATE SUPPLY ISSUES

Figure 1 demonstrates the problem with aggregate supply for the Auckland region. Fast expanding urbanisation and consequent need for the construction of new roads—local and motorway—north of Auckland city is taking place in an area which lacks quality aggregate resources. In the Auckland isthmus all the young basalts have either been exhausted or sterilised by urban expansion. Most of the aggregate resources near to the urban areas have also been sterilised by the designation of land as water reserves and regional parks. High quality basalt aggregates are now only sourced from South Auckland where reserves are limited.

The most readily available local aggregate resource is greywacke. However because of a motorway roadbase failure in the mid 1960s (Reed, 1966; Buckland, 1967), attributed to the use of fine grained (argillite) aggregate with a particularly high swelling clay content (Sameshima, 1977), contract specifications for supply of basecourse (i.e. roadbase) aggregate for Auckland roads are often overwritten with the statement that it shall include no argillite, thus effectively ruling out greywacke quarries.

The only area in the region with substantial aggregate resources is South Auckland but if aggregates sourced from there are to serve the expanding northern urban areas they must be transported on the already overloaded motorway system and across the harbour bridge to meet the demand. Aggregates consumed by the Auckland area are now being supplied from quarries more than 100 km to either the north or south of Auckland city.

The cost of road transportation of aggregate from source to market is now averaging close to 50% of the average delivered cost and is projected to increase in the near future (Johnson and Happy, 1998). In order to reduce, or at least contain transportation costs, consumers are being forced to use materials that are often of lower quality than is desirable i.e. marginal aggregates. In New Zealand reading terminology a marginal aggregate is one whose characteristics do not comply with TNZ M/4 Specification (2000).

Transit New Zealand (TNZ) Specification B/3 “Performance Based Specification for Structural Design and Construction of Flexible Unbound Pavements” recognises that the best quality aggregate should not necessarily be used just because it is available and permits the use of modified marginal aggregate as roadbase on state highways but it is required to have satisfactory values for Crushing Resistance, Weathering Quality Index and California Bearing Ratio Tests. In order to meet these specifications aggregates are treated (stabilised) with chemical additives. Additionally, the roadbase aggregate must comply with either a sand equivalent of not less than 40 or a clay index of not greater than 3 or a plasticity index of not greater than 5 when the aggregate is tested according to the specifications for each test as defined in NZS 4407:1991 A.

5 AGGREGATE STABILISATION ISSUES

In the Auckland region marginal aggregates are currently being stabilised with lime derivatives: lime, cement, and a high calcium-titanium steelwork slag.

Lime stabilisation has two actions. The first is on the swelling clay itself where calcium is exchanged into the interlayer position in the clay structure thus reducing the inherent swelling capacity of the mineral. The second is reactions with other phases in the aggregate to form compounds which cement the fines particles. While the cation exchange reaction is well understood, the cementing process is not and the new phases that might form during the curing process will be determined by both the nature of the stabilising compound and the type of aggregate.

There is some concern in the industry that the lime-rich phases produced by the stabilisation process may not be permanent, i.e. survive the design life of the road. In the case of the lime stabilised greywackes, we know that one of the phases formed by the stabilising process is hydrogrossular [$\text{Ca}_3\text{Al}_2(\text{SiO}_4, \text{CO}_3, \text{OH})_3$] (Sameshima and Black, 1982), which has a wide stability range in nature and is unlikely to be converted to other phases while in service. However, there are certainly other phases formed in greywacke aggregates by the stabilisation process but we know very little about them or their stability. The mechanisms and phases formed by the lime stabilisation of basaltic and other aggregates are also extremely poorly known.

6 SUMMARY AND RESOLUTIONS

The aggregate resources available to meet the Auckland region’s road and other infrastructural requirements of the next decade are in large part greywackes i.e. rocks which are considered marginal aggregates and which will require stabilisation. For

specifications allowing use of marginal aggregates in roadbase to be fully embraced by the industry it is clear that the prediction of the likely “in-service” behaviour of the material, including the stabilised aggregate, is a crucial component in any decision to use them. To date there is no good empirical data relating rock mineral composition with the performance of the rock as indicated by standard materials tests. In order to obtain this information a major project, funded by the Foundation of Research Science and Technology, in association with aggregate producers in the Auckland region, will start at the University of Auckland in 2004. This project will:

- undertake a programme of testing using the methods specified by New Zealand Standard 4407:1991 to determine the relationship between mineral content, particularly swelling clay mineral content, and the physical properties of rocks used as reading aggregates
- analyse the relationship between mineral content and performance to determine the boundaries which separate aggregates needing no treatment from those that need to be stabilised to achieve acceptable levels of in service performance and
- investigate chemical stabilisation processes to determine how they react and work with the different rocks used as aggregates in the Auckland Region.

REFERENCES

- Black, P.M., Clark, A.S.B. & Hawke, A.A. 1993. Diagenesis and very low grade metamorphism of volcaniclastic sandstones from contrasting geodynamic environments, North Island, New Zealand: The Murihiku and Waipapa terranes. *Journal of Metamorphic Geology* 11:429–435.
- Buckland, A.H. 1967. The degradation of reading aggregate. *Roading Symposium New Zealand, 1967*, Session L, Preprint 1–30.
- Findlay, M.R.W. 2000. *The aggregate resources of the Auckland Region*. Unpublished undergraduate resource project, Department of Civil and Resource Engineering, University of Auckland.
- Johnson, J.D. & Happy, A.J. 1998. The dawn of a new stone age for Auckland. *Roading Geotechnics* '98:65–70.
- Reed, J.J. 1966. Geological and petrological investigations of wacke aggregate used in Auckland—Hamilton motorway Redoubt and Takanini sections. *New Zealand Geological Survey Report* 20:47 pp.
- Sameshima, T. 1977. Hydrothermal degradation of basecourse aggregate. *National Roads Board Pavement Research Committee Project B.C. 21*, 91 pp.
- Sameshima, T. & Black, P.M. 1980. Hydrothermal alteration of basecourse aggregates and its effect on basecourse performance. *National Roads Board Pavement Research Project B.C. 23*, 125 pp.
- Sameshima, T & Black, P.M. 1982. Stabilisation of aggregates with additives and their effects on fines. *National Roads Board Pavement Research Project B.C. 39*, 108 pp.

Unbound mixtures for pavement layers—BS EN 13285

D.Rockliff & R.Dudgeon

Rock40C Technical Support & Highways Agency

Pavements Unbound—Dawson (ed.)

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ABSTRACT: The Specification for Highway Works (SHW) in the UK is currently being amended to reflect the requirements of BS EN 13285:2003, Unbound mixtures—specification. This paper explains the requirements of the new Standard for UK users. It also explains the link with BS EN 13286, a series of standards detailing test methods for unbound and hydraulically bound mixtures. The changes support the construction industry's transition to the new BS EN standards for aggregates.

1 INTRODUCTION

This paper examines the contents of prEN 13285 (BSI 2003a), the specification for unbound mixtures and outlines complementary requirements from EN 13242 (BSI 2002), the specification for aggregates for unbound and hydraulic bound materials. It also summarises the supporting test methods for the determination of laboratory dry density and water content.

The UK specification for unbound mixtures is the Specification for Highway Works or SHW (Highways Agency etc. 1998a), in particular Series 600, 700 and 800. These changes have recently been issued by the Highways Agency as an Interim Advice Note 52/04 (Highways Agency etc. 2004). This will form the basis of the May 2004 update to the SHW and its associated Notes for Guidance (Highways Agency etc. 1998b).

2 BS EN13285, UNBOUND MIXTURES—SPECIFICATION

2.1 *The standard*

BS EN 13285, Unbound mixtures—specification has been developed by Task Group 2 of CEN/TC 227/WG4. The Task Group quickly realised that the range of practices across Europe is very varied. One specification to suit all countries was not a practical target.

Countries adopt specifications that reflect the characteristics of their locally available aggregates, typical traffic loading and the local climate. For example, subbase layers in France are densely graded with small size aggregates whilst most Scandinavian countries use open graded layers with large size aggregates that are free of water during freezing conditions.

The Standard presents a menu of requirements—it adopts a “pick and mix” approach. A specifier can create a list of requirements by selecting a combination of characteristics to suit local needs. The SHW defines the menu to be used in the UK.

The Task Group also identified examples of subbase specifications that would be considered appropriate as a base or a capping specification in other countries. Because of this, the Standard does not allocate mixtures to a particular pavement layer. National design standards and specifications must do that.

2.2 Mixture designation

BS EN 13285 allows any of the CEN values of upper (D) sieve size between 8 mm and 80 mm to be used, as listed in Table 1. The upper (D) sieve size is equivalent to the nominal aggregate size, not the maximum permitted particle size. This is discussed further in Section 2.4 about oversize requirements.

All the unbound mixtures in BS EN 13285 have a lower (d) sieve size of zero.

2.3 Fines content

Fines content is the percentage of a mixture that passes the 0.063 mm sieve, when tested using the washing and sieving method. There is no better illustration of the variety of specifications in use in across Europe than the menu of maximum fines contents permitted by the Standard and summarised in Table 2.

2.4 Oversize requirements

The specification of oversize controls the percentage of particles larger than the upper (D) sieve size. Four categories are defined in the menu, two based on 1–4 times D and two based on 2 times D , as illustrated in Table 3.

2.5 Overall grading

As Table 1 shows, the specification has to cover fourteen different values of the upper (D) sieve size. The Task Group had to find a systematic way of defining intermediate sieves between the upper (D) sieve size and the 0.063 mm sieve—the sieves that characterise the grading of each mixture. Table 4 shows the intermediate sieves defined for the three mixture designations used in the SHW.

The next step was to define a grading band for each of the intermediate sieves that together make up the grading envelope. The grading requirements also have to allow for different levels of

Table 1. Permitted unbound mixture designations.

0/8	0/10	0/11, 2	0/12, 5	0/14
0/16	0/20	0/22, 4	0/31, 5	0/40
0/45	0/56	0/63	0/80	

Values in bold are used in SHW Series 600 and 800

Table 2. Categories of maximum fines content, UF_X .

Percentage passing	≤ 3	≤ 5	≤ 7
0.063 mm sieve, by mass	≤ 9	≤ 12	≤ 15
	no requirement		

Table 3. Categories of oversize requirements, OC_{XX}

(% passing, by mass).		
$2D$	$1.4 D$	D
–	100	90 min.
–	100	85 min.
100	–	80 min.
100	–	75 min.

tolerance and grading density. For most mixtures, the grading is based around target values for each source of the mixture—the supplier declared value illustrated in Annex B (informative) of the Standard. The chosen solution is illustrated in Table 5 for a mixture with Grading Category G_p .

Despite much effort, it has not been possible to replicate the traditional SHW Type 1 grading envelopes within BS EN 13285. The gradings for UK granular subbase materials were derived from empirical research into typical ‘crusher run’ gradings. Other countries adopt a more theoretical approach based on ‘packing theory’ and Fuller curves.

