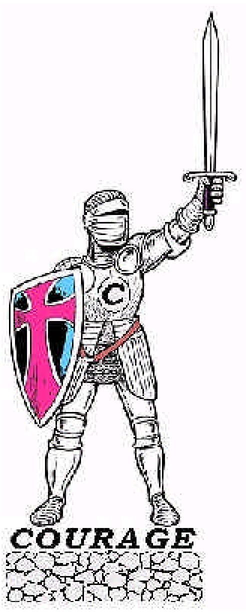
European Commission – DGVII
4th Framework Programme (1994 – 1998)
ROAD TRANSPORT RESEARCH

COURAGE

COnstruction with **U**nbound **R**oad **Ag**gregates
in **E**urope



FINAL REPORT

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EXECUTIVE SUMMARY

Background

This document reports on a EU-supported RTD project on COnstruction with Unbound Road Aggregates in Europe (*COURAGE*). Unbound granular materials (UGMs), as used in pavement layers excluding the surfacing layer, are formed of compacted aggregate from geologic or, sometimes, industrial sources. In many European countries only the top 80-100 mm of a road pavement are constructed of bound asphaltic concrete material. The remainder, which varies from 300-1500mm in thickness, is constructed from UGM. This equates to between 6000 and 30 000 tonnes of aggregate per kilometre of single carriageway pavement. The great majority of this aggregate is sourced from gravel pits or is obtained from rock quarries. In Europe, the total aggregate consumption from quarries and gravel pits into unbound pavement layers is of the order of 750 million tonnes per annum.

Importance of the Study

It has recently been estimated that 12% of the GDP of the USA is spent on highway-related work (construction, maintenance, journey costs, etc.). In the USA, the pavement construction and maintenance budget is approximately equal to that spent annually on aircraft. European expenditure is not expected to vary greatly from this. Once wages, equipment and profit elements are allowed for, around 40% of costs are incurred on the provision of materials used to build and maintain the road network. Despite this huge financial commitment, only some 0.2 to 0.3% of total spending is directed towards research. This may be compared with around 5% for electronics, or double figures for pharmaceuticals. In such an under-researched area, almost any study has the potential to deliver large efficiency and/or environmental savings. The COURAGE project seeks to do this by providing a basis for optimising the use of UGMs in road construction. At the end of this Executive Summary an estimate is given of the potential savings which could result if the findings of this study were widely implemented throughout Europe. The sum is very large.

Aim

The overall aim of the COURAGE project has been to improve the use of UGMs leading to more efficient use and to characterise them more reliably so that aggregates, which are presently discarded, may be exploited sensibly. At present, the reliable performance of the UGM layers of the pavement is ensured by the addition of an extra thickness of aggregate and by the refusal to use materials for which behaviour has not been ascertained by performance testing, experience or field trials. Both practices are inefficient and lead to unnecessary wastage. An alternative strategy is to down-grade the UGMs and to use thicker bituminous surfacings. This increases reliability, but only by incurring greatly increased costs. Thus, a reliable framework has been sought for the assessment of any potential UGMs (e.g. industrial residues and by-products such as ash, slag, demolition waste, etc.). If exploited in a reliable way, this could reduce the need to quarry for rock, the need to dump possible alternative materials as wastes and the need for excessive expenditure on asphaltic material surfacings. This overall aim has been met:

- by assessing UGMs in a variety of ways to determine those test procedures which can deliver useful characterisations,
- by determining the variability of in-situ pavement conditions which can have a large affect on actual UGM performance,
- by providing an analytical framework for describing performance, which can be used as a basis for more reliable design computations.

Programme

A programme of testing both in the laboratory and on real and experimental pavement sections was therefore developed. Four different types of aggregates were thoroughly examined, all of which are of types available widely throughout Europe. One of the UGMs chosen was a demolition waste, the other three were derived from marginal and good quality crushed rock sources. The field testing work comprised assessments of the pavement response to both traffic loading and to climatic effects. The laboratory work used both simple test procedures, as currently employed throughout Europe, as well as more advanced performance-related test procedures designed to more reliably characterise likely in-situ performance.

Partners

A consortium of nine centres has carried out the work. The co-ordinator has been the University of Nottingham (UK) with the main partners being the Laboratoire Central des Ponts et Chaussées (France), Instituto Superior Téchnico (Portugal), the Technical Research Centre of Finland and the National Roads Authority (Ireland). Associated with them have been the Finnish Roads Administration, the Public Roads Administration (Iceland), ZAG (Slovenia) and TEI-Athens (Greece). Each of the main partners was responsible for leading one of the five work packages.

Observations

The following key observations have been made from the studies undertaken.

- The in-situ monitoring of UGM condition in pavements has revealed that the moisture content of the UGM varies considerably with the season. In base layers (those immediately beneath the bound surfacing) the variation is between 40 and 90% of optimum moisture content. For the lower sub-bases and even greater variation between 30 and >100% of optimum moisture content was measured. The structural contribution to the pavement of UGM with varying moisture was investigated in-situ and in the laboratory (see below).
- The in-situ monitoring also revealed that the moisture in the pavement structure is very dependent on the:
  - precipitation levels,
  - integrity of the sealed surface,
  - final preparation applied to the shoulders of the pavement (sealed or unsealed and seal width, partial or full),
  - level of the pavement (raised pavement or pavement in cutting),
  - ability of the pavement to self drain (the UGM's permeability and the adequacy of the pavement's drainage system).
- Pavement performance, as measured by deflection in-situ, shows a very serious degradation as the moisture content of the UGM rises.
- Some of the pavements constructed during the project were assessed as having an excellent performance, thus demonstrating the viability of building pavements in this manner, with thin surfacings.
- Empirical laboratory tests are widely used in Europe at present. They *can* aid material assessment but, due to their simplistic nature, often fail to clearly differentiate between material qualities. The repeated load triaxial test (RLT) was found in this project to give much more reliable indications and the draft CEN standard for the RLT has been shown to offer a significant advance. However, on the evidence of this study, the procedure needs improving in particular ways (particularly so that aggregates for sub-base layers are not over-specified). The project team is making representations to the relevant CEN committee to incorporate these improvements into the revised standard. When adopted, the revised standard will assist in the aim of maximising use of marginal materials in pavement construction.
- The RLT was employed to allow the effects of changing moisture content to be assessed. It was found that significant reductions in the stiffness of the UGMs (which in the pavement largely dictates its ability to spread traffic loads safely onto the underlying soil without over-stressing it) were observed as the moisture content increased. In addition, significant increases in

permanent deformation (experienced in the pavement as susceptibility to rutting) were observed as the moisture content increased.

- ♦ The rutting behaviour as measured in simulative wheel tracking testing (performed at the University of Oulu, Finland), confirmed the applicability of the RLT to characterise the permanent deformation propensity of UGMs. It was also found that the LCPC constitutive relationship and the TSA strain rate parameter were reliable tools for characterising the observed permanent deformation developed in the RLT test with numbers of load applications.
- ♦ Design procedures which adequately incorporate the behaviour of the UGM layers have been developed and trialled in this project. These have shown that:
  - the moisture content must be carefully matched for thinly surfaced pavements, given the significance of the moisture content on the UGM and pavement performance,
  - linear elastic computations are acceptable for thickly surfaced pavements and for simplistic representations of the pavement structure,
  - the Boyce constitutive model of resilient stress-strain behaviour provided an appropriate non-linear representation of the UGM for use in calculating the response to traffic loading for a pavement having an intermediate thickness of bituminous surfacing.
- ♦ Analytical computations employing the materials data from the RLT showed that, at design stage, more thinly surfaced pavements require a non-linear approach to stiffness characterisation of the UGM. As a result, the more advanced approach described by the COURAGE project is necessary to fully exploit the benefits of the UGM in a reliable manner. However, for more thickly surfaced pavements, where a linear design approach is workable, a simplified RLT regime will be possible.

Recommendations

Overall, the COURAGE study recommends that more effort should be given to determining the relevant mechanical properties of the compacted aggregate mixture (the UGM), rather than to determining the properties of material indices (which are often performed only on some of the large particles taken loose from the mixture) which is currently the common European practice. To achieve a reliable characterisation it is recommended that the RLT test is employed, following the draft CEN standard but with modifications. In addition, the testing programme must assess UGMs at the likely in-situ moisture contents which they will have during the life of the pavement. The moisture content variation may be expected to be large and further studies may be necessary to ensure that a reasonable estimate can be obtained. Some seasonal exhumation of current pavements in similar situations may be helpful. Design of the pavement should then utilise an adequate representation of the pavement structure and the UGM. This will necessitate the use of a non-linear approach for the UGM drawing on the data obtained in the RLT. RLT data can also be used to check for rutting susceptibility although, at present, direct computation of rutting development with trafficking is not possible.

Significance of Findings

A cost benefit analysis has been performed to assess the utility of the findings of the COURAGE project. In the study, four thinly surfaced pavement constructions, such as might be used in different European countries, were selected. In one of these, a more marginal UGM was employed as might be recommended following the use of the testing procedures outlined in the COURAGE report, together with the analytical design techniques described in it. This analysis demonstrated that the thinly surfaced, aggregate rich pavement offers significant cost advantages for moderate traffic levels, given a thirty year assumed maintenance period before complete reconstruction is required. For a single carriageway pavement a saving over current practice of between 50 000 and 150 000 Euros/kilometre is believed to be achievable over the thirty year period. Given the magnitude of the consumption of aggregates into road pavements, which was indicated at the beginning of this executive summary, European pavement construction could be saving of the order of 3.5 billion Euros per annum, taking reasonable assumptions about usage in construction and reconstruction. These savings do not include the additional benefits which would result from a reduction in environmental impact if industrial residues are used so that conventional aggregate quarrying and dumping of wastes are both reduced.

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1. INTRODUCTION

1.1 Overview

Roads are an essential part of everyday life since they provide a platform for freight haulage, in addition to satisfying the diverse range of business and recreational commuting needs. The design of pavement structures must take into account the many uncertainties which exist. Some of these include predicting traffic volumes (present and future and their distributions, day and night), varying vehicle axle mass configurations, constituent material properties and seasonal variations (which affect in-situ conditions such as moisture content, freeze/thaw periods and drainage requirements). The design of roads has not been one of engineering's "exact sciences", having been more an "art", principally based on a designer's experience and knowledge. The **COURAGE** project (which stands for COnstruction with Unbound Road AGgregates in Europe) was established to improve the understanding of unbound granular materials through targeted laboratory and field based testing programs in order that designers should be better, and more reliably, equipped. A principal means to achieving this is by developing a more scientific understanding of the materials by which some of the inefficiencies of empiricism could be addressed.

The COURAGE Project is concerned with investigating the fundamental characteristics and mechanical behaviour of unbound granular materials that are used in pavement construction. In order to characterise the behaviour of granular materials, COURAGE draws on functional and simplified laboratory tests. This work is closely aligned to experimental field studies aimed at monitoring seasonal moisture variations and structural stiffness, through non-destructive testing. In addition, two full-scale pavement test sections were established and analysed to provide a link between the laboratory and field data.

Work undertaken within the scope of COURAGE provides an improved practical means of assessment to enable the performance of such materials to be more rigorously defined. This in turn will assist in maximising the efficient use of unbound granular materials in road construction, improve consumption of currently wasted materials and provide increased reliability of pavement performance.