2.6 Grading requirements for batches

An important principle of the new standards is that they control products when they are placed on the market rather than when received by the customer. Compliance is judged on each production batch—usually the mixture produced in a fixed time period. The supplier will have to operate a system of factory production control. This is essentially a ISO 9001 (BSI 2000) quality management system. Consistency is assured by routinely grading the mixtures and comparing the results using the supplier declared value and tolerances.

For most mixtures, the specification also imposes control over the differences between the values passing adjacent sieves. This protects against gap-graded mixtures.

2.7 Other requirements

The new standard recognises that frost susceptibility of the mixture, permeability of the compacted layer and leaching of potentially harmful components are parameters that need to be

Table 4. Sieves for grading requirements.

Sieve, mm	Mixture designation		
	0/31, 5	0/40	0/80
Sieve A	16	20	40
Sieve B	8	10	20
Sieve C	4	4	10
Sieve E	2	2	4
Sieve F	1	1	2
Sieve G	0.500	0.500	1

Table 5. Grading requirements mixtures with Grading Category G_P, based on supplier declared values.

% Passing sieve, by mass	Overall grading	Range for supplier declared value
Sieve A	43–81	54–72
Sieve B	23–66	33–52
Sieve C	12–53	21–38
Sieve E	6–42	14–27
Sieve F	3–32	9–20
Sieve G	no requirement	no requirement
Tolerance on supplier declared value for sieve sizes		
A, B & C	±15	
E	±13	
F	±10	

considered under certain conditions. At the current time, there is insufficient experience to define a specification that can be used in all parts of Europe, so the continued use of a

National Standard is permitted. The BS 812 frost heave test (BSI 1989) still forms part of the UK national specification.

2.8 Recycled aggregates

The Standard covers recycled aggregates by not specifying individual aggregate sources. If the properties of the aggregates in the mixture meet the requirements of BS EN 13242 and can be graded to fit the requirements for the mixture, the source does not matter. Recycled aggregate is just another member of the greater aggregate family.

Annex A (informative) of the Standard provides some direct support for the use of recycled aggregates by defining a protocol for allocating descriptions to mixtures. This part of the Standard is not a specification, just a way of ensuring that descriptions are used consistently.

3 EN 13242, UNBOUND AGGREGATES

3.1 The standard

BS EN 13242, Aggregates for unbound and hydraulically bound materials for use in civil engineering work and road construction, has been developed by SC4 of CEN/TC 154.

The principle purpose of the specification is to define the characteristics of aggregates that are placed on the market as being suitable for use in unbound and hydraulic bound mixtures. The specification can also be used to define aggregates for secondary uses such as drainage materials. BS EN 13285, the unbound mixture specification, uses a cross-reference to BS EN 13242 to define the required aggregate properties. The aggregates specification also uses menus to reflect the wide range of aggregate types available across the countries of Europe.

3.2 Resistance to fragmentation

The test for resistance to fragmentation is the Los Angeles test. This is a modified version of the American ASTM method, developed in France. Please note that it is an impact test, not an “abrasion” test—the ASTM standard is wrongly named.

Extensive work in UK laboratories has established very good correlation between the Los Angeles method and the Aggregate Impact Value (AIV) test, and good correlation with the Ten Percent Fines Value (TFV) test for most roadstones. Because every particle is exposed to the action of the impact balls in the rotating drum, the method readily identifies aggregates with a proportion of weak particles. Weaker fractions are often shielded by stronger particles in the confined cylinders of the superseded BS tests.

3.3 Resistance to wear

The resistance to wear of coarse aggregates is determined using the micro-Deval test developed in France. The test uses a ball mill to abrade aggregate particles in water. The test is particularly suited to unbound mixtures because it imposes similar abrasive

conditions to those experienced by the surface of interlocking aggregate particles in a saturated and trafficked unbound layer. Like the Los Angeles test, all particles are equally exposed to the action of the ball charge and weaker fractions are exposed. It is possible that the test may prove to be more effective at identifying unsound aggregates than the Magnesium Sulfate Soundness Test. It is certainly a lot quicker and the marginal cost of each test is much less.

There is insufficient experience with the micro-Deval test to define reliable specified requirements at this time, but data should be collected so that its future use can be evaluated.

3.4 Durability requirements

Durability is specified in two clauses of the aggregate specification. The first clause uses a test to identify “Sonnenbrand” of basalt; the second considers the resistance to freezing and thawing of aggregate particles using either a freeze-thaw test or the Magnesium Sulfate Soundness test. It should be noted that the freeze-thaw test considers the breakdown of aggregate particles, not the potential frost heave of a compacted mixture.

3.5 Fines quality

All of the TC1 54 specifications for aggregates include guidance on the assessment of harmful fines. These clauses use the sand equivalent test (BS EN 933–8) and methylene blue test (BS EN 933–9). Tests for the assessment of fines are widely used in parts of France, Belgium and the Netherlands. There are concerns about the relevance of the tests to many UK aggregates. Work to date has not been conclusive.

4 EN 13286, TEST METHODS FOR UNBOUND MIXTURES

4.1 Laboratory dry density and water content

BS EN 13285 specifies unbound mixtures placed on the market. It does not specify the properties of the compacted layer. This is a matter for national road design standards.

However, BS EN 13285 recognises that the purchaser of a mixture needs information about laboratory dry density and water content to assist the site operations. The specification requires the supplier to use the system of factory production control to monitor laboratory dry density and water content at least once each year and declare values, when requested by the customer.

It is also recognised that a declared value is only meaningful if it is associated with a test method which is carried out in the same way in each country. There are many methods used throughout Europe and lots of variations to what are considered as standard tests—one man’s Proctor test never seems to be the same as his neighbour’s!

BS EN 13286 (BSI 2003, 2004 and in prep.) is made up of many parts covering a wide range of tests for unbound and hydraulically bound mixtures. Two of them are most relevant to UK practice for unbound mixtures. The vibrating hammer method is based on established techniques. The vibrating table method is derived from experience in

Denmark and is based on the ASTM method. For many years UK suppliers have been expected to supply data on capping layer mixtures that are too large for the mould used in the vibrating hammer test. The vibrating table would seem to be a sensible and practical alternative.

4.2 Repeated load triaxial test

Members of the Task Group have been very involved with the development of the repeated load triaxial test for unbound mixtures. Although not called up by the specification, it was felt appropriate to document the test and signal its potential as a basis for future performance based specifications. The method has recently been published as BS EN 13286–7 (BSI 2004).

5 UK IMPLEMENTATION—THE SPECIFICATION FOR HIGHWAY WORKS

5.1 Series 600, earthworks

The introduction of BS EN 13285 requires separate classes for imported Class 6F granular capping material. Clause 613 and Table 6/1 have been amended to specify two new classes:

- 6F4 for capping—a 0/31.5 unbound mixture to BS EN 13285, similar to the current 6F1 but imported onto the Site.
- 6F5 for capping—a 0/80 unbound mixture to BS EN 13285—similar to the current 6F2 but imported onto the Site.

A new SHW Table 6/5 gives the grading requirements for Classes 6F4 and 6F5 materials. The properties of the aggregates used in these mixtures are defined using BS EN 13242 and the guidance in PD 6682–6 (BSI 2003b). SHW Table 6/1 has numerous minor amendments, including the replacement of Ten Percent Fines Value by the equivalent Los Angeles value.

Asphalt arisings are not permitted in Class 6F4 and 6F5 materials. 6F3 capping made with asphalt arisings has not been changed because it has the necessary additional controls on compaction characteristics.

5.2 Series 700, road pavements—general

The classification test for recycled aggregates defined by SHW Clause 710 has been extensively revised to remove an anomaly in the calculation. The method now includes detail from a proposed BS EN method that will be published as BS EN 933–11 (BSI in prep.) in due course. The quality control protocol referred to in Clause 710 has been updated and is now published by WRAP (Waste and Resources Action Programme). It is available on the WRAP website.

5.3 Series 800, unbound mixtures for subbase

5.3.1 Structure of Series 800

Series 800 has been totally revised to reflect the requirements of BS EN 13285 and BS EN 13242. The changes give five new clauses:

- 801, General requirements for unbound mixtures for subbase.
- 802, Transport, laying, compaction and trafficking of unbound mixtures for subbase.
- 803, Type 1 unbound mixtures for subbase.
- 804, Type 2 unbound mixtures for subbase.
- 805, Type 3 (open graded) unbound mixtures for subbase.

The technical contents of Clauses 806, 850SE and 850NI have been incorporated into the revised clauses. Clause 806 for Type 4 granular subbase material made from asphalt arisings has been deleted; its requirements are now included in Clause 804. Clause 805, Slag Bound Material has been renumbered, with minor changes, as Clause 809, Slag Bound Mixtures.

5.3.2 Clause 801, general requirements for unbound mixtures for subbase

Three size and grading category combinations for unbound mixture have been selected from the many permitted by BS EN 13285. Sub clause 801.1 sets out the general requirements for each one, using the categories defined in BS EN 13285. SHW Table 8/1 is replicated as Table 6.

Table 6. Mixture and grading requirement categories for unbound mixtures for subbase.

Unbound mixture	Type 1	Type 2	Type 3 (open graded)
Clause	803	804	805
Standard	BS EN 13285 Categories for unbound mixture properties		
Mixture requirement category			
– Designation	0/31.5	0/31.5	0/40
– Maximum fines	UF_9	UF_9	UF_5
– Oversize	OC_{75}	OC_{75}	OC_{80}
Grading requirement category			
– Overall grading	G_P	G_E	G_o

For Type 1 and Type 2 unbound mixtures, the opportunity has also been taken to move from 37.5 mm size to 31.5 mm size mixtures. This reflects recent trends in production practice and seeks to minimise the risks of segregation. Decisions about the appropriate choice of mixture size/grading category for UK use have considered potential compliance with production control tolerance as well as the overall grading.

Although SHW Table 8/1 is sufficient to define the grading of the chosen unbound mixtures for subbase, it has been felt appropriate to replicate the grading envelopes as tables within the clauses for individual mixtures, to aid the transition to BS EN 13285.

A new SHW Table 8/2 links the clauses with the aggregate properties required by BS EN 13242, using the recommendations from PD 6682–6. Guidance on the aggregate property subclauses is included in the Notes for Guidance on the SHW.

Sub clause 801.6 collects the requirements for CBR from a number of existing clauses into a general clause. It can be triggered by a contract Appendix 7/1. Sub clause 801.7 and 801.8 are the sexisting frost heave clauses with an amended source of reference filler.

5.3.3 Clause 802, Transport, laying etc.

This clause contains (without technical amendment) the “contractor” requirements from the current SHW clause 801. This reverses a change made a few years ago that left clause 802 unused. SHW Table 8/4 is the previous Table 8/1, without technical change.

Sub clauses 802.12 to 802.18 consolidate the trafficking trial clause previously in Clauses 806 (Type 4, crushed asphalt) and 850SE (crushed gravel), for use when triggered by sub clauses in Clause 803, Clause 804 and contract Appendix 7/1.

5.3.4 Clause 803, Type 1 unbound mixtures for subbase

Type 1 is the standard unbound mixture for use in subbase layers. The revised grading rules in SHW Table 8/5 are replicated in Table 7. The plasticity requirement replicates the current sub clause. BS EN 13285 specifically permits this. Sub clause 803.7 incorporates the crushed gravel requirements from 850SE into the new clause. CBR and trafficking trial requirements for crushed gravel mixtures are triggered by contract Appendix 7/1.

Table 7. Summary grading requirements for Type 1 unbound mixtures for subbase.

Sieve size, mm	Percentage by mass passing		
	Overall grading range	Supplier declared value grading range	Tolerance on the supplier declared value
63	100		
31.5	75–99		
16	43–81	54–72	±15
8	23–66	33–52	±15
4	12–53	21–38	±15
2	6–42	14–27	±13
1	3–32	9–20	±10
0.063	0–9		

Grading of individual batches—differences in values passing selected sieves			
Retained sieve size, mm	Passing sieve size, mm	Percentage by mass passing	
		Not less than	Not more than
8	16	7	30
4	8	7	30

5.3.5 *Clause 804, Type 2 unbound mixtures for subbase*

Type 2 continues to be available for more lightly trafficked roads. The Notes for Guidance and the pavement design guidance in HD 25 (Highways Agency etc. 1994) place a limit of 5 msa on design traffic levels (as they did previously) unless local experience indicates that a higher value may be appropriate. Clause NG 804 makes specific reference to the proven performance of subbase made with a high proportion of asphalt arisings.

Sub clause 804.6 uses Appendix 7/1 to trigger a minimum CBR requirement, as now. The current moisture content requirements are replicated in 804.7, using the declared values required by BS EN 13285.

The grading category selected for Type 2 does not use the supplier declared value/tolerance approach. A simpler approach is appropriate for lower grade end uses. It will usually be impractical to consider a complex system of factory production control for mixtures with a high proportion of asphalt arisings because of the potential variability of the source “aggregate”. Such mixtures are therefore covered by sub clauses 804.8 to 804.11, where permitted by contract Appendix 7/1. The requirements are taken from the superseded Type 4 clause. Contract Appendix 7/1 can also trigger a trafficking trial for asphalt arisings mixtures.

5.3.6 *Clause 805, Type 3 (open graded) unbound mixtures for subbase*

This clause transfers the requirements from the superseded Type 3 clause used in Northern Ireland, with crushed blast furnace slag and crushed concrete added to the permitted constituents list. Similar requirements have also been issued at times as Type 1X. A 0/40 size mixture has been chosen to give a potentially higher value of d_{10} (a key permeability parameter) than would be the case with a 0/31.5 mm size mixture.

6 CONCLUSIONS

The changes to the Specification for Highway Works support the construction industry’s transition to the new BS EN standards for aggregates. It completes a lengthy process to consolidate specifications for unbound mixtures across Europe.

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REFERENCES

- British Standards Institution 1989. BS 812:Part 124, *Testing aggregates, Method for determination of frost heave*. London: BSI.
- British Standards Institution 2000. BS EN ISO 9001, *Quality management systems—specification*. London: BSI.
- British Standards Institution 2002. BS EN 13242, *Aggregates for unbound and hydraulically bound materials for use in civil engineering work and road construction*. London: BSI.
- British Standards Institution 2003a. BS EN 13285, *Unbound mixtures—specification*. London: BSI.
- British Standards Institution 2003b. PD 6682–6, *Aggregates—Part 6: Aggregates for unbound and hydraulically bound materials for use in civil engineering work and road construction—Guidance on the use of BS EN 13242*. London: BSI.
- British Standards Institution, 2003, 2004 and unpublished. BS EN 13286, *Unbound and hydraulic bound mixtures, Test methods etc*. London: BSI.
- British Standards Institution, 2004. BS EN 13286–7, *Unbound and hydraulic bound mixtures, Cyclic load triaxial test for unbound mixtures*. London: BSI.
- British Standards Institution, unpublished. BS EN 933–11, *Tests for geometrical properties of aggregates—Part 11: Classification test for the constituents of coarse recycled aggregate*. London: BSI.
- Highways Agency etc. 1994. *Design Manual for Roads and Bridges, Pavement design and maintenance, Foundations, HD25/94*. London: The Stationery Office.
- Highways Agency etc. 1998a. *Manual of Contract Documents for Highway Works. Volume 1, Specification for Highway Works*. London: The Stationery Office.
- Highways Agency etc. 1998b. *Manual of Contract Documents for Highway Works. Volume 2, Notes for Guidance on the Specification for Highway Works*. London: The Stationery Office.
- Highways Agency etc. 2004. *Interim Advice Note IAN 52/04, Changes to Aggregate and Concrete Specification affecting MCHW Series 500, 600, 700, 800, 1700 (and associated Notes for Guidance)*. London: The Stationery Office.

Material and performance specifications for wearing-course aggregates used in forest roads

G.Légère & S.Mercier

*Forest Engineering Research Institute of Canada (FERIC), Pointe-Claire,
QC, Canada*

Pavements Unbound—Dawson (ed.)

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ABSTRACT: A literature review was conducted to help develop a performance-based specification for materials intended to be used as the wearing course in forest roads built from unbound aggregate. A survey of eastern Canadian forestry companies revealed that few use appropriate specifications; instead, most use specifications that are designed for base-course layers and that typically lack sufficient plastic fines. Two aggregate wearing-course materials produced to meet different specifications were tested on a 10.2-km section of haul road with high traffic levels. A detailed performance evaluation and an analysis of the rate of road deterioration through the collection of surface roughness data revealed that both materials performed exceptionally well and significantly improved roads versus those without the specified wearing-course materials.

1 INTRODUCTION

Haul distances to the mill are increasing every year, and several Canadian forest companies must maintain more than 300 km of unpaved (unsealed) roads, on which some haul considerably more than 1 million metric tonnes of wood per year. The performance of these roads directly affects trucking costs and productivities, but the forest industry has little expertise in selecting an appropriate specification for the aggregates used on these roads. Instead, most companies use specifications provided by local and provincial agencies that were not necessarily designed for use on unpaved roads. The performance of unbound wearing-course materials in forest roads has been a growing concern for many Canadian companies. The lack of readily available high-quality materials, combined with heavy axle loads, high traffic levels, and frequent grading, make it challenging and expensive to keep haul roads in good condition. Thus, the industry needs more appropriate aggregate specifications.

The main objective of a study conducted by the Forest Engineering Research Institute of Canada (FERIC) was to establish a performance-based specification for the aggregates that would be used as a wearing course on forest roads. To achieve this:

- a literature review to identify existing specifications was conducted,
- the aggregates and specifications currently being used in eastern Canada were evaluated,
- a full-scale performance evaluation of two recommended specifications was performed in the operations of Bowater Canadian Forest Products Ltd. (Mistassini, QC, Canada), and
- recommendations for the future production, application and maintenance (grading) of wearing-course aggregates were formulated.

2 LITERATURE REVIEW

A literature review of aggregate specifications designed for wearing-course applications (i.e., unsealed surfaces) and for base-course applications was conducted. The review showed that few provincial agencies in eastern Canada provide specifications designed for unsealed surfaces (e.g., for wearing courses rather than for highway shoulders); most only provide specifications suitable for the base- and sub-base layers. Specifications for the latter two layers generally lack sufficient fines (materials that can pass through a 0.075-mm sieve), which are needed to provide good cohesion (binder) between particles if the aggregate is to be used as a wearing course. Since base layers must drain freely, their fines content is generally kept under 5%.

In a review of wearing-course specifications used around the world, most specifications had similar criteria. More fines were required in the surfacing layer, with a desired range of 8 to 15% (Tyrrell 2000) or 4 to 15% (Selim 2000). These fines must also contain plastic materials (clays) to improve their cohesion (Ferry 1986). The recommended plasticity index (PI) for these clays has been reported by various authors: between 4 and 9 (AASHTO 2001, Tyrrell 2000), between 4 and 12 (Selim 2000) and between 4 and 15 (Giummarra 1993). As well, the liquid limit should not exceed 35% (AASHTO 2001, Tyrrell 2000).

Other important requirements for wearing-course materials have been identified (Netterberg and Paige-Green 1988): “the ability to provide an acceptably smooth and safe ride without excessive maintenance, stability in terms of resistance to deformation under both wet and dry conditions, an ability to shed water without excessive scouring, resistance to the abrasive action of traffic and erosion by water and wind, freedom from excessive dust, and freedom from excessive slipperiness in wet weather”. The following physical characteristics are also required (PaigeGreen 1999):

- a particle-size distribution that permits a good interlock between particles without excessive amounts of fine or coarse material,
- appropriate cohesion so as to resist raveling,
- adequate material strength so as to resist shear failures, and
- adequate aggregate hardness so as to retain the structural integrity of the compacted material.

Figure 1 compares a typical base-course specification (Ontario's granular "A"; MTO 1993) with a typical wearing-course specification (Selim 2000). In this figure, the recommended range of compositions for wearing courses has higher overall proportions of fines, sands, and fine gravels than the corresponding range for base courses. This figure also highlights various particle-size distributions that are prone to different kinds of surface-distress problems.

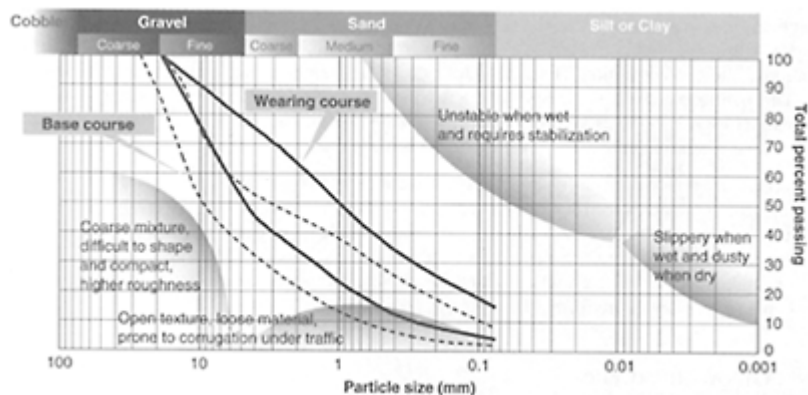


Figure 1. Typical specifications for the range of particle-size gradations for a base course and a wearing course, and size distributions that typically pose surface-distress problems (Légère and Mercier 2003).