The Project, which is funded by the European Commission under the DG VII Fourth RTD framework programme, comprises a research team selected from highway testing laboratories, university researchers and road pavement owners from all parts of Europe. The proposal is supported by FEHRL - the Forum of European National Highway Research Laboratories - and is closely linked to COST Action 337 on "Unbound Granular Materials", from whom more than half of the team is drawn. The list of Partners and Associate Partners for the Project is as follows:

Co-ordinator	University of Nottingham – United Kingdom
Partners:	IST – Portugal
	VTT - Finland
	LCPC – France
	NRA – Ireland
Associate Partners:	ZAG – Slovenia
	Finnra - Finland
	PRA - Iceland
	University of Hannover – Germany
	TEI-Athens – Greece

The distribution of work between the Partners is given in Annex 1.

1.2 COURAGE Objectives

The objectives for COURAGE were:

- to produce a fundamental and universal mechanical behaviour framework for unbound granular materials drawing on functional and simplified laboratory tests
- to produce new, rapid and practical assessment approaches based on this framework
- to deliver a fundamentally acceptable classification system for granular materials
- to collect measurements of real, in-situ, mechanical performance thereby assessing effects of site and climatological variability
- to produce models of the appropriate mechanical behaviour characteristics of unbound granular materials based on both the laboratory and field assessments
- to propose better means of use of unbound granular materials using the improved understanding and experience resulting from the previous objectives
- to draft guidelines to testing, modelling and incorporation of unbound granular materials (conventional and secondary) into structural and foundation pavement layers.

1.3 Project Justification

More than half of the countries in Europe use an analytical approach to designing their road pavements. In doing so, at least 30% of these countries use performance-based road material information as an input into their design process. These percentages can be expected to increase as reliable performance data for the unbound granular materials is obtained.

The need to have reliable information on the mechanical performance properties of the constituent pavement materials, in addition to climatic and environmental information, is essential to the analytical design process. A poor understanding of these properties can lead to the uneconomical over-design of pavements, with a subsequent wastage of valuable material resources. Alternatively, it can result in the under-design of pavements which carry a high risk of failure, with the resulting expense of maintenance, inconvenience and cost to the road users.

In general, the behaviour of unbound granular materials is poorly understood and little research has been performed in Europe to date, especially when compared to the other pavement materials - asphalt and concrete. Therefore, this project has aimed to support a real advance in structural pavement design and performance by investigating this material in some depth.

2. BACKGROUND

Pavements commonly consist of a bound layer (often asphalt) over on or more granular layers (base and sub-base) which, together, are compacted over the soil which exists at the site of interest.

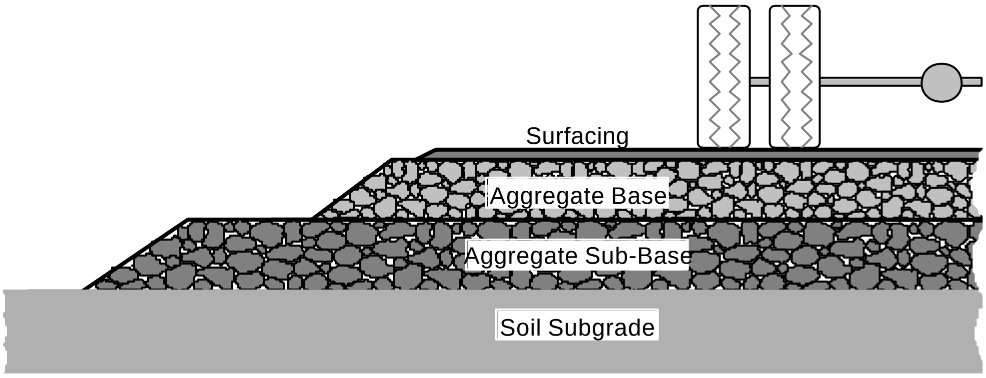

FIGURE 2.1: SCHEMATIC VIEW OF A GRANULAR PAVEMENT

The COURAGE project is concerned with the materials which may be used in the base and sub-base layers. Sometimes aggregates are used as a partial subgrade improvement, known as capping or fill. In many European countries only the top 80-100 mm of a road pavement are constructed of bound asphaltic concrete material. The remainder, which varies from 300-1500mm in thickness, is constructed from UGM. This equates to between 6000 and 30 000 tonnes of aggregate per kilometre of single carriageway pavement. The great majority of this aggregate is sourced from gravel pits or is obtained from rock quarries. In Europe, the total aggregate consumption from quarries and gravel pits into unbound pavement layers is of the order of 750 million tonnes per annum.

2.1 Importance of Sustainable Road Building Materials

The efficient management of mineral resources used for road building materials is vital for a sustainable future to all countries around the world, particularly given the current levels of aggregate consumption mentioned. Some countries have very limited resources, either in quantity or quality, and so it is imperative that materials are put to good practical use, with material wastage kept to a minimum. In addition, it is possible through correct life cycle management to extend the use of unbound granular materials by in-situ stabilisation with small quantities of stabilising agents, such as cement or bitumen.

To further support the preservation of high quality geological aggregate sources and minimise the impacts of extractive processes, more use is being made of alternative sources of road building materials. Some alternative sources include:

- recycled crushed concrete and asphalt planings (with masonry blended in small proportions)
- blast furnace slag
- steel slag
- municipal solid waste incinerator ash
- blended material (primary and secondary aggregates)

However, little is known about the performance of such materials both mechanically and chemically. As a result, such materials are often termed 'secondary aggregates'. In addition, given the broad range of in-situ conditions to which conventional or secondary materials may be subjected in the pavement structure (such as climate, compaction conditions, traffic-imposed stress regimes, etc), what is deemed a suitable use for each of these very different materials? Some materials may be suitable for sub-base or capping/fill applications and some possibly for basecourse applications.

The European Commission has supported two such projects under the 4th Framework Programme, which strongly target the improved understanding of conventional and alternative material sources. The Alternative-Materials (or **ALT-MAT**) project has been established to assess the range of materials mentioned above for their chemical and environmental effects when used in road construction. Testing in this area is investigating leaching potential and hydraulic influences on the materials. In addition, this project will investigate material performance using some basic mechanical tests. The second research activity supported by the EC, which is the basis of this report, is the Construction with Unbound Road Aggregates in Europe (or **COURAGE**) project. Here some primary and secondary aggregate sources are investigated, along with a recycled concrete and asphalt planings (alternative) material. The materials are subjected to a wide range of mechanical tests to look at the performance of such products by simulating in-situ pavement conditions.

Recycling industries/plants are rapidly being established in a number of European countries, with large volumes of material being supplied to road construction consumers, particularly local road authorities. Some companies are developing a national network of strategically located operations to handle a range of incoming materials, which are being recycled into construction aggregates. A number of countries such as Germany, Sweden, Denmark, the Netherlands and France have been identified as nations which have proactive policies and programs to promote the reuse of recycled materials in the highway environment. Given the difficulties of progressing this area, Government support and direction is strong in this area, and is expected to further strengthen in coming years. Thus, it is *vitally* important that funds are available for strategic research into the performance of such materials in road pavements.

2.2 Empirical vs Performance-Based Design Data

In most pavement construction projects the use of aggregates is largely determined by the design of the pavement. The design specifies the quality of the materials to be used and the layer thicknesses which has an impact on quantity. The under-design of pavement layers or poor specification of pavement materials can lead to premature road failures. Conversely, over-design of individual pavement layer thicknesses will lead, in effect, to material wastage. In addition, over-specification of aggregate quality will lead to the wastage of high-grade materials which should be reserved for more demanding road requirements.

A number of techniques have been used over the years for designing pavements. This information has been well summarised in the EC funded **COST 333** and Advanced Models for Analytical Design of European Pavement Structures (or **AMADEUS**) Projects. These projects undertook a comprehensive survey of many European countries to determine the approaches used to design pavements and the sources from which material design data is derived. In addition, a simple design example was presented to each participating country's road agency to investigate the variability in design expected using these wide range of techniques and materials data sources.

The 'technique' or method used for pavement design over the years has generally been one of the following approaches[1]:

- ***experienced-based*** (or prescription or catalogue) ***methods*** which provide a catalogue of standard designs and materials based on past successful practice. This approach is often best applied when the effects of variation in local materials, layer thicknesses, traffic

[1] Country-by country review by COST 333 project.

loadings/volumes and climate is well understood. Pavement failures have often resulted, however, when this approach, established in one locality, has been transferred and applied in another which may have totally different conditions (eg, subgrade soils or climate). Alternatively, it may lead to the design of very conservative pavements, which are uneconomical.

- ***empirical methods*** where design is calculated from measured or estimated material properties, for example, the CBR or deflection, and is related to past successful experience.
- ***mechanistic or analytical or rational methods*** where the response, and subsequent performance, of different pavement types to traffic loadings depends solely on the measured performance properties of the material components which comprise the pavement. This method of analysis relies on the designer selecting a 'trial pavement' cross-section with consideration given to the number and thicknesses of each layer, the material types, material state conditions, material performance properties from laboratory or field tests and estimating load levels throughout the design life. The designer then analyses the stresses and strains caused by the applied traffic loadings on this 'trial pavement' (using a response model) and then determines the 'critical life' for selected layers, based on established performance prediction models. If necessary, the pavement design is then revised and a further iteration of the cycle is conducted.
- ***semi-emperical method*** uses a combination of empirical and mechanistic methods.

It should be remembered that aggregates have to meet specifications to achieve their required performance. At present few specifications world-wide for unbound granular materials are truly performance-based, rather being empirically-based, despite the fact that most countries employ some form of mechanistic analysis in their pavement design procedures. Given that pavement material performance data derived from laboratory and/or field testing is critical to the mechanistic design processes mentioned above, it is essential that these properties are measured. As a result, it is evident that strong links exist between the materials testing research performed in COURAGE and the design process investigated by AMADEUS.

2.3 Overview of Mechanistic Pavement Design Requirements

In order to utilise a mechanistic pavement design technique, which requires the development of a pavement 'response' model for analysing the structure, certain information is required pertaining to the constituent material properties. Basic linear-elastic modelling programs simply require knowledge of elastic resilient modulus[2] (or stiffness) and Poisson's ratio of the constituent pavement and subgrade materials for their given state conditions (of density and moisture content). The COURAGE project deals with the provision of these material properties, in addition to investigating climate conditions.

[2] In this report the phase resilient modulus is used to describe the ratio of stress change to unloading strain

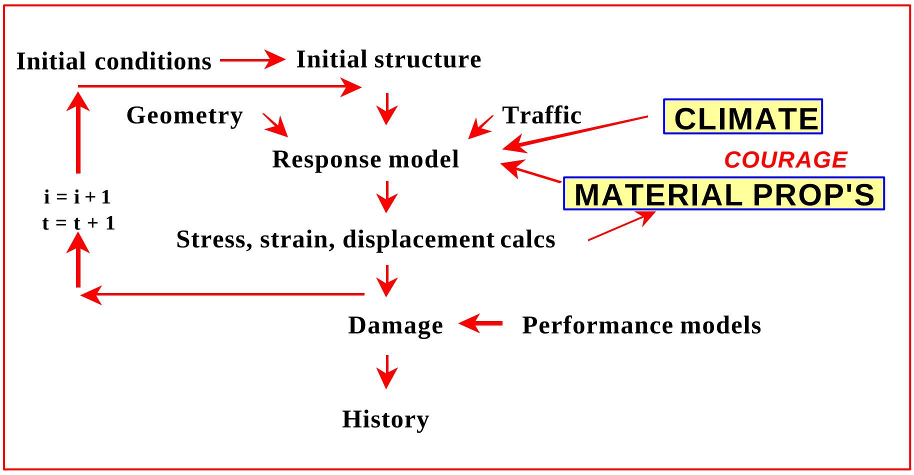

FIGURE 2.2: FLOW CHART OF ITERATIVE PAVEMENT DESIGN PROCEDURE

Given that resilient modulus in particular is stress-dependent, each sub-layer of material within the pavement can be characterised by a "unique resilient modulus" based upon the average traffic-imposed stress conditions to which it is subjected. These pavement design response models can be used in a manual-iterative sense (see Figure 2.2) until the agreement between resilient modulus used and the resilient modulus from calculated stress is satisfied to within certain tolerances (Mundy, September 1992). Introducing a number of sub-layers into the response model has the effect of accounting for this material non-linearity.