3 SURVEY AND CHARACTERIZATION OF MATERIALS

Several eastern Canadian forestry companies were surveyed and samples were collected (following standard BNQ 1982) from their aggregate stockpiles for laboratory characterization. Some results were provided by independent labs working for the companies. The results were compared to nine provincial base-course specifications and five wearing-course specifications from various international sources. The analysis (Légère and Mercier 2003) revealed that:

- Most of the companies were using specifications designed for base-course applications.
- Only 31% of the samples met the criteria in one or more of the nine provincial specifications.
- Only 14% of the samples met the company's own specifications, which suggests that quality control during the production of crushed materials is suboptimal.
- Only 11% of the samples met one or more of the five wearing-course specifications.
- None of the samples measured contained plastic fines.
- Few companies were aware of the importance of adding *plastic* fines to their mixtures.

4 FIELD PERFORMANCE EVALUATION

A full-scale evaluation of field performance followed this survey. Two wearing-course specifications were tested. The project began during the summer of 2003, and focused on a 10.2-km section of road with a surface comprising 200 to 250 mm of aggregate.

4.1 Laboratory characterization and quality control

Table 1 presents the two selected specifications. The MG20B specification (MTQ 2000) is used by several companies in Québec and is designed for unsealed highway shoulders. Although no plasticity index is required to meet this specification, we used a range of 4 to 12 based on the results of the literature review. The South Dakota Gravel Surface (SDGS) specification (Selim 2000) was also

Table 1. Target specifications for two wearing-course standards and average sieve-analysis results from samples taken during production and during on-site application.

Particle size (mm)	Percent passing					
	MG20B	SDGS				
	Spec (min. to max.)	During production	After application	Spec (min. to max.)	During production	After application
31.5	100	100	100	100	100	100
25.4	96–100	98	96	100	(98)	(96)
19	87–99	91	91	100	(92)	(90)
12.7	64–90	70	75	84–93	(75)	(69)
4.75 (sieve #4)	34–59	48	54	50–78	50	48
2.00 (sieve #10)	23–45	38	40	35–63	42	39
0.425 (sieve #40)	11–21	(22)	(25)	13–35	26	25
0.075 (sieve #200)	5–11	9	9	4–15	12	13
Plasticity index (PI)	4–12*	5	n.a.	4–12	5	n.a.
Grading coefficient	16–34	29	31	16–34	29	28

(Gc)**						
Dust ratio	0.4–0.6	0.4	0.4	0.4–0.6	0.5	0.5
(DR)***						

* Not required to meet the provincial standard, but recommended by the authors.

** $G_c = [(\% \text{ passing } 26.5 \text{ mm} - \% \text{ passing } 2.0 \text{ mm}) \times \% \text{ passing } 4.75 \text{ mm}] / 100$ (Paige-Green 1999).

*** $DR = \% \text{ passing } 0.075 \text{ mm} / \% \text{ passing } 0.425 \text{ mm}$ (Thompson and Visser 2000).

Table 2. Results of additional laboratory and field measurements for all test sections.

Test	Average value
Laboratory	
Liquid limit	20%
Optimum moisture content (modified Proctor)	5%
Micro-Deval (large aggregate)	4.3%
Los Angeles abrasion	22.0%
Mineralogy	Limestone
In situ	
Moisture content during compaction	4%*
% of modified Proctor density	96
Clegg Impact Value (CIV) (top 150- to 200-mm lift)	63
California Bearing Ratio (CBR) correlated from CIV (top 150- to 200-mm lift)	257
Young's modulus (measured with a GeoGauge, top 230- to 310-mm lift)	96 MPa
Road width (including shoulders)	14.2 m
Running-surface width	9.3 m
Surface crown	4%

*Compacted sections.

selected since it was designed specifically for wearing-course applications and had criteria similar to those of other wearing-course specifications. A total of 18 8001 of material (to be applied over 9.5 km of road) were crushed to meet the MG20B specification, versus 1200 t (to be applied over 0.7 km of road) to meet the SDGS specification. Within the section surfaced with MG20B, two sections were treated with dust suppressants (1.5 km with calcium chloride and 1 km with Solnat calcium chloride – based brine solution) immediately after regravelling. Aggregate samples were collected during production to permit laboratory characterization of the materials. Sieve analyses were performed on all samples (Table 1). Sieve analyses were also performed on samples collected during the application of the material to monitor whether the material had been

stockpiled correctly and whether the material gradation had changed as a result of segregation during storage or transportation.

The sieve analysis showed that the materials generally met the MG20B standard, though slightly (1%) more particles than recommended passed through the #40 sieve (0.425 mm). The results for the SDGS specification showed that the requirement for a higher fines content than in the MG20B specification was met (based on #40 and #200 sieves). Fines were hauled from a nearby source and added to the blend during production. However, the requirements for the larger particle sizes (particles passing 12.7, 19 and 25.4-mm sieves) were not met; instead, the proportions were similar to those in the MG20B samples, which suggests that the producer failed to adjust his equipment to meet the SDGS specification.

Thompson and Visser (2000) found that the material parameters of plasticity and grading are the primary factors that control a haul road's functional performance. Variations in the grading coefficient (G_c), dust ratio (DR), and PI contributed to the rate of increase or decrease in defect scores. Both aggregates in this study met the PI requirements and the G_c and DR values recommended by Paige-Green (1999) and Thompson and Visser (2000), respectively.

The results calculated from samples collected during application of the aggregates suggested that the materials had been handled properly so as to minimize segregation, since the values were close to those measured during production of the aggregate. Additional laboratory characterizations and field measurements were also conducted, and the results are presented in Table 2.

4.2 Traffic

Approximately 300 vehicles per day use this road, including 75 haul trucks (Table 3). The truck traffic is a mixture of heavy (oversized) 7-axle off-highway trucks and 8-axle on-highway trucks that conform with regulated legal load and size limits.

Table 3. Annual traffic data.

Volume of wood hauled per year (m^3)	1.25 million
Metric tonnes of wood hauled per year	1 million
Average Annual Daily Traffic (AADT) (75% passenger vehicles and forestry workers)	300
Off-highway haul trucks (167 t loaded)	5685 trips
Legal on-high way B-train haul trucks (61 metric tonnes loaded)	9366 trips

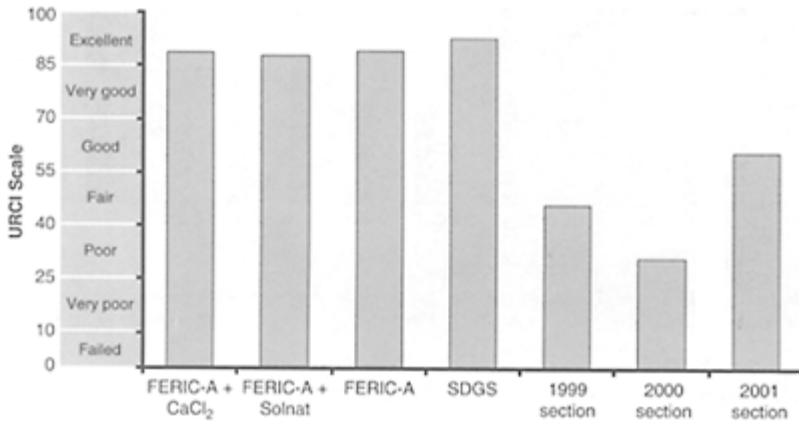


Figure 2. Average Unsurfaced Road Condition Index (URCI) values for four test sections and three older (resurfaced) sections.

4.3 Detailed performance evaluation

Five detailed evaluations of the road were conducted from July 18 to October 29 using the Unsurfaced Road Condition Index (URCI) method developed by the U.S. Army Corps of Engineers (Eaton *et al.* 1988). Further evaluations were not possible after October 29 because snowfall began in early November. Evaluations will resume in 2004.

The following seven types of surface distress were measured using the URCI system: changes in cross-section, roadside drainage, corrugation, dust, potholes, ruts, and loose aggregate. Both the severity level (low, medium, and high) and the frequency (number of occurrences or density per unit area) were measured for each type of distress for a specific surface area. Based on these inputs, we used the URCI system to rate the road on a scale from 0 to 100 (Figure 2).

Systematic sampling was conducted throughout each section (Figure 2). No significant differences were noted between test sections, which all scored around 90 (Excellent) on the URCI scale. The two subsections treated with dust suppressant performed similarly. For comparison, three sections graveled in 1999, 2000, and 2001 were also evaluated; these scored 46 (Fair), 31 (Poor), and 61 (Good), respectively. These sections had been surfaced with a crushed aggregate that met the MG20B specification, but without the addition of plastic fines.

The only surface distresses in the test sections were potholes of low to medium severity. No dust was noted in the subsections treated with dust suppressants, and only light dust production was noted in the untreated MG20B and SDGS sections; this dust production was not sufficient to be counted as a defect according to the URCI method. Surface distresses in the 1999, 2000 and 2001 sections were potholes of low to medium severity and dust of low severity.

Short segments were mechanically compacted within the MG20B and SDGS sections to evaluate the cost-effectiveness of surface compaction (not a standard practice for forest roads). No

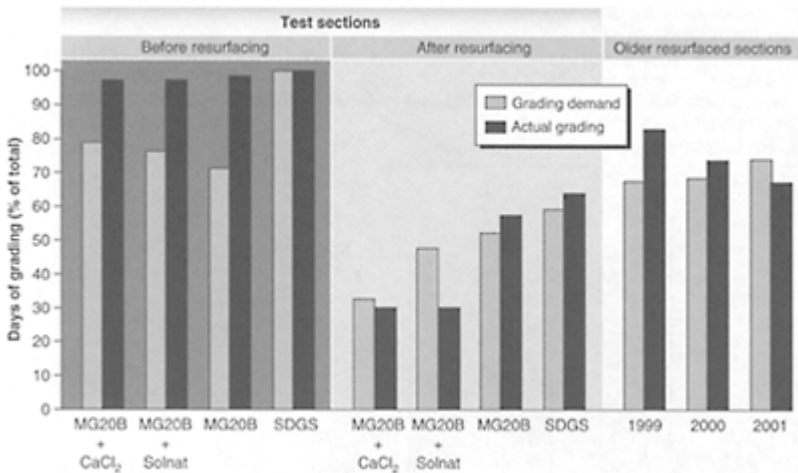


Figure 3. Grading demand estimated using the *Opti-Grade RF* system and actual grading (% of total number of days based on 5 days/week) for the test sections and for three older resurfaced sections.

differences in performance were measured after a short-term assessment. After only a few days of high traffic, the densities of the non-compacted sections were equivalent to those in the compacted section. Given that grading is conducted frequently to correct the rapid development of roughness, the benefits of surface compaction (increased density) would soon be lost.

4.4 Development of road roughness

FERIC's *Opti-Grade RF*TM grading-management tool (Mercier and Brown 2002) was used to monitor the daily development of road roughness, and grading interventions. Figure 3 shows the number of days (expressed as a percentage of the total) that grading took place in each test section, before and after resurfacing, as well as in three older sections of road resurfaced in 1999, 2000, and 2001. Using roughness data collected for each section, the *demand* for grading was also analyzed based on a roughness threshold selected by the forest company. Before resurfacing, grading was typically carried out almost daily (>97% of the time) based on a 5-day work week, but these analyses found that grading could have been reduced by as much as 27% for most sections. Resurfacing reduced the grading demand by as much as 58%. Grading is now carried out only 30% of

the time in the sections treated with dust suppressants and only 57% of the time in the remaining MG20B section. Grading in the SDGS section decreased from 100% of the time to 64%, but these data were for a short (700-m) section of road located on a slight uphill grade, with vehicles travelling loaded uphill. This could explain why both grading demand and actual grading for the SDGS section were higher than in the other test sections before and after resurfacing. Grading demand in the older resurfaced sections was close to 70%.

The company performs two levels of grading intervention: “surface grading” and “full-depth grading”. Surface grading, which is done several times per week, returns loose aggregate available on the shoulders of the road to the running surface, thereby superficially patching the occasional potholes and washboards. This is typically a one-pass method, and the authors have found that this does not really reduce the average roughness of the road, though it may slow down the development of roughness. Full-depth grading is a more aggressive approach in which the grader’s blade digs deeper into the surface, below the potholes, thereby reclaiming the first 50 to 100 mm. This

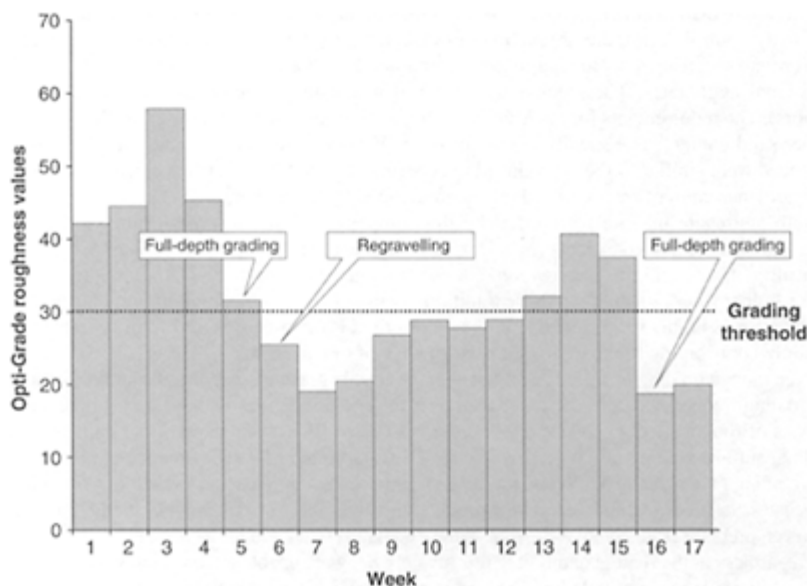


Figure 4. Average weekly roughness values for the MG20B section measured with *Opti-Grade RF* for 17 weeks beginning on June 5, 2003.

technique generally requires several passes, but it restores the road’s crown and reduces the average roughness. It is best to conduct such work under moist conditions.

To illustrate the differences between surface and full-depth grading, we extracted the average weekly roughness values for the MG20B section (the longest and most representative section of our study), presented in Figure 4. For weeks 1 to 3 (before

resurfacing), the average weekly roughness increased each week even though surface grading occurred almost every day. A full-depth grading was conducted at week 5, followed by a regravelling treatment in week 6. This rehabilitation lowered the average weekly roughness levels to well below the grading threshold. From week 7 to week 15, the average weekly roughness steadily increased, and had exceeded the grading threshold by week 13. During this period, surface grading was practiced approximately every 2 days (i.e., 57% of the time). Full-depth grading conducted at week 16 again restored the road's condition and reduced the average roughness to well below the threshold.

We recommended reducing surface grading by following the schedules provided by *Opti-Grade RF*. Except for sections treated with dust suppressants and the 2001 sections (Figure 3), actual grading always exceeded the grading demand after resurfacing. Surface grading didn't improve trucking productivity (as measured by travel speed), but full-depth grading did. Surface grading may also break down aggregate into finer particles, thereby reducing its life.

5 CONCLUSIONS AND RECOMMENDATIONS

Forest companies must maintain a reliable road network to ensure timely delivery of fresh wood to their mills. Trucking costs are thus weighed against road maintenance costs so as to find the most cost-effective combination. Because the performance of haul roads relates directly to the condition (roughness) of the running surface, the use of a proper specification for the wearing-course materials is a key factor in achieving good results.

This study demonstrated to the forestry company and to the aggregate provider the importance of quality control during the production of crushed aggregate and the importance of selecting an appropriate specification. Both aggregates contained a higher fines content than in the company's traditional aggregates. The fines added were of adequate plasticity (as recommended by the authors). The requirements for the MG20B specification were met, with the exception of a slightly higher sand content. The specifications for the SDGS mixture were not met, but the aggregates produced were similar to those produced in accordance with the MG20B specification. However, a higher fines content was achieved (as recommended by the authors).

Both aggregate materials produced by the company are currently performing exceptionally well, and have surpassed expectations. During every detailed performance evaluation, the test sections always scored in the *Very good to Excellent* range based on the URCI. These aggregates all have a higher fines content than those traditionally used by the company. Although the addition of plastic fines is highly recommended in the literature, this practice was new to the company and is certainly contributing to the improved performance of the materials.

Even though grading is much less frequent in the freshly graveled sections, the authors believe that the grading frequency could still be reduced by 8 to 10% on sections without dust suppressants. An analysis of roughness data collected with the *Opti-Grade RF* system showed that unlike full-depth grading, surface grading doesn't reduce the road's overall roughness. The practice of near-daily surface grading may actually accelerate the deterioration of the aggregate as a result of grinding of the aggregates, and doesn't improve trucking productivity; thus, this form of grading should be decreased. However,

pressure from some of the road's users demands that a grader be present on the road every day, regardless of the road's condition. Unfortunately, we were unable to control this factor.

The following recommendations are provided for companies seeking to produce optimal aggregate mixtures for use as a wearing course on unsealed roads:

- Choose an aggregate specification designed for wearing-course applications and adapted for regional conditions.
- Aim for a fines content (the percentage that passes a 0.075-mm sieve) of 4 to 15% by weight.
- Use plastic fines with a plasticity index of 4 to 9.
- Control the quality of the aggregates by conducting quality-assurance inspections during the production of this material. This is the key to producing a material that performs well.
- Ensure that the method of stockpiling and handling of the aggregate minimizes segregation of the component materials.
- Light surface grading should be kept to a minimum and should follow grading schedules based on road roughness rather than on “politics”.

REFERENCES

- AASHTO. 2001. *Standard specifications for transportation materials and methods of sampling and testing*. Part I—specifications. 21st ed. American Association of State Highway and Transportation Officials, Washington, D.C. Unpaginated.
- BNQ. 1982. *Échantillonnage*. Bureau de normalisation du Québec, Québec, QC. NQ-2560-010. 4 p.
- Eaton, R.A.; Gerard, S.; Cate, D.W. 1988. *Rating unsurfaced roads: a field manual for measuring maintenance problems*. U.S. Army Corps of Engineers, Cold Regions Research and Engineering Laboratory (CRREL), Hanover, New Hampshire. Special Report 87-15. 33 p.
- Ferry, A.G. 1986. *Unsealed roads – A manual of repair and maintenance for pavements*. R.R.U. Technical Recommendation TR/8. Road Research Unit, National Roads Board. Wellington, New Zealand. Unpaginated.
- Giummarra, G. 1993. *Unsealed roads manual: guidelines to good practice*. Australian Road Research Board Limited (ARRB), Vermont, South Victoria, Australia. 62 p.
- Légère, G.; Mercier, S. 2003. *Improving road performance by using appropriate aggregate specifications for the wearing course*. Forest Engineering Research Institute of Canada (FERIC), Pointe-Claire, QC. Advantage 4(13). 6 p.
- Mercier, S.; Brown, M. 2002. *The Opti-Grade(r) grading-management system*. Forest Engineering Research Institute of Canada (FERIC), Pointe-Claire, QC. Advantage 3(17). 4 p.
- MTO. 1993. *Material specification for aggregate—Granular A*. Ontario Ministry of Transportation, Toronto, Ontario. Ontario Provincial Standard Specification 1010. 4 p.
- MTQ. 2000. *Matériaux granulaires pour fondation, sous-fondation, couche de roulement granulaire et accotement*. Ministère des Transports du Québec, Ste-Foy, QC. Norme 2102, Tome VII, Chapitre 2.1—Granulats. 2 p.
- Netterberg, F.; Paige-Green, P. 1988. Wearing courses for unpaved roads in southern Africa: a review. *Proc. 8th Quinquennial Convention of the South African Institute for Civil Engineers and Annual Transportation Convention*, Pretoria, South Africa, Vol. 2D.

- Paige-Green, P. 1999. Geological factors affecting performance of unsealed road materials, pp. 10–15. In: *Seventh International Conference on Low-Volume Roads*, Baton Rouge, Louisiana. Transportation Research Board, Washington, D.C.
- Selim, A.A. 2000. *Gravel roads: maintenance and design manual*. U.S. Department of Transportation, Federal Highway Administration, South Dakota Local Transportation Assistance Program, Bookings, South Dakota. 64 p.
- Thompson, R.J.; Visser, A.T. 2000. *The functional design of surface mine haul roads*. University of Pretoria, Pretoria, South Africa. 32 p.
- Tyrrell, R.W.W. 2000. Aggregates for forest roads, pp. 335–341. In: Dawson, A.R. (editor). *Proceedings of the fifth international symposium on unbound aggregates in road construction (UNBAR5)*. University of Nottingham, Nottingham, United Kingdom. 21–23 June 2.

An end product specification for road foundations

B.C.J.Chaddock & D.B.Merrill

TRL, Crowthorne, UK

Pavements Unbound—Dawson (ed.)

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ABSTRACT: Research has been conducted on behalf of the Highways Agency to develop an end product specification for road foundations that incorporate unbound and hydraulically bound materials. This study has identified the primary functional requirements of road foundations as providing a construction platform and long-term support to the pavement. The individual test techniques and procedures to ensure compliance of road foundations at construction are described and justified with illustrative examples and then the main features of a proposed end product specification are outlined. The application of the main elements of the specification to prove a foundation of an in-service road is described. The current limitations of the specification applied during pavement construction to provide the client assurance of the quality of the foundation throughout its service life are outlined. Finally, the potential benefits for client and contractor of introducing an end product specification for road foundations are summarized.