More advanced non-linear response models, as constructed using finite element code, generally rely upon more complex models (and model parameters) which are used to predict modulus and Poisson's ratio. These models need to model well the resilient axial and radial strains as measured in laboratory testing systems in order to account for non-linearity and material behaviour under varying stress and state conditions. Such material models do exist, such as the Boyce model, but their parameters *must* be determined using quality material data, which is derived by testing at a range of expected in-situ conditions and takes into account equipment strain measuring limitations.

Thus, three challenges exist:

i) to know the conditions in-situ so that laboratory testing may be performed using the same conditions
ii) to measure material response in the laboratory well
iii) to have good quality analytical tools so that in-situ performance can be reliably predicted from the laboratory data

The COURAGE project seeks to address all three elements.

Once a harmonised set of resilient modulus values and stresses have been developed, the stresses and strains in the pavement can be calculated and compared to those which are known to be permissible for acceptable pavement performance. Generally, there will be different stress or strain criteria to limit fatigue in bound layers, rutting of different granular layers, localise shear failure, etc.

3. SEASONAL VARIABILITY EFFECTS ON GRANULAR MATERIALS WITHIN CONSTRUCTED PAVEMENTS

3.1 Introduction

Unbound granular materials in road pavements are known to undergo changes in density and moisture content from those determined at the end of the construction period. Initial trafficking causes further densification of the granular layers which slightly improves pavement stiffness, but the parameter which causes greatest variation in the strength of granular pavements is the moisture content. Knowledge of the variation of moisture content during the life of a pavement is fundamental to the selection of appropriate strength parameters for granular pavements, which are needed for accurate prediction of pavement performance leading to more economical pavements. Variation in moisture content in granular pavements can be effected by the severity of the weather conditions, the road cross-section, the type and thickness of bituminous surfacing over the granular layers, the permeability of the granular materials and the depth to the water table.

The recent availability of a new moisture-measuring device (TDR, Time Domain Reflectometer) has enabled researchers to more accurately monitor the moisture in granular pavements. This and other equipment used is described in Section 3.3. Because of the limited time scale and funding, it was impossible to construct new specially dedicated test roads for the project so all the monitoring was carried out on existing test road sections but in some instances monitoring equipment was installed in new roads which were undergoing construction. A survey of available test roads was carried out and those roads useful to the project were selected. Moisture content results from previously reported test sites in the project area were evaluated and are included where they can be compared with data from the test sites. The test sites were selected to include a variety of environmental conditions, construction materials and traffic levels.

3.2 Site Descriptions

Details of the test sites, which were located in five countries, are reported in Table 3.1. Winter climatic conditions are generally severe in Iceland, Finland and Slovenia where frost penetration is normal, while such conditions are rare in Ireland and Portugal.

In Iceland, the thickness and quality of the base course layers were deliberately reduced to encourage early failure of the test sections. On most sites, the shoulder was less than 1m wide and unsealed, but in Ireland the shoulder was 3m wide and had the same bituminous surfacing as the carriageway. At Site FI.2 in Finland, the 1.5m wide shoulders were paved for a width of 1.25m.

Country	Site Identification		No. of test Sections	Sensor Type	Pavement Material	Surfacing		Traffic	Site Details
	Local	Courage				Type	Depth (mm)	AADT	
Iceland	A 1.4.1 1.4.2	IS.1 & 2	2	TDR/A	Gravel	S.D.	46, 37	445	Raised pavement
	B 3.2.3.1	IS.3	1	TDR/A	Gravel	S.D.	36	370	Raised pavement
Finland	Mt 1531	FI.1	1	TDR/M	Gravel	A.C.	50	1,400	Raised pavement
	Raahe Vt.8	FI.2	2	TDR/A	Crushed Rock	A.C.	80	6,500	Raised pavement and pavement in cutting
Ireland	N4	EI.1&2	2	RES/M	Crushed Rock	H.R.A.	100	3,600	Pavement in cutting
	N7	EI.3	1	RES/M	Gravel	A.C.	100	9,000+	Pavement in cutting
Slovenia	A2	SI.1	1	TDR/A	Stab. Base Silty clayey gravel	A.C.	130	13,000	Pavement with sensors in shallow cutting
Portugal	IC.1	PT.1	1	TDR/A	Crushed Rock	Mainly A.C.	295		Pavement in cutting

TDR/A = TDR Automatic
TDR/M = TDR Manual
RES/M = Resistance type, manual

S.D. = Surface Dressing
A.C. = Asphaltic Concrete
H.R.A. = Hot Rolled Asphalt

TABLE 3.1: DETAILS OF TEST SITES USED TO MONITOR SEASONAL VARIABILITY EFFECTS

3.3 Measuring Equipment

The measuring equipment used in all but the Irish sites were Time Domain Reflectometer (TDR) probes together with temperature sensors in the form of thermocouples. The moisture probes are based on the principle of Time Domain Reflectometry, a non-destructive electromagnetic technology, which utilises the relationship between the dielectric constant of porous materials and their moisture content. The moisture probes have two or three parallel metal conductors of known length. A transmitter inside the probes generates an electromagnetic pulse of frequencies that propagate along the metal conductors of the probes and reflect back to the source at the end of the conductors. As the pulse travels along the conductors an electromagnetic field is established in the soil material between and around the conductors. The transit time of the pulse is measured and it provides an estimation of the dielectric constant of the soil material. The dielectric constant (81 in water, 1 in air, and approximately 5 in dry soil) is directly proportional with the volumetric amount of water in the influenced soil material and the relationship has been estimated empirically using the oven drying method. The volumetric moisture content can thereafter be converted to the gravimetric moisture content if the in-situ dry density is known. Results from TDR measurements are dependent on the density of the soil and their registrations must therefore be calibrated. The calibration process varies from one producer to another. The basic information needed is the actual value of the dry density of the soil material based on in-situ density determinations and/or compaction tests.

The TDR probes used on sites in Iceland, Finland and Slovenia, are referred to as “intelligent” probes, because all the electronics necessary for their operation are contained within the probe, which transmits the readings in digital form to a data-logger. With such equipment the data-logger can be located at any distance from the probe as the recorded measurements are independent of the cable length to the data-logger. The TDR probe used on Site PT.1 in Portugal, however, did not contain the electronics within the instrument, and the readings were transmitted through long cables to the measuring unit, which apparently influenced the test results.

As the TDR method measures the free (unfrozen) water in the volume of surrounding material it cannot accurately record the moisture content of frozen material. Hence values recorded during periods of significant frost penetration in Iceland and Finland were noticeably lower than those recorded in more temperate conditions and cannot be considered as a true indication of moisture content in such conditions.

The measuring equipment and data-logger used with the TDR probes can be automatic, as was the case in the Icelandic and Slovenian sites, or manual, as was the case at site FI.1 in Finland. In the former sites a continuous flow of data was available, whereas in the latter case the site had to be visited at intervals varying from a week to a month to take readings.

In Ireland a simpler manually operated system was used to monitor moisture and temperature variations in the two test sites. The sensors used are constructed from fibreglass wrapped in a non-corrodible metal protection with wire leads connected to a portable meter which is capable of measuring changes in moisture and temperature. The meter is a battery powered alternating current ohmmeter, which includes a thermistor for measuring temperature. Temperature readings are read directly from the meter, while moisture content is measured by recording resistance readings and relating these values to moisture content by calibrating resistance and moisture content for each type of soil or granular material monitored. At sites EI.1-3, readings were generally taken at monthly intervals.

3.4 Procedures and Experiences Involved in the Use of Field Measuring Equipment

The procedures used in the installation of the equipment for measuring moisture and temperature varied slightly from one country to another. In Iceland the sensors were mounted in predrilled holes in the vertical walls of the sensor trench to ensure that they were located in the undisturbed part of the pavement. In Iceland it was considered that subsequent traffic loading would eliminate voids around the sensors. The installation procedure used in Iceland is shown in Figure 3.1.

a) Measurement of Layer Thickness

b) Sub-base TDR and Compaction Equipment

c) Measurement of Depth to TDR Probe

d) Installation of TDR Probe in Basecourse Layer

e) Datalogger and Solar Cell Mast and Instrumented Pit after Compaction of Unbound Layers

f) Compaction of Surface Dressing Layer

FIGURE 3.1: INSTALLATION OF MOISTURE MEASURING INSTRUMENTS IN ICELAND

In Finland, at Site FI.1, the probes were installed horizontally on a flat surface compacted with a light plate compactor. As TDR probe readings are sensitive to air gaps around the conductors, which can decrease the true moisture content values, fine material was placed around the probes followed by normal production material and the granular layers then compacted in accordance with good engineering practice. The asphaltic concrete surfacing was laid two weeks after installation of the moisture probes. Initially there were fears that the use of the fine material surrounding the material would have adversely effected the recorded values. However, a comparison of the test results recorded in the autumn of 1998 and 1999 shows that the Finnish results were sensible in relation to the initial readings taken in August 1998. The Finnish results from FI.1 are reported in Figure 3.5.

Difficulties were encountered at Sites SI.1 and PT.1 as the initial recordings were not considered reliable by the researchers responsible for monitoring these sites. In the case of Site SI.1, the initial test installation had to be dismantled, the sensors re-calibrated and installed a second time, after which, credible readings were recorded. In the case of Site PT.1, even after re-calibration of the sensors, it was not possible to correlate sensor readings with moisture contents for the materials under investigation, and moisture/temperature monitoring at this site was abandoned.

In the case of Sites EI.1-3, fine material sieved from the main material was placed by hand around the sensor probes to protect them and ensure that close contact between the sensor probes and material occurred. In the case of Site EI.3 where the gravel was re-compacted in the trial trench with a plate compactor, further densification occurred after re-opening to traffic. Because of this and the time needed for the moisture to be absorbed into the probes, the moisture content readings for the first two months from this site are probably suspect. At Site EI.1&2, where crushed limestone was used in the base and sub-base layers, after installation procedures similar to Site EI.3, full-scale compaction plant was used to finally compact the crushed stone base and the bituminous surfacing was laid about two weeks after installation of the sensors. This ensured that the conditions were comparable with those that prevail in normal new road construction.

In Slovenia difficulties were experienced when the TDR probes were installed on the first site in 1998, as the recorded readings were too high for the materials under investigation. Because of the unreliability of the data from this site, the sensor probes were removed from this site. An extensive programme of calibration was then carried out in the laboratory using different pavement materials before the probes were re-installed in a second test site where pavement reconstruction was carried out in early 1999. The pavement construction on this test site was 200mm of cement (22kg/m^3) stabilised clayey gravel over a silty or clayey gravel sub-base.

The sensor probes were installed horizontally on a flat surface of sub-base, compacted with a light plate compactor. Fine material from the silty gravel sub-base was placed around the probes, after which the probes were covered with the original granular material from the excavation pit and compacted according to good engineering practice. Sensor probes were placed in a similar manner in the stabilised base layer before the bituminous surfacing was laid.

3.5 Results of Moisture Content and Temperature Readings from COURAGE Sites

The results recorded from the monitoring sites were influenced by:

- The amount of precipitation
- Temperature
- Construction materials used in base and sub-base layers
- Thickness, type and extent of surfacing
- Drainage conditions
- Interval between readings

3.5.1 Sites IS.1-3, Iceland

The most severe conditions encountered were those recorded in Iceland, where continuous recording clearly showed the fluctuations in moisture content which take place in the base and sub-base layers especially between the months of January and May each year. Results from Site IS.2 are reported in Figures 3.2-3.4 which show how precipitation, freezing and thawing of the pavement and subgrade layers, vary throughout the year (Bjarnason et al, September 1999). Of particular interest is the effect of a temporary thaw period in February 1999 on the moisture content of the base course, the freeze-thaw cycles between February and mid-March, and the increases in moisture content of all the layers from mid-March to April during the Spring thaw period. Increases in precipitation, during periods of heavy Summer rainfall, were also noted to give rise to moisture content increases in the base and sub-base layers. At Site IS.2, the gravimetric moisture content varied from a minimum value of 6.1% (optimum – 4.7%) in late September 1998 to a maximum of 14.1% (optimum +3.3%) during the Spring-thaw in April 1999.

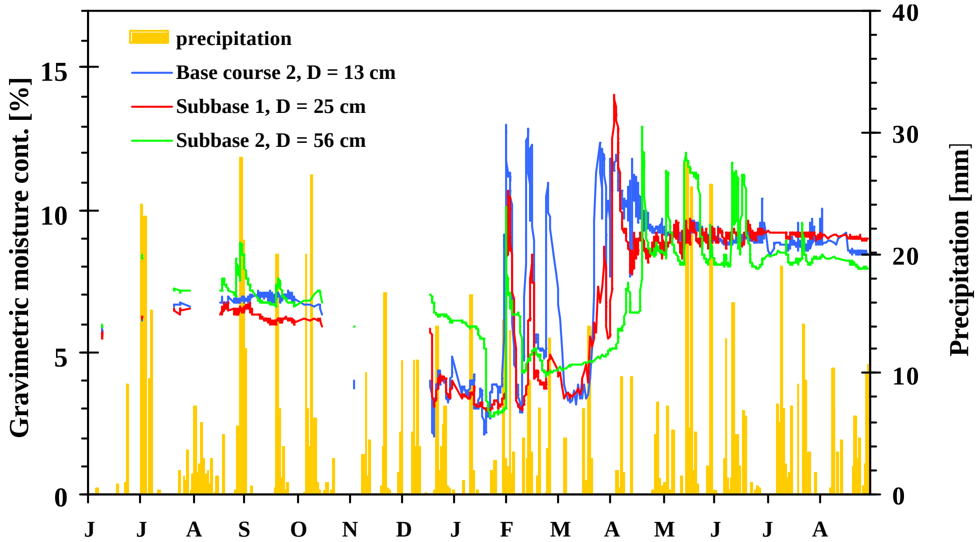

FIGURE 3.2: GRAVIMETRIC MOISTURE CONTENT AT SITE IS.2, ICELAND, PLOTTED ALONG WITH THE DAILY PRECIPITATION DURING THE PERIOD JUNE 1998 - AUGUST 1999

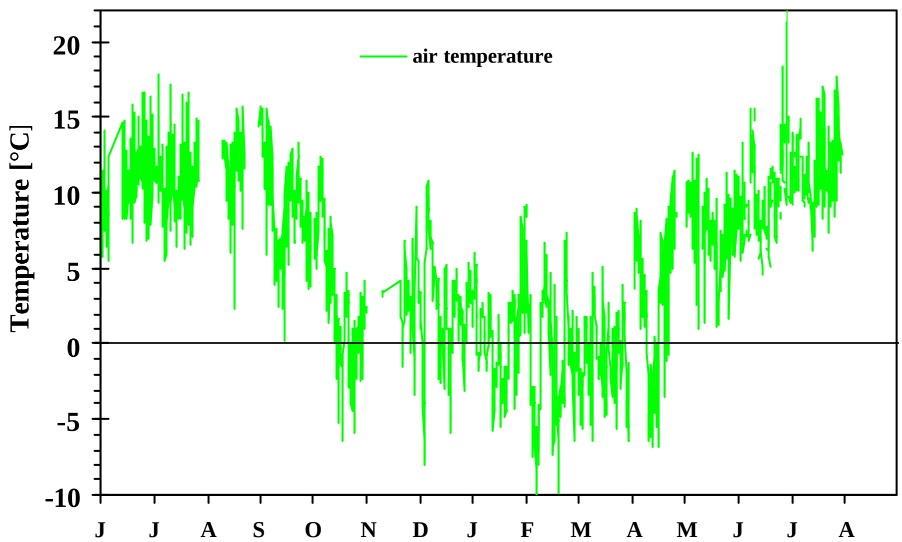

FIGURE 3.3: AIR TEMPERATURE AT SITE IS.2, ICELAND, DURING THE PERIOD JUNE 1998 - AUGUST 1999

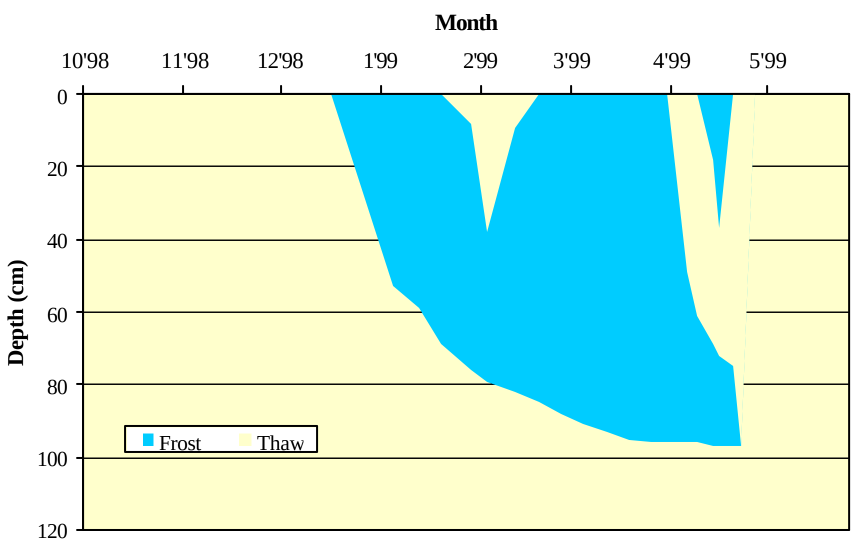

FIGURE 3.4: FREEZE AND THAW AT SITE IS.2, ICELAND, DURING THE PERIOD OCTOBER 1998 - MAY 1999

3.5.2 Site FI.1-2, Finland

The results reported for Site FI.1 (Laaksonen et al, October 1999) indicated much less variability than that recorded in Iceland.

A plot of the gravimetric moisture contents from Site FI.1 is reported in Figure 3.5, which shows that during the one-year period under investigation the moisture content variations were insignificant. The apparent reduction in moisture content in the silty sub-soil and the sandy moraine sub-base between mid-December and April is due to frost action when true moisture contents could not be measured with the TDR probes

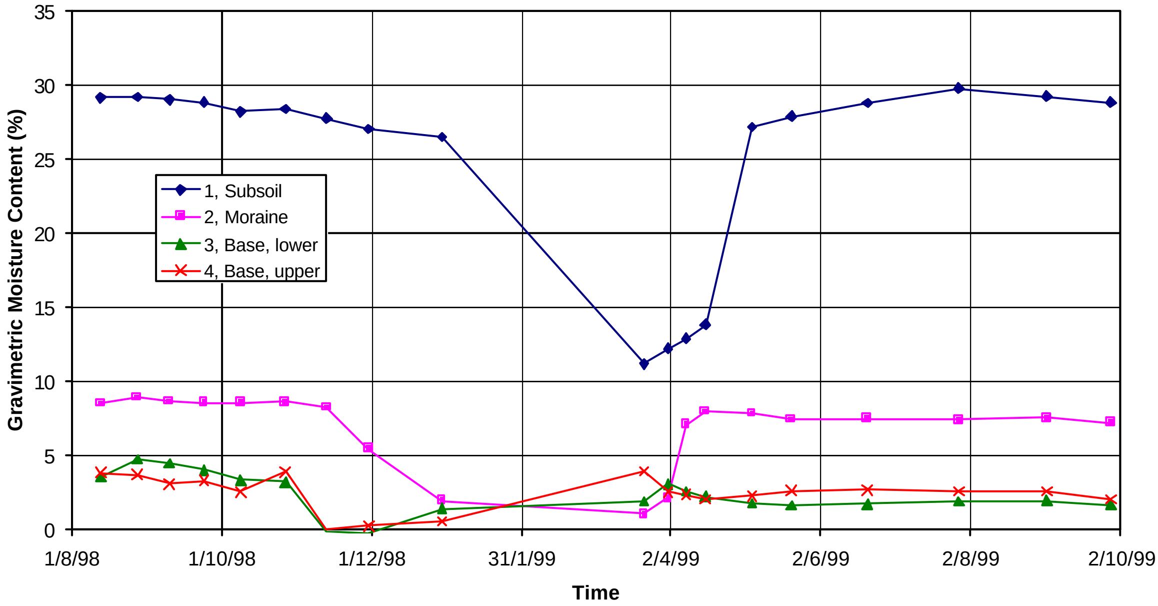

FIGURE 3.5: GRAVIMETRIC MOISTURE CONTENT AT SITE FI.1, FINLAND, DURING THE PERIOD AUGUST 1998 - AUGUST 1999

At Site FI.2 (reported by Suni and Kujala, November 1999) similar trends were observed in the moisture contents of the pavement layer materials and in the underlying filter sand and subsoil layers. This site is located about 500km north of Helsinki where the depth of frost penetration was estimated to be in excess of 2m and the amount of annual precipitation was calculated at 518mm during the period of the investigation. Two sections of road were investigated on this site, one in cutting and the other in raised embankment. The materials and construction depths used in both sections were typical for Finnish road construction. In each of the test sections, two sets of four TDR probes and temperature detectors were installed in the pavement and supporting layers. One set was located near the middle of the road and the other towards the carriageway edge. The variation in the moisture content of all the structural layers followed seasonal variations, with the moisture content being highest in Autumn and Spring. The greatest variations were observed in the lower filter and sub-soil layers. In the crushed stone base and the gravel sub-base layers the moisture content and variation in moisture content was lower than in the underlying layers. The moisture contents in the pavement layers in cutting were slightly higher than those recorded in the embankment section.

The variation of gravimetric moisture content with time for the cut section at Site FI.2 is shown in Figure 3.6. The reduction in moisture content from January to mid-April, which was most evident in the lower layers was due to frost action. However, it appears that outside this period of the year, there is very little variation in moisture content in the respective layers.