1 INTRODUCTION

The foundation and pavement are designed to protect the natural soil from excessive stresses caused by traffic as well as adverse environmental conditions. With regard to traffic loads, these structural layers reduce the stresses on the soil by spreading the traffic loads over a larger area than the contact areas of vehicle tyres. The magnitude of the stress reduction is a function of the thickness of the structural layer and its load spreading ability. Different degrees of stress reduction are permissible during construction of the pavement than during its service life. That is, the stresses in the soil can be relatively high during road construction compared with those incurred during its service life because far fewer vehicles load the foundation in the construction phase than are carried by the completed road. It is the purpose of foundation and pavement designs and material specifications to provide a framework within which roads can be economically

constructed on natural soils of various strengths for different estimated amounts of in-service traffic.

Current UK practice for the construction of pavement foundations employs a method specification, which defines the materials to be used for the constituent layers of the foundation, their thicknesses and how they should be compacted. The requirements of a method specification are based on experience of the construction and behaviour of foundations built up over many years. Although these foundations have generally been shown to be adequate, this approach can restrict the contractor's choice of materials and foundation designs to conventional materials with known behaviour laid to accepted thicknesses. Also, the design of flexible and flexible composite pavements is currently independent of the quality of the road foundation. That is, there is no reduction in thickness of the foundation or the pavement when superior materials are used in the foundation. Current specifications and designs thereby inhibit the development of more economic and environmentally friendly solutions. However, increasing emphasis on the use of non-conventional materials, such as recycled materials or industrial by-products, requires the evolution of different approaches that will encourage innovation by the contractor whilst providing the client with improved consistency and assurance that the requirements of the foundation are met.

Nunn et al (1997) reported that in other European countries minimum elastic stiffness values at different levels in the foundation are specified in addition to minimum material density requirements. In Germany, if a minimum stiffness measured at the top of formation level is not attained, then it is the responsibility of the contractor to improve the soil to meet this criterion (RStO 86, 1989). There are also various minimum mandatory stiffness requirements at the top of the foundation that are dependent on the intensity of traffic to be carried by the completed road with reductions in asphalt thickness permitted for higher values of foundation stiffness. For foundations in France, there is both a short and a long-term requirement for the foundation (LCPC & SETRA, 1997). In the short term, the foundation must comply with criteria associated with one of three methods of measuring its elastic behaviour to ensure that it is robust enough to support the construction traffic. The application of the method leads to the foundation being assigned to one of four long-term stiffness classes. For the design of the upper pavement layers, there are various construction options and values of layer thickness from which the design is selected according to the foundation stiffness class and pavement traffic classification.

Within the UK, Chaddock & Brown (1995) described research on the in situ assessment of composite foundation structures that incorporated unbound granular sub-base and, if used, capping on the underlying soil. This research led to outline proposals for an end product specification for road foundations. Fleming & Rogers (1999) described similar work for the underlying earthworks to develop a draft end-performance based specification for capping and subgrade.

This paper reports work carried out on behalf of the Highways Agency as part of their programme of research. In the paper, the primary functional requirements of road foundations are reviewed and further developments of the individual test techniques and procedures to ensure compliance of the road foundations at construction are reported. The main features of a proposed end product specification for standard as well as superior foundations are then outlined. An application of the specification is described. The

limitations of the specification are stated before, finally, its potential benefits are summarised.

2 REQUIREMENTS OF ROAD FOUNDATIONS

The following general requirements for foundations can be distinguished:

- Adequate trafficability during construction to permit the movement of construction vehicles and plant under most circumstances. Exceptions may be during periods of frost, thaw, heavy rain or temporary high groundwater level.
- Good working platform to allow the satisfactory construction of the over-lying pavement.
- Sufficient support to the pavement throughout its design life.

These requirements imply that the foundation should have the following properties:

- Adequate resistance to deformation and, if bound, cracking by construction traffic and, following pavement construction, in-service vehicles.
- Sufficient stiffness to permit good compaction of the overlying pavement and to limit pavement deflection by in-service traffic.
- Adequate durability to adverse environmental factors.

Each constituent layer of the foundation, in turn, is required to have adequate stiffness, resistance to deformation and durability. All of these properties depend to some degree on the compacted density of the constituent materials. To formulate a successful end product specification, it is necessary to develop practical methods that ensure the structural properties are adequate and the specified foundation designs have been built. This requires the monitoring of the thickness of each designated structural layer. Also, direct measurement of the density, durability and, if hydraulically bound, strength of each material layer is necessary. The adequacy of foundation stiffness and resistance to deformation is, however, assessed on the complete foundation rather than on each component separately.

3 EQUIPMENT

In this section a description is given of equipment, both standard and novel, adopted to measure the required material and foundation properties together with an outline of their derived experimental procedures. Investigations to guide the choice of equipment and its design and to develop test procedures are also reported.

3.1 *Material density*

For good performance of the foundation, it is essential that the constituent materials be adequately compacted through the application of sufficient compactive effort by construction equipment. The in situ densities of the foundation materials, expressed as a percentage of their maximum densities achieved in a standard laboratory test, indicate

their state of compaction. This comparison is normally based on dry density for unbound granular materials. Wet density, however, is usually adopted for cement bound granular materials and stabilised soils as frequent “refusal” density measurements are made on wet specimens that are manufactured from as-laid material for strength tests.

In this specification, the recommended test for in situ density is based on the nuclear density gauge (NDG). This device is used in a direct transmission mode in which a hole is formed in the foundation material and a small radioactive source in a probe is extended from the NDG and inserted into the material. The intensity of the radiation reaching the detector positioned in the body of the apparatus on the surface of the test material is related to the bulk density of the material through which it passed. Calibration of the NDG on a sample of known wet bulk density may be required. From the measurement of wet bulk density of the material and knowledge of its moisture content, the dry density of the material can be deduced. Further details of the testing procedure can be found in BS 1377 and BS 1924. The minimum in situ density is required to be at least 95 per cent of the maximum density measured in laboratory tests.

If the method specification for compaction as described in the MCHW 1 is adopted, then the frequency of density testing can be reduced especially when there is evidence, for example from roller location records, that the specification was followed.

3.2 *Stiffness*

Chaddock & Brown (1995) investigated the measurement of foundation stiffness by the falling weight deflectometer (FWD), a prototype lightweight, dynamic plate apparatus and the Loadman. In situ tests were carried out on several pilot-scale foundations constructed of unbound granular material to various thicknesses on clays of different strength. The unbound granular material comprised either Type 1 sub-base on its own or Type 1 sub-base on a gravel capping for the weaker subgrades. Each device was fitted with two sizes of loading plate and a wide range of stresses was applied to the foundations. The results show that different equipment can produce different values for foundation stiffness. Even different experimental arrangements of plate size and applied stress for the same apparatus can lead to different measured values of foundation stiffness. Consequently, standardisation of equipment and test procedures is necessary for a practical end-product specification. One approach is to permit a wide specification for the test, which could lead to a multiplicity of apparatuses and their associated test procedures, and to develop relationships between the tests. However, closer study of the work of Chaddock & Brown (1995) demonstrates that the variation of foundation stiffness with stress has opposing dependencies according to the foundation structure tested and the plate test size adopted. More specifically, foundation stiffness measured by a small plate on a thick unbound granular material increased with stress magnitude, whereas foundation stiffness measured by a large plate on a thin unbound granular material decreased with stress magnitude. Consequently, it could be difficult to develop relationships between different tests unless the comparison is done on a site-specific basis. Another more practical approach, which is currently advocated, is to constrain the test specification with a generic description of one type of apparatus and a defined test procedure.

The prototype lightweight, dynamic plate loading apparatus was developed as a potential low cost apparatus that, during foundation construction, could reliably measure the stiffness of a wide range of foundation types incorporating both unbound and stabilised materials. The apparatus comprises an instrumented loading plate that is placed on the foundation. A mass is then raised and dropped. The mass impacts the loading plate via a rubber damping system and a load cell measures the force applied to the plate. The resulting deflection of the ground is deduced from measurements by a velocity transducer at the centre of the loading plate. The ratio of the applied load to deflection is used to provide a measure of the stiffness of the structure. A layered foundation comprised of materials of different stiffness is therefore represented as a uniform material of a notional stiffness that produces the same deflection under load as the layered foundation. The prototype lightweight, dynamic plate loading apparatus is therefore similar in principle to the full-scale, falling weight deflectometer and, despite restricted capabilities, is more portable. It was developed by Loughborough University under contract to TRL Ltd. Commercial equipment that embodies these principles is produced by both Carl Bro and Dynatest and is known as the PRIMA. TRL assessed an early version of this apparatus that, on their request, was modified to duplicate the prototype apparatus by sensing the deflection of the foundation directly through a hole in the loading plate as opposed to inferring ground deflection from a measurement of the movement of a solid loading plate. This modification is justified by the results shown in Figure 1 of comparative tests on clayey, sandy gravel by FWD and PRIMA.

In this study, the values of foundation stiffness measured by FWD were compared with those values determined by PRIMA, where PRIMA was fitted with either a solid loading plate or, as with FWD, a plate with a central hole. The device with a solid plate measures comparatively low values of stiffness, which indicates that excessively large plate deflections were measured, presumably, due to imperfect plate-soil contact. Consequently, direct measurement of ground movement through a hole in the plate was adopted. This design is especially important for bound foundations or strong unbound materials that do not allow the plate to bed down.

3.3 Deformation resistance

There is no satisfactory in situ, small-scale test method of directly assessing the resistance of a foundation to permanent deformation by traffic. Therefore, Chaddock & Brown (1995) examined whether a relationship could be established between the stiffness of a foundation and its resistance to deformation. They carried out trials on unbound granular foundations and found that foundation stiffness was an approximate indicator of the resistance to deformation of a foundation. For more precise relationships, however, trials were considered necessary for the particular materials and foundation designs adopted at each site. Bound foundations were studied by Nunn et al (1997). They showed that a thin, stiff bound layer would probably remain intact under the low stresses induced in the foundation by the dynamic plate loading apparatus and result in the measurement of an acceptable value of foundation stiffness. However, this layer might suddenly fracture under the higher

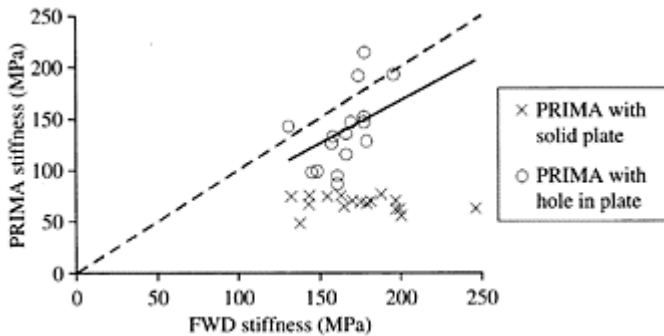


Figure 1. Effect of hole in base plate on foundation stiffness.

stresses induced within it by construction traffic. Its load spreading ability would then reduce dramatically and the foundation would fail by excess permanent deformation. On the basis of the above studies, it was evident that, in the first instance, full-scale trials of the actual materials used and the proposed foundation design would be required for each construction site if the measurement of stiffness is to be used as a means of guarding against excessive permanent deformation of the foundation by construction traffic.