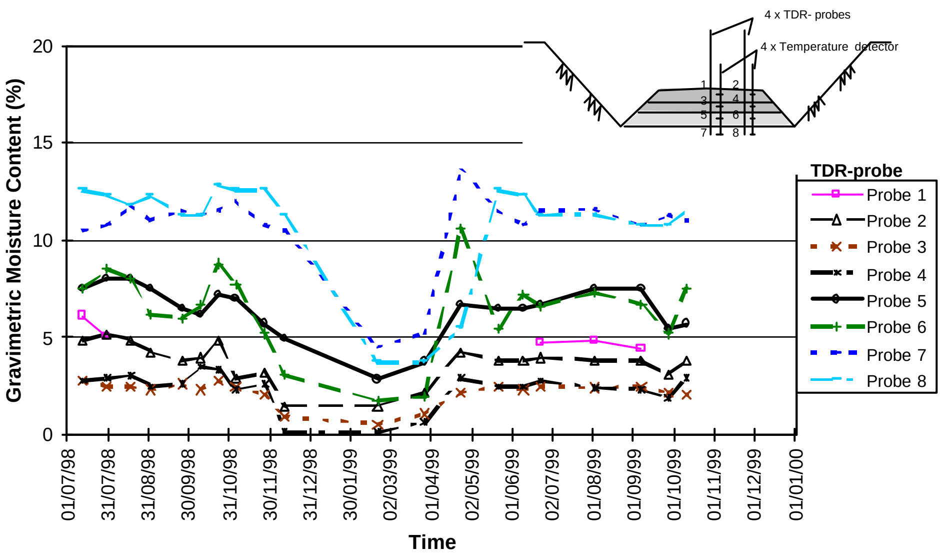

FIGURE 3.6: GRAVIMETRIC MOISTURE CONTENT AT SITE FI.2, FINLAND, DURING THE PERIOD JULY 1998 - OCTOBER 1999

3.5.3 Sites EI.1-3, Ireland

After the initial installation period when the granular materials were reaching a stable moisture content and the sensors were slowly absorbing the moisture from the surrounding granular materials, the moisture contents of the materials have remained very stable at both sites since April 1999. The results to date indicate that the moisture content of these layers is effected minimally by weather conditions in Ireland, which are less severe than in Iceland or Finland. The stable moisture content regime in the granular materials in the Irish pavements is greatly influenced by the 3m width of hard shoulder, which has the same depth of surfacing as the carriageway so that infiltration of water through the shoulder cannot take place.

FIGURE 3.7: PROVISION OF 3M HARD SHOULDER ALONG IRISH ROAD AT SITE EI.3

Frost penetration seldom gives rise to problems in Irish roads, but precipitation is higher in Ireland than in most European countries and, because of this, the provision of impervious surfacings to pavements with unbound bases is considered to be very important. At Sites EI.1-2, located in the north-west of the country, annual precipitation during the period of investigation was 1243mm, while at Site EI.3 the figure was 909mm.

The moisture content recorded for the granular materials at Site EI1-2 was about 2% while those at Site EI.3 was 3.9%, or about 2%-2.5% below the optimum moisture content. The moisture content of the gravel at Site EI.3 is similar to that recorded at the time of construction of the road in 1978. The in-situ moisture contents determined for Sites EI.1-3 compare favourably with in-situ moisture contents determined during previous investigations of granular road pavements in Ireland.

3.5.4 Site SI.1, Slovenia

The results of the variation in moisture content of the pavement materials from Site SI.1, recorded every four hours between July and September 1999, are reported in Figure 3.8. They show that the values recorded for the cement-stabilised base are generally about 4% and compare favourably with laboratory calibration values for the same material. They also show that after heavy rainfall there were noticeable increases in moisture content in the stabilised base. In the case of the unbound sub-base layer, excessively high moisture contents up to 20% were recorded and such high values do not correspond with laboratory calibrations for this material. The results available from this site show, however, the same relationship between precipitation and moisture content in the pavement layers as was demonstrated in the Icelandic sites. The temperature of the unbound base layer for the period July-September 1999 varied between 20°C and 28°C.

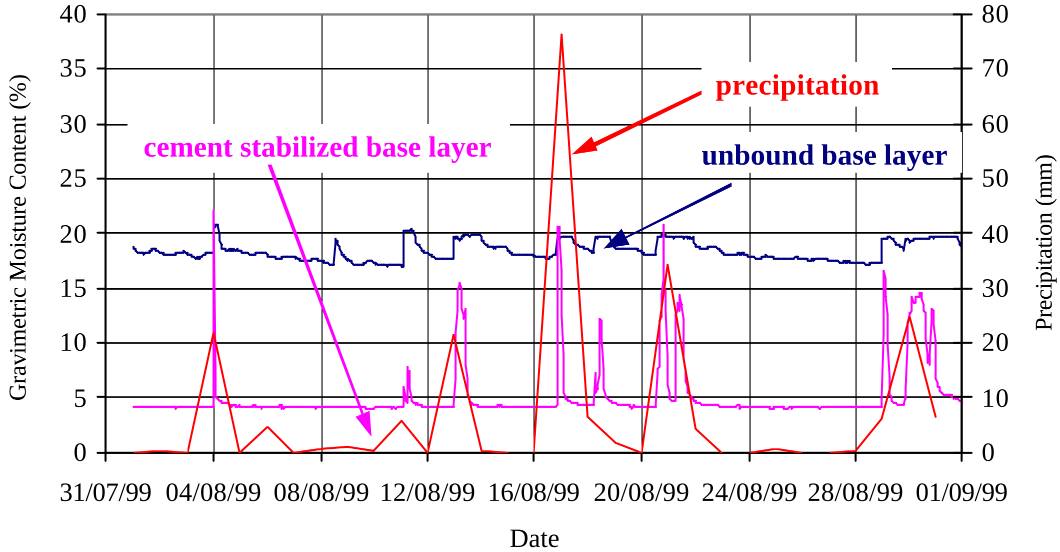

FIGURE 3.8: GRAVIMETRIC MOISTURE CONTENT AT SITE SI.1, SLOVENIA, DURING THE PERIOD JULY - SEPTEMBER 1999

3.6 Falling Weight Deflectometer (FWD) Measurements

FWD surveys are a valuable means of monitoring the strength of the pavement structure as it varies during the year, and it also allows comparisons to be drawn between similar pavement types from the different countries participating in the COURAGE Project.

The parameters from FWD surveys which are most used to indicate the bearing capacity of pavements are the central deflection (Do) under the influence of a 50kN falling weight, or the moduli of the pavement layers which can be back-calculated from the survey readings. The central deflection gives an indication of the strength of the pavement and supporting layers while the outer deflection measurements give an indication of the subgrade condition or that of the lower pavement layers.

Where major changes take place in the moisture content of granular layers in road pavements, as was noted in the case of the Icelandic test sites, there is an obvious need to monitor the strength of such pavements during the Spring thaw period. The seasonal changes in the bearing capacity of granular road pavements in Iceland is clearly illustrated in Figure 3.9, where central deflections measured under a 50kN load are reported for the three Icelandic test sections in 1998 and 1999. In both years, large increases in deflection occurred between April and May, at the end of the Spring thaw, and these deflections steadily reduced between the months of May and August.

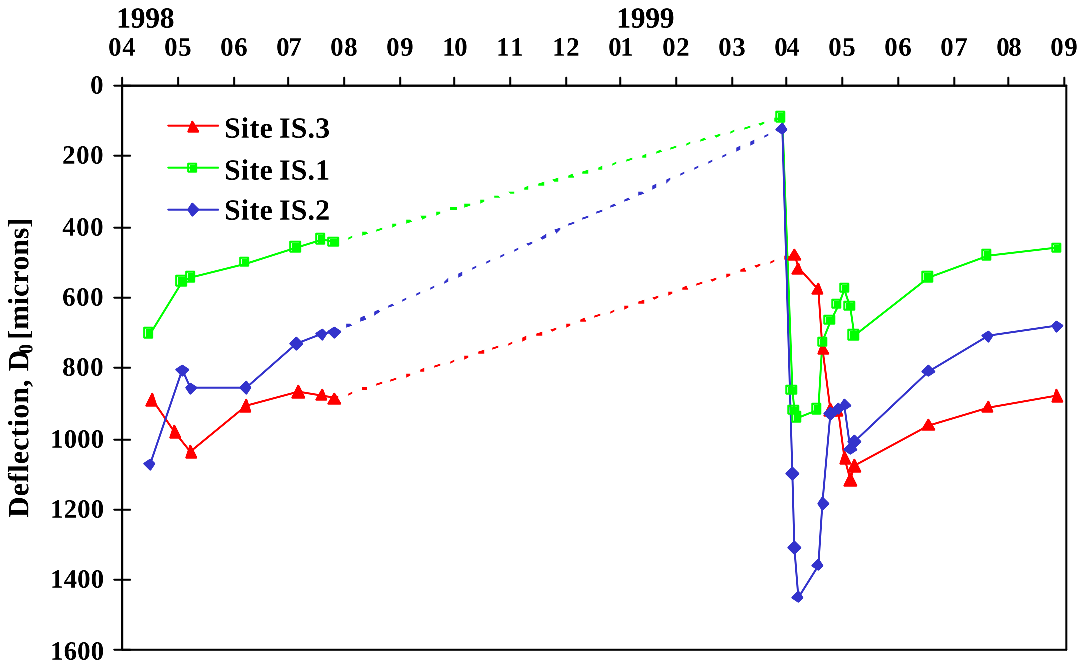

FIGURE 3.9: CENTRAL DEFLECTION (DO) MEASURED UNDER 50KN LOAD AT SITES IS.1-3, ICELAND, DURING THE PERIOD APRIL 1998 - AUGUST 1999

The results of FWD surveys carried out on sites in the different countries which participated in the COURAGE project are summarised in Table 3.2. Deflections measured at the centre of a 50kN load, Do, and at distances of 900mm and 1200mm from the load are reported. The results show that there is considerable variation between the experimental pavements in the different countries. These differences are partly due to weather conditions, pavement construction depths, type of granular materials used, and the type, depth and temperature of bituminous surfacing. Other factors which influenced the FWD readings were the strength of the underlying subgrade layer and the thickness of surfacing used on the shoulders. The high deflections measured at Site IS.2 are greatly influenced by the peat subgrade. At Site FI.2, where FWD readings were taken in both the inner and outer driving lines, it was shown that the readings on the outer line were about 5%-10% higher than those recorded towards the centre of the pavement. This occurred even though the shoulder was surfaced for a width of 1.25m.

A comparison of the FWD results from Sites FI.2 in Finland with the two Irish sites, shown in Figure 3.10, indicates that where a sufficient depth of good quality granular material is used, and the soil support conditions are favourable, a strong serviceable pavement can be constructed. The autumn results from Site FI.2 are comparable with results from Site EI.3. Both of these sites compare favourably with Site PT.1 where a greater thickness of bituminous material was used at the expense of the granular layer depth. The lowest deflections were recorded at Site EI.2 where the combination of a rock subgrade, and the use of high quality crushed limestone in the base and sub-base layers, produced a very strong stiff pavement.

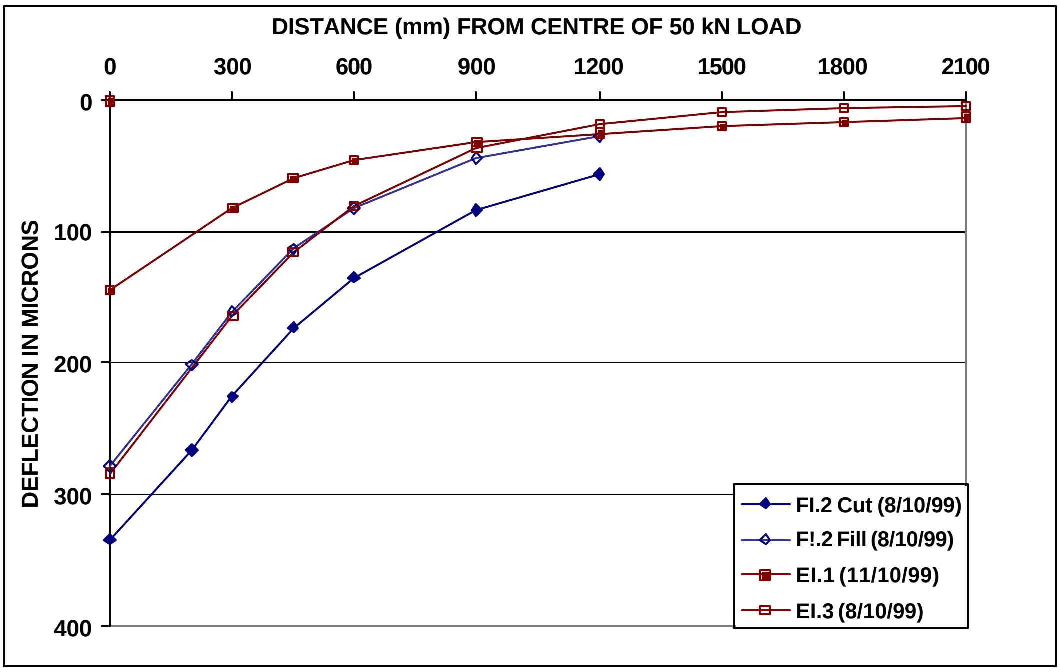

FIGURE 3.10: COMPARISON OF DEFLECTION MEASURED UNDER 50KN LOAD AT SITES FI.2, FINLAND AND EI.1AND 3, IRELAND

Site	Date	Pavement Layer Thickness (mm)		Mean Deflections (Microns)				Remarks
		Surfacing	Base + Sub-base	Do	D900	D1200	D1500	
IS.2	31/3/99	37	770	122	80	-	-	Pavement frozen. Thaw period. Peat subgrade.
	9/4/99			1449	438	-	-	
	27/4/99			934	322	-	-	
	30/8/99			681	259	-	-	
IS.3	9/4/99	36	814	517	91	-	-	Moraine subgrade
	8/5/99			1076	204	-	-	
	30/8/99			877	197	-	-	
FI.1	9/10/98	50	500 Base + 430 sub-base	1011	187	-	88	Silt subgrade
	11/5/99			983	197	-	94	
	28/9/99			858	160	-	79	
FI.2/1	13/10/98	80	500+300 Filter sand Layer	340	83	56	-	Road in cutting.
	19/11/98			71	41	34	-	Pavement frozen.
	28/5/99			374	85	60	-	Spring thaw.
	8/10/99			334	84	57	-	Outer driving line.
FI.2/2	19/11/98	80	500+300 Filter sand Layer	38	20	17	-	Road in
	28/5/99			295	42	30	-	embankment
	8/10/99			278	45	29	-	Outer driving line.
EI.1	11/10/99	100	300+ 600 Capping	145	32	26	20	Boulder clay subgrade.
EI.2	11/10/99	100		98	8	5	3	Rock subgrade.
EI.3	26/3/99	100	600	348	39	20	11	Firm boulder clay subgrade. Road in service over 20 years.
	8/10/99			284	37	19	9	
PT.1	19/10/98	295	535	256	108	83	62	Site located at Section 2 of IST experiment.

TABLE 3.2: DEFLECTIONS MEASURED UNDER FWD 50KN LOAD FROM TEST SITES IN THE PARTICIPATING COUNTRIES

3.7 Summary of Key Findings

- Considerable variations in the moisture content of unbound base and sub-base materials were observed, particularly in some experimental sites in the Northern countries. This was primarily due to material type, temperature and rain/snow fall, but in other countries the variation was less marked. A material of low OMC has a much lower moisture variation than a material of high OMC. Thus, relative moisture content variations are important for all materials.

- Test pavements showed the following variations of moisture content relative to optimum:

➢ Finland, Site FI.1	base 1 layer, RMC = 42% to 81% of modified Proctor OMC
	base 2 layer, RMC = 35% to 98% of modified Proctor OMC
	sub-base layer, RMC = 122% to 151% of modified Proctor OMC
➢ France, (test track)	base layer, RMC = 42% to 98% of modified Proctor OMC
➢ Iceland, Site IS.1-3 #	base 2 layer, RMC = 67% to 133% of standard Proctor OMC
	sub-base 1 layer, RMC = 44% to 131% of standard Proctor OMC
	sub-base 2 layer, RMC = 54% to 119% of standard Proctor OMC
➢ Ireland Site EI.1-2	base layer, RMC = 43% to 68% of modified Proctor OMC
	sub-base layer, RMC = 43% to 68% of modified Proctor OMC
Site EI.3	base layer, RMC = 54% of modified Proctor OMC
	sub-base layer, RMC = 54% of modified Proctor OMC
➢ Portugal, PT.1*	sub-base layer, RMC = 27% to 97% of modified Proctor OMC

It should be noted that high levels of moisture in the Icelandic pavements are strongly influenced by short thaw periods during winter and the spring-thaw period in April.

* measurement ranges applicable to the period of time shortly after construction.

- Increases in moisture content of unbound granular materials were more likely after periods of high precipitation and after the Spring thaw in Northern countries.

- In pavement structures through cuttings the moisture content was slightly higher, but the variation was lower, compared to the pavement structure founded on an embankment. For structures through cuttings, water has the opportunity to flow into the pavement layers particularly if deep side drains are not installed.

- The variations in moisture content in unbound base and sub-base layers could be significantly reduced by providing an adequate depth of impervious bituminous surfacing and by providing a sufficiently wide "hard" shoulder with the same surfacing as the carriageway.

- FWD surveys showed that in Northern countries, increases in moisture content of the granular layers were accompanied by increased pavement deflections and a reduction in strength which was most noticeable after the Spring thaw.

- The strength of the experimental pavements monitored during the project varied considerably. The Portuguese site in particular exhibited quite high deflections considering the large total thickness of the bituminous materials (295mm). However, FWD results from two sites in Ireland and one site in Finland, showed that provided sufficient depths of high quality granular material for the prevailing subgrade conditions are used, strong pavements with a long life can be constructed mainly with granular materials.

- Difficulties were encountered in the installation and operation of TDR proves on some sites. The evidence from the experimental sites suggests that more work is needed to develop equipment which can confidently monitor low moisture contents (<4%) in granular materials.

4. LABORATORY TESTING

4.1 Introduction

The laboratory testing for the COURAGE project deals with the mechanical characterisation of existing highway unbound granular materials (UGMs). The main objectives of this work were to:

1) define tests and tests procedures suitable to determining the functional properties of unbound granular materials. Specialised tests such as repeated load triaxial tests were used.
2) compare the results obtained from new performance tests with those of simple tests, easier to perform in practice. The tests were simplified mechanical performance and identification tests.
3) assess a number of granular materials, which were representative of those used in different European Countries, with these two types of tests.

4.2 Laboratory Testing Program

4.2.1 Selection of materials

A number of crushed natural aggregates were selected across the European region for the laboratory test program. The primary materials used were:

- gneiss from France
- granite from Portugal
- limestone from Slovenia

In addition, two other materials were introduced as they were considered more marginal in quality. They were:

- recycled crushed concrete mixed with asphalt planings from United Kingdom (RCC&A)
- "flaky" gravel from a failed pavement in Slovenia

Further, the project also included the assessment of a number of aggregates from Ireland due to the generous parallel funding provided by Roadstone Ireland. The Irish materials assessed were:

- greywacke
- white reef limestone
- dark argillaceous limestone
- ryolite
- crushed limestone
- crushed granodiorite
- basalt/dolerite

Following material selection, split representative samples of each of the materials were sent to the different laboratories involved in testing.

4.2.2 Selection of tests

A distinction in the laboratory testing was made between 'simple' tests, which are empirically based, to those described as 'functional' which are aimed at testing material 'elements' by replicating pavement field conditions. Table 4.1 lists the simple tests performed and Table 4.2 the functional tests.

The index tests chosen to characterise resistance of aggregates to fragmentation were the Los Angeles abrasion test, modified Bg index test, the Dutch static compression test and the gyratory compaction test. The modified Bg index test was developed in Iceland and therefore introduced into COURAGE as an innovative method capable of being performed with equipment used by most laboratories. The LA test was chosen as a method because the test has, as well as the German "Schlagversuch" test, been adopted as a CEN standard test (EN 1097-2). The Dutch static compression test has also been selected because it is also a test introduced to CEN TC 154. The procedure used was according to the Dutch National Standard, NEN 6245. Further details of this test may be found in "The Icelandic Status Report 2" (Bjarnason et al, February 1999). In addition, PRA Iceland was one of 16 European national laboratories that took part in a DG XII round robin project for that test. A novel fragmentation test used by COURAGE is the gyratory compaction test.

Test	Material				
	Gneiss	Granite	Limestone	RCC&A	Flaky Gravel
General Material Assessment:					
Petrographic Description	✓	✓	✓	✓	✓
Flakiness Index (Shape)	✓	✓	✓	✓	✓
Plasticity (LL, PL, PI)	✓	✓	✓	✓	✓
Compaction (Modified Proctor)	✓	✓	✓	✓	✓
Specific Gravity	✓	✓	✓	✓	✓
Fragmentation / Abrasion Assessment:					
Los Angeles (LA)	✓	✓	✓	✓	✓
Modified Bg Index (MBg)	✓	✓	✓	✓	✓
Dutch Static Compression (DSC)	✓	✓	✓	✓	✓
Wet/Dry Strength (10% Fines) #	-	-	-	✓	-
Micro Deval (MDE)	✓	✓	✓	-	-
Gyratory Compaction (PCG)	✓	✓	✓	✓	-
General Strength/Stiffness Assessment:					
Static Triaxial (STXL)	✓	U	U	✓	U
Ultrasonic Wave Velocity (UWV)	✓	✓	✓	U	U

TABLE 4.1: SIMPLE TESTS APPLIED TO SELECTED MATERIALS
# Additional testing performed above the initial requirements of COURAGE program
U = Unable to test materials due to equipment limitation

Test	Material				
	Gneiss	Granite	Limestone	RCC&A	Flaky Gravel
Performance Assessment:					
Repeated Load Triaxial (RLT)	✓	✓	✓	✓	✓
Wheel Tracking (WT)	✓	✓	✓	-	-

TABLE 4.2: FUNCTIONAL TESTS APPLIED TO SELECTED MATERIALS

As mentioned, the functional tests aim to test material 'elements' by replicating pavement field conditions. It should be remembered that materials are compacted in the field at different density conditions, viz:

- basecourse material ranging from 97 to 100% of maximum dry density (modified compaction)
- sub-base material ranging from 95 to 97% of maximum dry density (modified compaction)

also, a material's moisture state may vary from the time of compaction to:

- peaks levels in winter possibly near optimum moisture content (modified compaction)
- low levels in summer possibly as low as 40% optimum moisture content (modified compaction)

As a result, the COURAGE project has selected a range of density and moisture content state conditions for RLT testing to investigate the effect of these variations on resilient modulus and permanent strain. In addition, three distinct material gradings were selected (low fines, control and high fines gradings) as a basecourse or sub-base supply product may vary from one quarry to the next in accordance with varying specification tolerances allowed from one country to the next.