3.4 Strength and durability tests

For bound materials, it is necessary to prove, usually by laboratory tests, that the material in the permanent works is at least as strong as that specified and examined in site trials. For the stronger materials, unconfined compressive strength tests on cubes or cylinders are usually carried out whereas, for weaker materials, CBR tests may be performed. To examine material durability under adverse conditions of moisture, specimens can be soaked prior to strength tests. The susceptibility of a material to freezing conditions can be investigated by measuring its expansion in the frost heave test as described in MCHW, where failure, if it occurs, will be restricted to unbound and weakly bound materials.

4 FOUNDATION TRIALS

4.1 Pilot-scale trials

The work by Chaddock & Brown (1995) involved five full-scale trials and suggested procedures for incorporating test methods into an end product specification. These trials were subject to the natural variability of the subgrade. A pilot-scale trial was therefore carried out by Merrill (2002) to examine standard foundations under more controlled conditions. The trial was constructed within the Pavement Test Facility (PTF) at TRL and, as shown in Figure 2, comprised crushed rock Type 1 sub-base on a gravel capping, a gravel Type 2 sub-base only and a crushed rock Type 1 sub-base only. These three foundations were constructed according to HD25 for a subgrade CBR of 2.75 per cent

and the specification contained in the MCHW in a manner that replicated as closely as possible the methods and practices used in the field. FWD tests shown in Figure 3 were carried out on the foundations immediately after their construction.

The measurements of foundation stiffness are thought to be low with values in the range 25 to 45 MPa, whereas values over 65 MPa were predicted. However, as expected, the 'sub-base on capping' foundation had the highest stiffness and the Type 1 'sub-base only' foundation in Bay A was slightly stiffer than the Type 2 'sub-base only' foundation in Bay B.

During the trial, there were indications of the stiffening of the foundations with time; this effect is illustrated in Figure 4, where test results from the PRIMA at 7 and 44 days after foundation construction are shown. The smallest increase in foundation stiffness is associated with the Type 1 sub-base only foundation in Bay A, whereas the largest increases are in Bays B and C that contain the gravel material on its own or underlying Type 1 sub-base.

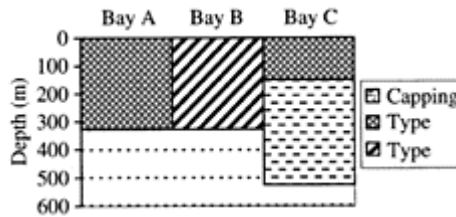


Figure 2. Foundation designs for PTF trials.

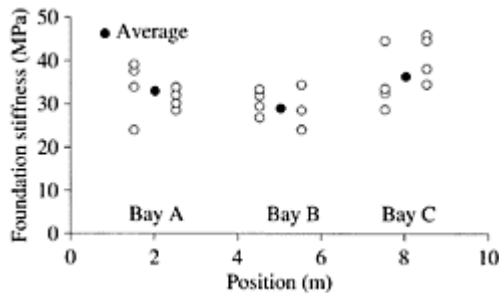


Figure 3. Foundation stiffness.

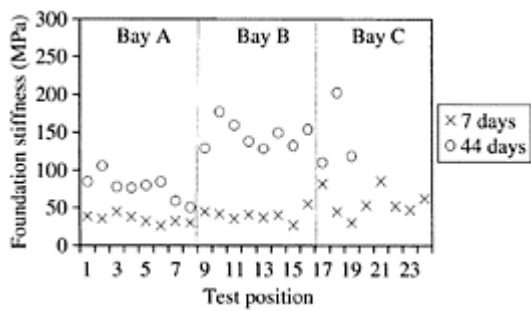


Figure 4. Foundation stiffness measured by PRIMA at different ages.

Table 1. Changes in the trial foundations between construction and trafficking.

Bay	Moisture content shortly after construction (%)	Moisture content after trafficking (%)	Change in foundation stiffness (%)
A	2.3	1.4	127
B	6.0	4.3	258
C	3.5	2.1	223

Stiffening of the foundations was thought to be due to drying out of the uncovered trial areas. The change in the moisture content of the upper part of each foundation was measured by a nuclear gauge placed on the top of the foundation and is recorded in Table 1. All foundations are seen to have partially dried. In particular, foundations B and C, which were initially wetter than foundation A, also lost more moisture than foundation A. It can be deduced by comparing foundations A and B that the moisture changes in the gravel material had a larger effect in changing foundation stiffness than the moisture changes in the Type 1 sub-base.

This drying out effect has implications for the specification, as failure to specify when testing is to be performed could allow the timing of tests to be manipulated to obtain compliance.

The trial foundations were trafficked using the heavy vehicle simulator in the Pavement Test Facility. The machine was fitted with a single wheel loaded to a standard 40 kN. The trafficking was interrupted regularly to permit rutting in the foundations to be monitored.

Figure 5 shows the development of rutting in each foundation for three lines of trafficking. The subbase and capping design (Bay C) developed much less rutting than either of the sub-base only designs. According to Powell et al (1984), a road foundation should be designed so that rutting by construction traffic is no more than 40 mm. Furthermore, a foundation is considered to give adequate

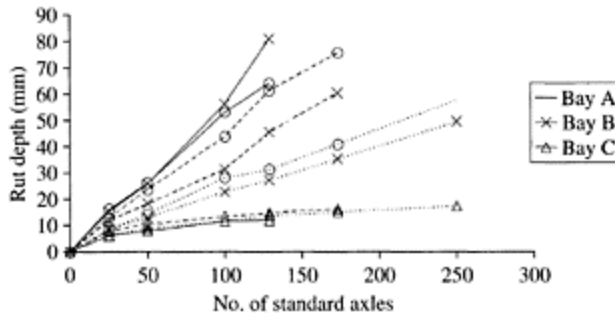


Figure 5. Rut development under trafficking.

performance if it can carry about 1000 standard axles prior to developing this magnitude of rut. Only Bay C satisfied these requirements.

Foundation stiffness was also measured with the PRIMA along the traffic lines just prior to trafficking. The average stiffness of Bay B was highest (149 MPa), closely followed by Bay C (132 MPa) but with Bay A lowest (69 MPa). Foundation B therefore had the highest stiffness yet rutted excessively. Also, Foundation C was the only foundation with acceptable rutting but it was not the stiffest. This observation demonstrates the difficulty of selecting a single, reasonable value of stiffness that would always ensure adequate rutting performance. It is evident that rutting is not controlled by the stiffness of the foundation alone; grading and type of imported material as well as its thickness also effect rutting.

When considering each foundation separately, it was found that the degree of rutting accurately correlated in terms of rank with the measured stiffness. Therefore, if a foundation with a given stiffness in a trial shows good rutting performance, then exceeding that level of stiffness along the site can ensure adequate rutting resistance as long as the foundation materials and their thickness do not change. These observations support the conclusions made by Chaddock & Brown (1995) following the earlier field trials that site-specific target stiffness values need to be established to guard against excessive foundation deformation by construction traffic.

4.2 Full-scale trials

Outline procedures for an end product specification for road foundations were developed by Chaddock & Brown (1995) for unbound granular materials. Subsequently, in TRL's pavement test facility (PTF), Chaddock & Atkinson (1997) and Atkinson & Chaddock (1999) studied trial foundations constructed with hydraulically bound materials and an unbound granular material foundation built as a control. These foundations demonstrated a wide range in performance, where most of the foundations constructed with hydraulically bound materials were shown to potentially provide superior support to the pavement at the time of road construction than traditional unbound granular foundations. More recently, research was carried out for the Highways Agency on foundations built on the road network to evaluate the effects of naturally occurring cracks in hydraulically

bound materials and the environmental effects of moisture and temperature. Results from four trials are shown in Figure 6 where foundation stiffness is plotted against formation stiffness. These values of stiffness were determined from FWD tests conducted on the top of sub-base and on top of the capping or subgrade soil respectively.

The stiffness of each foundation increased with the stiffness of the underlying formation. For each trial, the thickness of the bound material was nominally constant. The curves shown are not average, or 50th percentile relationships, but conservative 10th percentile curves in which 90 per cent of the stiffness measurements plotted higher than the relevant curve with only 10 per cent of readings plotting lower than the curve. Also shown in Figure 6 is the stiffness of standard unbound granular foundation designs of HD 25 plotted against the stiffness of the formation that were calculated

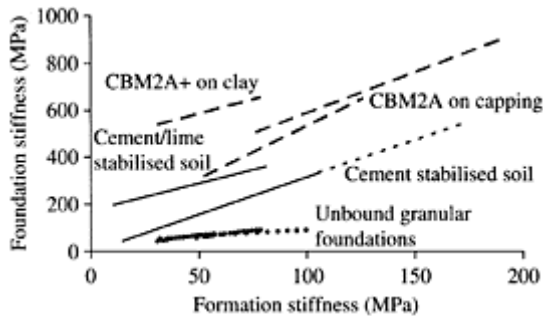


Figure 6. Dependence of foundation stiffness on formation stiffness.

by a multi-layer, linear elastic representation of a road foundation. The calculated values are consistent with the mean values measured on unbound granular foundations by Chaddock & Brown (1995). For a given formation stiffness, the stiffness of foundations constructed with cement bound granular materials (CBM2A) was greater than the stiffness of foundations built with stabilised soil that, in turn, was higher than the stiffness of standard unbound granular foundations. The results of the trials in the PTF and on the road network suggest the establishment of various foundation classes with the unbound granular foundations in the lower classes and the hydraulically bound materials normally assigned to the higher classes. In situ measurements of foundation stiffness could provide assurance of the foundation class assignment at the time of pavement construction. If the superior behaviour of stabilised foundations in the higher classes is maintained, then these benefits could be taken as longer pavement lives for unchanged pavement thickness or thinner pavements for the same pavement life.

5 SPECIFICATION

5.1 Main features

The main elements of the end product specification are as follows:

- A foundation approval trial, or FA trial, is used to set criteria, or target values, for stiffness for a foundation with a proven resistance to deformation by traffic.
- Limiting values of material density and, for bound materials, of strength are also set.
- The permanent works are then tested.
- Any section of the foundation that fails is strengthened and retested.
- Just prior to pavement construction, the foundation is retested if it is thought that the condition of the foundation may have deteriorated since the compliance tests were performed.