Grading was considered an important variable to test as part of the COURAGE program. It is considered that a dense mixture, containing selected percentages of coarse and fine aggregate sizes, will allow the material in a pavement structure to withstand the stresses imposed upon it. A dense mixture may be obtained when their particle size distributions tend towards the theoretical distribution given by Fuller's power grading law. The particle size distributions limits adopted by most of the highway authorities followed this distribution. If fines are excessive, the mixture may lack stability; if the fines content is low, the mixture will tend to be stony and porous, and usually require additional fines to obtain good stability. In addition, segregation may occur due to a lack of middle aggregate sizes or bad surface finish due to too much, or too little, fines. As might be expected, the addition of fines to a granular mixture at first increases density and strength as the interstices are filled, and then decreases density and strength as the fine grained material takes up a larger amount of space and prevents interlocking.

Given that grading is an important issue for material performance, the three high-grade aggregates used for RLT testing were tested for three different gradings. The gradings were categorised according to the percentages of fines (passing a 75μm sieve) for each selected material (refer to Table 4.3). The reference, or control grading, varied slightly between the three materials according to the quarry's typical production grading. In addition, the recycled crushed concrete and asphalt blended product, like most products derived from these constituent materials, was low in fines. Since it is mostly unlikely that a product of this nature will give rise to high fines testing was only performed for the reference and UK supply gradings for this material.

Sieve Size (mm)	**Coarse Grading (3%)**	**Reference or Control Grading (5 to 7% fines)**			**Fine Grading (10% fines)**	**Supply Grading**
		Portugal	**France**	**Slovenia**		**UK**
20	100		100		100	98.9
14	68		79.7		91	84.8
10	55		68.6		82	72.4
6.3	42		58.3		70	56.5
4.0	32		45		60	44.4
2.0	22		33.3		49	32.6
0.6*	-	-	-	-	30	17.6
0.5*	11		16.4			-
0.212 **	-	-	-	-		6.6
0.2 **	7		10.8		20	-
0.075	3		7.4		10	2.7

TABLE 4.3: SELECTED GRADING CURVES FOR RLT TESTING FOR MATERIALS
NOTE: * and ** either / or sieve size

For each percentage of fines, different moisture and density conditions were selected. Table 4.4 summarises the test various conditions adopted for the three high-grade aggregates. It was proposed that, twelve RLT tests were performed in total per material, one at each density and moisture combination with an additional (repeat) test at the 'central' test state. Some additional tests were performed on the granite and limestone materials to more rigorously assess their performance.

Gradings	Moisture Content			Dry Density
Control grading (5% to 7% fines according to the country)		w_{OPM} - 2%		100% $\rho_{d\,OPM}$
	w_{OPM} - 3%	w_{OPM} - 2% (2 tests) followed by method B	w_{OPM} - 1%	97% $\rho_{d\,OPM}$
		w_{OPM} - 2%		95% $\rho_{d\,OPM}$
Coarse grading (low fines content = 3% fines)	w_{OPM} - 3%	w_{OPM} - 2% followed by method B	w_{OPM} - 1%	97% $\rho_{d\,OPM}$
Fine grading (high fines content = 10% fines)	w_{OPM} - 4%	w_{OPM} - 2% followed by method B	w_{OPM} - 1%	97% $\rho_{d\,OPM}$

TABLE 4.4: MATERIAL TESTING CONDITIONS FOR RLT (METHOD A) - ALL MATERIALS
w_{OPM} = optimum moisture content $\rho_{d\,OPM}$ = maximum dry density

As mentioned, the recycled crushed concrete and asphalt material was tested for two gradings variations and only at 97% of OMC (see Table 4.5).

Gradings	Moisture Content			Dry Density
Control grading (7% fines)	w_{OPM} - 6%	w_{OPM} - 4% (2 tests) followed by method B	w_{OPM} - 2%	97% $\rho_{d\,OPM}$
Supply grading (2.7% fines)	w_{OPM} - 6%	w_{OPM} - 4% followed by method B	w_{OPM} - 2%	97% $\rho_{d\,OPM}$

TABLE 4.5: MATERIAL TESTING CONDITIONS FOR RLT (METHOD A) - RCC&A ONLY
w_{OPM} = optimum moisture content $\rho_{d\,OPM}$ = maximum dry density

In addition to controlling the grading, the quality of fines needs to be carefully controlled. A significant quantity of clay in the mixture will attract water, which, since it cannot escape by evaporation, will accumulate within the layer. This in turn will cause a softening of the roadbase and may eventually lead to the complete destruction of the pavement. The plasticity indices were tested for to determine fines quality of all materials.

The testing methodology and stress path sequences for the RLT test are set in accordance with the draft CEN procedure prEN (WI00227138). Further details are presented in Annex 4.

The conditions chosen for the wheel tests aim to provide a comparison between the three high-grade aggregates assessed using this test. All materials were to be tested at the Control grading according to the conditions outlined in Table 4.6.

Parameters	Test Conditions
Materials	Gneiss, Granite, Limestone
Pavement Structure	Granular, 30mm asphalt seal
Grading of UGM	Control Grading
DDR of UGM	97%$\rho_{d\,OPM}$
MC of UGM	55 - 60% of OMC (max.) ***or*** w_{OPM} - 2.4 to 2.7% for Gneiss and Granite w_{OPM} - 1.9 to 2.2% for Limestone
Surface Wheel Load	approx. 670kPa at top of asphalt layer
Measurement Intervals	5, 10, 20, 200, 500, 750, 1000, 1500, 2000, 5000, 7500, 10000, 15000, 20000

TABLE 4.6: TEST CONDITIONS FOR WHEEL TRACKING SIMULATIVE TESTS

The wheel tracking and repeated load triaxial tests enabled the moisture sensitivity of resilient modulus and resistance to permanent deformation of the different materials to be assessed. A summary table illustrating the test methods to be used for assessing the materials properties is presented below (Table 4.7).

Mechanical properties	Functional tests	Simple tests
Resilient behaviour	Repeated load triaxial Simplified triaxial test	Stiffness evaluation by UWV CBR
Resistance to rutting/shear	Repeated load triaxial Wheel trafficking test	Triaxial shear test CBR
Resistance to degradation	Los Angeles Modified Bg-index Dutch static compression Micro Deval Gyratory Compaction	
Residual moisture content	Water retention test	

TABLE 4.7: TEST METHODS USED FOR MATERIAL ASSESSMENT

4.2.3 Relationships between tests

As far as the simple tests are concerned, extensive research has been done in Iceland on aggregates resistance to fragmentation, weathering and abrasion. The report “Aggregates Resistance to Fragmentation, Weathering and Abrasion – Comparison of Different Test Methods” describes the findings of this research (Bjarnason et al, 1999).

This report, whose results are only based on the testing of Icelandic aggregates, demonstrates that the degradation tests are divided into three groups, namely, fragmentation, durability and abrasion tests. The correlation between test results within each group is generally good but not so between test results from different groups. The correlation between the fragmentation test results is the strongest, whilst the durability test results exhibit a weaker correlation than within the fragmentation tests. The correlation between the abrasion tests is good. A 'factor analysis' was used to investigate the different test grouping categories.

4.3 Laboratory Procedures

Most test procedures within the COURAGE test program used, where possible, CEN or draft CEN procedures. The test procedures for any 'novel' tests are reported in Annex 4.

4.3.1 Fragmentation Tests

4.3.1.1 Modified Bg Index (MBg) Test

The equipment used to perform this test is simply that of the modified Proctor compaction test. After compacting the material according to the Proctor method, the fragmentation is determined by comparing the modified grading curve, resulting from compaction, with the known initial grading curve. The sum or the difference in percentage remaining on each sieve is calculated (all plus or minus values). The more fragmentation that occurs, the higher the modified Bg index value that is obtained. Further details of this test may be found in "The Icelandic Status Report 1" (Bjarnason et al, February 1999).

4.3.1.2 Gyratory Compaction Test

This test aims to simulate the type of compaction conditions to which materials may be subjected in the field as it combines an applied static vertical load along with a gyratory action. The test aims to determine the amount of material breakdown, at a range of sieve sizes, which occurs under this type of compactive process. Material breakdown curves are established for a number of 'key' sieve sizes as a function of the applied number of gyrations. This novel test is described in detail in Annex 4.2.

4.3.2 Static Triaxial Test

The test was used to evaluate the shear strength of the materials tested under COURAGE in terms of the maximum principle stresses obtained at the point of failure. The specimens were tested in a four stage drained test, using confining pressures of 10, 20, 40 and 80kPa. The strain rate used was 0.5mm/min to allow for the dissipation of internal pore water pressures in what are considered relatively "free-draining" granular materials.

4.3.3 Ultrasonic Wave Velocity Test

This procedure covers the testing of a material to determine its density and elastic properties by using ultrasonic wave pulses travelling through the material. The object is to measure the time taken for the onset of the pulse to propagate through the specimen. If one was to estimate a "stiffness", then the resonant frequency test may be used to determine the fundamental frequency of vibration of the specimen and, hence, Poisson's ratio. For COURAGE, testing was only performed to investigate if a relationship between propagation time and the RLT measured characteristic resilient modulus exists. Also, to see if propagation time is dependent on the material's moisture content. Only the gneiss material was tested in this way.

4.3.4 Repeated Load Triaxial Test

The testing method and stress path sequences for the RLT test are set in accordance with the draft CEN test procedure prEN (WI00227138). Testing carried out for the project utilised test method Method A (variable confining pressure), where the cell pressure is cycled in phase with the load for different moisture content (w) and dry density (ρ_d) conditions. Details are given in Annex 4.1.

4.3.5 Wheel Tracking Test

Wheel tracking testing is performed for COURAGE using the Pavement Test Facility at the University of Oulu. The Pavement Test Facility is a laboratory-scale test track device with a moving vehicle load (Figure 4.1). It can be used easily and reliably to simulate the stresses caused by traffic in the road structure (Lämsä, et al, 1999).

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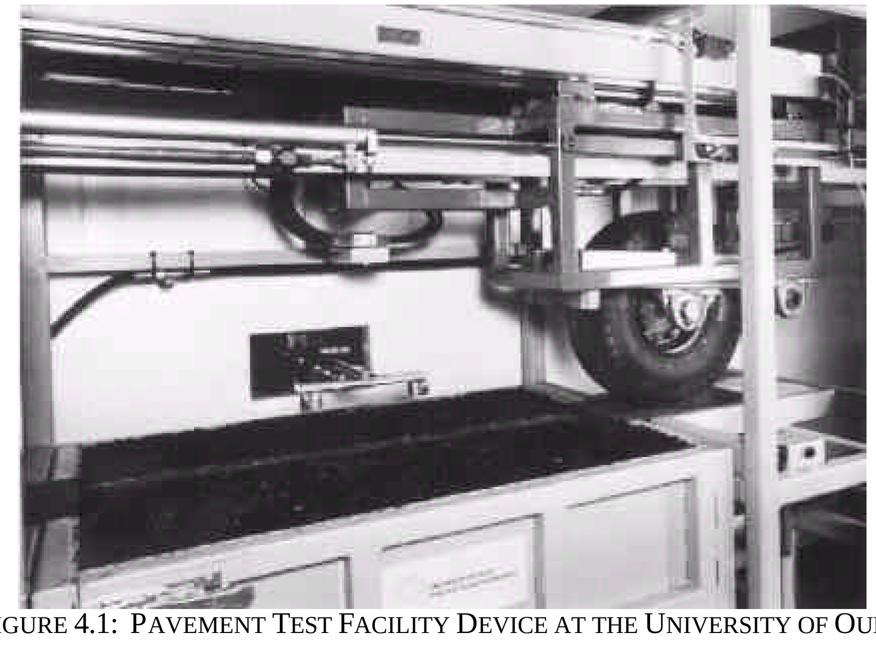


FIGURE 4.1: PAVEMENT TEST FACILITY DEVICE AT THE UNIVERSITY OF OULU

The pavement structures examined were constructed in a test box with internal dimensions (generally) being 1200 x 900 x 600 mm (length x width x height). A pavement configuration of a

30mm asphaltic concrete surfacing overlying a 300mm granular material base, with an underlying 270mm if filter sand was used as the test structure (see Figure 4.2). Three different test structure boxes, one for each of the three COURAGE material types, were constructed in this research. The moving wheel load selected was 9.15kN, with a measured static pressure at the surface of the asphaltic concrete layer being 670kPa. The loading speed at the centre of the test box was 1.4m/s (5km/h).