The FA trial is a short length of foundation at least 50 m long built on a subgrade representative of the soil type likely to be encountered and would use materials for capping, if required, and subbase as well as construction practices provisionally adopted for the main works. The trial should be constructed in that region where, at the time of the trial, the subgrade is weakest. By measuring foundation stiffness and trafficking the trial, site-specific criteria for foundation stiffness could be derived that would ensure that complying regions of the foundations would not deform excessively under construction traffic. Compliance testing with the dynamic plate loading test would then seek out regions in the permanent works with inadequate stiffness and therefore with increased likelihood of failure under construction traffic. The proposed specification describes the form of the stiffness test apparatus and specifies test procedures. Trafficking the trial is recommended as best practice but it is non-mandatory where documentary proof exists that the planned traffic to construct the foundation will not cause excessive deformation. Setting material density and, for bound

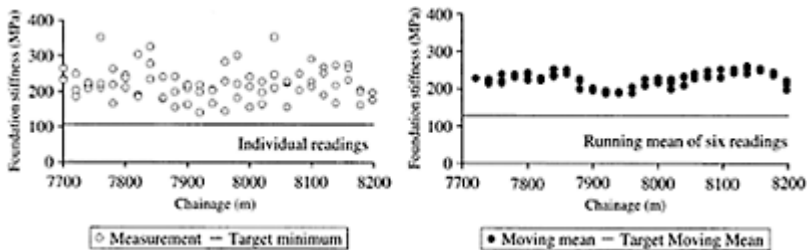


Figure 7. Stiffness of foundation relative to site-specific criteria.

materials, strength ensures that materials constructed in the permanent works are the same, or better, quality than those materials accepted in the FA trial. Compliance testing of the permanent works therefore includes measurements of layer thickness, material density, foundation stiffness and strength of bound materials.

5.2 Application of specification

The test and analysis procedure was applied to FWD measurements of stiffness of an unbound granular foundation comprising Type 1 sub-base on a substantial thickness of chalk fill. In the analysis, the inherent variability in material behaviour was addressed by

setting stiffness criteria for individual readings and running means of six readings from the FA trial. The targets were conservatively chosen and discriminated the lowest 10 per cent of readings. Foundation stiffness measurements in the permanent works are then compared to these criteria in Figure 7.

In the left hand graph, individual foundation stiffness readings taken along the road are compared to a target value for a single measurement; whereas in the right hand graph, a running mean of six readings is compared to the running mean target along the road. This running mean analysis shows more clearly the trend in foundation stiffness along the site and guides the engineer when to make changes to the foundation construction to avoid non-compliance. In this example, the foundation complies with the specification at the time of construction.

The deformation of the FA trial by approximately 1550 standard axles of a loaded lorry was only about 3 mm. This amount of trafficking was therefore predicted to produce 3 mm, or less, of deformation in the permanent works. The foundation had a high stiffness for this foundation type and was highly deformation resistant.

In practice, the proposed specification permits less onerous testing of foundations built with traditional materials to standard designs. Therefore, a foundation, such as that tested above, would probably not be subjected to a trafficking trial. The above results, however, demonstrate the principles of the method. For small works, fixed conservative values of the foundation stiffness criteria would be adopted to avoid the need for a FA trial.

6 DISCUSSION

The end product specification cannot entirely prevent excessive deformation of the foundation by construction traffic due to variability in composition of materials and their response to moisture. Its implementation, however, should improve consistency in foundation performance.

The proposed specification is currently limited in that it assures the client of the satisfactory quality of the foundation just prior to pavement construction. The support the foundation provides the pavement, however, can change with time. Soil subgrades can weaken or strengthen as they wet and dry respectively to equilibrium values. Hydraulically bound capping or sub-base materials can strengthen as they cure, but weaken if they crack in service. Hence, the stiffness of a foundation just prior to pavement construction may differ significantly from its long-term stiffness, which is adopted for pavement design. It is necessary to quantify the effects of these changes for various foundation materials and designs. For good long-term performance of foundations, it is important to ensure, for example, by material mix design and drainage that foundations do not markedly degrade under adverse environmental conditions or with traffic loading.

Eventually, FA trials of specific materials and foundation designs and the derivation of performance criteria may not be required for each new road construction project. Previously documented measurements obtained from road construction projects, or from specially commissioned Type Approval trials, could be used as evidence of the expected performance of the proposed foundation. It would, however, be necessary to demonstrate that the results presented are applicable to the proposed works. This exercise would entail

comparing the structural properties of subgrade, capping (if used) and sub-base materials and foundation design of the proposed works with that reported in the documented evidence.

The application of an end product specification should have the following benefits:

- Assurance for the client of the fitness-for-purpose of the foundation.
- More flexibility in choice of materials and foundation designs for the contractor.
- Introduction of foundation classes and optimisation of pavement design.
- The development and use of more sustainable road construction practices.
- Cost savings.

These benefits are obtained at the cost of investment in equipment, performance of trials, more rigorous testing and data analysis. However, trials to develop materials and construction methods and to demonstrate the fitness-for-purpose of foundations are becoming commonplace on jobs of the scale likely to benefit from this investment.

7 CONCLUSIONS

An end-product specification based on in situ tests and site-specific trials could give the contractor more freedom in the choice of materials and designs whilst providing assurance to the client of the adequacy of the foundation.

Foundations could be assigned to various classes according to the quality of the support to the overlying pavement. Foundations incorporating hydraulically bound materials would fall into the upper classes and those built with unbound granular materials in the lower classes.

The thickness of pavements built on superior foundations could be reduced.

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REFERENCES

- Atkinson, V.M., Chaddock, B.C.J. & Dawson, A.R. 1999. *Enabling the use of secondary aggregates and binders in pavement foundations*, TRL Report 408. Crowthorne: Transport Research Laboratory.
- BS 1377. 1990. *Soils for civil engineering purposes—Part 9, In situ tests*. London: British Standards Institution.
- BS 1924. 1990. *Stabilized materials for civil engineering purposes – Part 2, Methods of test for cement-stabilized and lime-stabilized materials*. London: British Standards Institution.

- Chaddock, B.C.J. & Atkinson, V.M. 1997. *Stabilised sub-bases in road foundations; structural assessment and benefits*, TRL Report 248. Crowthorne: Transport Research Laboratory.
- Chaddock, B.C.J. & Brown, A.J. 1995. In situ tests for road foundation assessment. In R.Jones and A.Dawson (eds.), *Unbound Aggregates in Roads (UNBAR 4); Proc. of Symp., Nottingham, 17–19 July 1995*. Rotterdam: Balkema.
- Fleming, P.R. & Rogers, C.D.F. 1999. *A performance based specification for subgrade and capping*, Contract Report No. 1. Nottingham: Scott Wilson Pavement Engineering.
- HD 25, Pavement Design and Maintenance-Foundations. *Design Manual for Roads and Bridges* (7.2.2.). Norwich, TSO.
- LCPC & SETRA. (Laboratoire Central des Ponts et Chaussées and Service d'Études Techniques des Routes et Autoroutes). 1997. *French design manual for pavement structures*. Paris: LCPC.
- MCHW 1. Specification for Highway Works. *Manual of Contract Documents for Highway Works*, Norwich, TSO.
- Merrill, D.B. 2002. *End performance specification for foundations*, Unpublished TRL Report PR/IP/02 0/02. Crowthorne: Transport Research Laboratory.
- Nunn, M.E., Brown, A.J., Weston, D. & Nicholls, J.C. 1997. *Design of long-life flexible pavements for heavy traffic*, TRL Report 250. Crowthorne: Transport Research Laboratory.
- Powell, W.D., Potter, J.F., Mayhew, H.C. & Nunn, M.E., 1984. *The structural design of bituminous roads*, TRRL Report LR1132. Crowthorne: Transport Research Laboratory.
- RstO 86, 1989. *Guidelines for the standardisation of the upper structure of traffic-bearing surf faces*, Bonn: Federal Minister of Transport.

Author Index

Pavements Unbound—Dawson (ed.)

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Alabaster D. 89
Allou F. 61
Arnold G. 89, 169

Baldo N. 51
Black P. 329
Breyse D. 179
Brown M. 149

Caicedo B. 43
Ceratti J.A. 23
Ceylan H. 139
Chaddock B.C.J. 355
Chazallon C. 61, 179, 191
Chik Z. 43
Cygas D. 219

Dawson A. 169
de Carvalho J.C. 249
de Rezende L.R. 249
Denis A. 179
Douglas R.A. 157
Dudgeon R. 335

Edwards J.P. 3
Ek Dahl P. 107
El abd A. 179
Endo K. 69

Flemming P.R. 3

Gehling W.Y.Y. 23
Ghazireh N. 259, 267
Gomes Correia A. 97
Gomez-Ramirez F. 139

Guclu A. 139

Habiballah T. 191

Hakim B. 133

Hameury O. 291

Hansson J. 107

Hoffman M.S. 277

Hornych P. 179, 191, 291

Hughes D. 169

Huvstig A. 107

Janoo V 115

Jones J.D. 301

Jones M. 133

Juzenas A.A. 219

Kamiura M. 125

Kergoët M. 291

Kinuthia J.M. 311

Kolisoja P 13

Koyanagawa M. 69

Kroesen B. 319

Légère G. 345

Lobo-guerrero S. 33

Maki T. 69

Malysz R. 23

McGill C. 239

Mercier S. 149, 345

Merrill D.B. 355

Mitchell S.A. 157

Moeller B. 209

Nakayaka S. 125

Neves J.M.C. 97

Nidzam R.M. 311

Nishizawa T. 69

Núñez W.P. 23

Oeser M. 209

Pasetto M. 51

Pidwerbesky B.D. 157

Provencher Y. 149

Puiatti D. 291

Quigley P. 239

Rahimzadeh B. 133
Ramanuj am J.M. 301
Ravindra P.S. 79
Robinson D. 169
Robinson H.L. 267
Robinson R.B. 311
Rockliff D. 335
Ryan T. 239

Saarenketo T. 13
Shoop S. 115
Small J.C. 79
Steven B. 89

Takeuchi Y. 69
Theyse H.L. 199, 229
Thom N. 133
Thom N.H. 3, 259
Thompson M.R. 139
Thorén H. 107
Tucek S. 43
Tutumluer E. 139

Vallejo L.E. 33, 43
van Gurp C. 319
Vuorimies N. 13

Wellner F. 209
Werkmeister S. 209
Wild S. 311
Wood O. 259

Zilioniene D. 219

Subject Index

Pavements Unbound—Dawson (ed.)

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This index gives the page numbers of the start of papers that contain relevant information. The page numbers given are not necessarily those that contain that information.

Aggregate Supply 43, 329

Back Analysis 89, 97, 125, 157, 277

Climatic and Environmental Condition 13, 43, 115

Design of Pavements 169, 179, 191, 199, 209, 219, 229, 239

Evaluation of Granular Layers and Pavements 89, 97, 139, 125, 133, 157, 277

Granular Material Behaviour 23, 33, 43, 51, 61, 69, 79, 169, 191

Life 89

Management and Maintenance 149, 157, 219

Peat 239

Performance in the Pavement 97, 107, 115, 125, 133, 249, 277, 345

Recycled and Secondary Aggregates 249, 259, 267, 277

Specification 133, 335, 345, 355

Stabilisation 291, 301, 311, 319, 259

Subgrade 319

Testing, In-situ 335, 345, 355, 133

Testing, Laboratory 3, 23

+