The tests were performed for approximately 20,000 loadings and measure the permanent and elastic deformations of the test layers at three different levels, as shown in Figure 4.2. Simple computations were made to determine the deformations in the top 150mm and bottom 150mm of the unbound granular material.

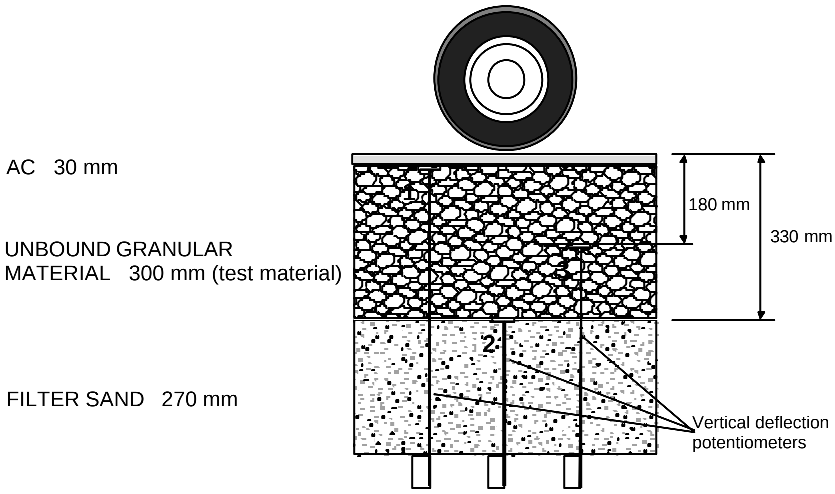


FIGURE 4.2: TEST PAVEMENT STRUCTURE

5. MODELLING OF LABORATORY RLT TEST DATA

5.1 Introduction

The COURAGE project has used the repeated load triaxial test as its prime method of assessing the performance of unbound granular materials. However, the results of these tests obtained at a variety of stress conditions and density/moisture states must be described in a manner suitable for inclusion in analytical pavement design procedures. This has been achieved using non-linear constitutive relationships for the material resilient stress-strain behaviour in order that their parameters can be used in structural pavement analysis computer code to iteratively reach a stress-strain solution. Some of the models evaluated in the parallel COST 337 Action were:

- k-θ model (one of the most widely used in practice),
- Uzan model
- isotropic and anisotropic Boyce models

The study concluded that the isotropic and anisotropic Boyce models gave the best predictions. It should be noted that the Dresden model was not investigated in that study.

5.2 Models Selected for COURAGE Materials

In order to provide models which could be applied in a finite element code for response and performance modelling of pavement structures with unbound granular bases, two non-linear elastic models were used in the COURAGE project:

- Boyce model using the isotropic form and a modified anisotropic form
- Dresden model

The two models have been selected on the basis of:

- the good predictions obtained with the Boyce models in the preliminary study carried out in COST 337
- their ability to describe results of repeated load triaxial tests with variable confining pressure (i.e. the response of the material to a wide range of different stress paths)
- the possibility to use these models for pavement structure calculations, using the finite element method. Both selected models were already implemented in a finite element code (CESAR LCPC for the Boyce model and ANSYS for the Dresden model)

In addition, two models were selected for the permanent strain analysis of the materials. The first being a model used by LCPC and the second a strain rate model used by TSA, Australia.

5.2.1 Hooke's Law 3D

The resilient modulus of a material according to 3-D Hooke's law equation is given by:

$$E_r = \frac{(\sigma_{1d} - \sigma_{3d})(\sigma_{1d} + 2\sigma_{3d})}{\varepsilon_{1r}(\sigma_{1d} + \sigma_{3d}) - 2\varepsilon_{3r}\sigma_{3d}}$$

with:

E_r	[MPa]	calculated equivalent E-modulus
ε_{1r}	[-]	measured axial resilient strain
ε_{3r}	[-]	measured radial resilient strain
σ_{1d}	[kPa]	difference in maximum and minimum principal axial stress
σ_{3d}	[kPa]	difference in maximum and minimum principal radial stress

This assumes the material behaves linear-elastically for any individual stress stage.

5.2.2 Boyce Model

5.2.2.1 Isotropic Boyce Model

This non-linear elastic model was first developed by Boyce (1980), to describe the resilient behaviour of unbound granular materials. It is defined using the following variables:

$p = (\sigma_1 + 2\sigma_3)/3$ = mean normal stress $\qquad$ $q = (\sigma_1 - \sigma_3)$ = shear stress

$\varepsilon_v = \varepsilon_1 + 2\varepsilon_3$ = volumetric strain $\qquad$ $\varepsilon_q = \frac{2}{3}(\varepsilon_1 - \varepsilon_3)$ = shear strain

for the conditions of the repeated load triaxial test.

The Boyce model can be expressed in terms of bulk moduli (a measure of compressibility) and shear moduli (a measure of distortion), with:

$K = \frac{p}{\varepsilon_v}$ = bulk modulus $\qquad$ $G = \frac{q}{3\varepsilon_q}$ = shear modulus

The values of K and G depend on the applied stresses as follows:

$$K = \frac{\left(\frac{p}{p_a}\right)^{1-n}}{\frac{1}{K_a} + \frac{(n-1)}{6G_a}\left(\frac{q}{p}\right)^2} \quad \text{and} \quad G = G_a.\left(\frac{p}{p_a}\right)^{1-n}$$

These expressions show that:

- K and G increase when the mean stress p increases
- K increases when the stress ratio q/p increases

5.2.2.2 'Anisotropic' Boyce Model

Results of repeated load triaxial tests performed at LCPC have shown that some granular materials present a somewhat anisotropic behaviour (characterised by different values of axial strains ε_1 and radial strains ε_3 for isotropic loadings). For these materials, the fit of the experimental results with the Boyce model was not very satisfactory. For this reason, an attempt was made to generalise the Boyce model for the case of a cross-anisotropic material (anisotropy between the vertical and the horizontal directions).

To introduce 'anisotropy' in the model, the expression of the elastic potential proposed by Boyce was modified by multiplying the principal stress σ_1 by a coefficient of anisotropy γ, such the mean normal and shear stress expressions were modified to:

$p^* = (g s_1 + 2 s_3)/3$ = modified mean normal stress $\qquad$ $q^* = (g s_1 - s_3)$ = modified shear stress

with: $\varepsilon_v{}^* = \varepsilon_1/\gamma + 2\varepsilon_3$ $\quad$ and $\quad$ $\varepsilon_q{}^* = \frac{2}{3}(\varepsilon_1/\gamma - \varepsilon_3)$

or

$\varepsilon_v{}^* = \frac{p^*}{K^*}$ $\quad$ and $\quad$ $\varepsilon_q{}^* = \frac{q^*}{3G^*}$

with: $$K^* = \frac{\left(\frac{p^*}{p_a}\right)^{1-n}}{\frac{1}{K_a} + \frac{(n-1)}{6G_a}\left(\frac{q^*}{p^*}\right)^2}$$ and $$G^* = G_a\left(\frac{p^*}{p_a}\right)^{1-n}$$

When $\gamma = 1$, these expressions are identical to those of the initial Boyce model.

The anisotropic model can also be expressed in terms of elastic moduli and Poisson ratios. Their expression is:

$$E_h = \frac{9K^*G^*}{3K^* + G^*} \qquad E_v = \frac{E_h}{g^2}$$

$$\nu_{hh} = \frac{3K^* - 2G^*}{6K^* + 2G^*} \qquad \nu_{hv} = \frac{\nu_{hh}}{\gamma} \qquad \nu_{vh} = \gamma \cdot \nu_{hh}$$

In general, experimental results lead to values of γ lower than 1, and so the stiffness of the material is higher in the vertical direction, which is in agreement with the fact that the material is compacted vertically (both in the laboratory and in the field).

5.2.3 Dresden Model

The second model selected is the Dresden model, used at the University of Hannover. This non-linear elastic model has been developed and modified by WELLNER et al. in the last ten years. The model is expressed in terms of a resilient modulus E and a Poisson ratio ν, where the values of E and ν depend on the applied stresses as follows:

$$E = \left(Q + C s_3^{Q_1}\right) s_1^{Q_2} + D$$

$$n = R\frac{s_1}{s_3} + A s_3 + B$$

with:

E	[kPa]	calculated resilient modulus
ν	[-]	calculated Poisson's ratio
σ_1	[kPa]	absolute value of the maximum principal stress
σ_3	[kPa]	absolute value of the minimum principal stress
Q_1, Q_2	[-]	material parameters
Q	$[kPa]^{1-Q2}$	
C	$[kPa]^{1-Q1-Q2}$	
R, B	[-]	
A	$[kPa]^{-1}$	
D	[kPa]	

5.2.4 Fit of the Model Parameters

A simple procedure for determining the Boyce model parameters (isotropic or anisotropic) from repeated load triaxial test results has been developed. It is an iterative procedure, based on the classical least-squares method. It consists in minimising the following quantities for the N stress paths of each test:

$$\sum_{i=1}^{N}\left[\Delta e^i_{1measured} - \Delta e^i_{1calculated}\right]^2 \quad \text{and} \quad \sum_{i=1}^{N}\left[\Delta e^i_{3measured} - \Delta e^i_{3calculated}\right]^2$$

The method was implemented in a spreadsheet and distributed to participating laboratories.

In addition, a procedure for determining the parameters of the Dresden model has been developed and implemented in an EXCEL sheet using a least squares technique to minimise the difference between measured and predicted (or model calculated) strains. In the Dresden procedure, some restrictions on the values of the parameters Q, Q_1, Q_2, R and C have been introduced, considering previous experience:

$6465 \leq Q \leq 10775$ $\quad Q_1 \geq 0$ $\quad Q_2 = 0.333$
$R \geq 0$ $\quad C \geq 0$

5.2.5 Typical Modelling Results

This section presents an example of the fit of the Boyce model based on the results of a repeated load triaxial test, in order to illustrate the modelling approach. The measured stress and strain parameters used to determine the model parameters are, for each stress path sequence:

- the minimum stresses p_{min}, q_{min} and maximum stresses p_{max}, q_{max} applied (average of several load cycles).
- The resilient volumetric strains ε_v and shear strains ε_q (average of several load cycles).

Figure 5.1 presents a typical example of experimental results obtained in a triaxial test on the Gneiss material. It illustrates the variation of the volumetric and shear strains obtained for each load sequence with the mean stress p and the stress ratio q/p. The resilient behaviour of the material is strongly non-linear, and depends on the stress ratio (or stress path) q/p:

- the resilient volumetric strain ε_v increases when p increases, and decreases when q/p increases;
- the resilient shear strain ε_q increases when p increases, and also when q/p increases;
- ε_q is negative for q/p = 0 , which indicates an anisotropic response of the material (for an isotropic material, when the loading is isotropic (q/p = 0), the shear strain ε_q is equal to 0).

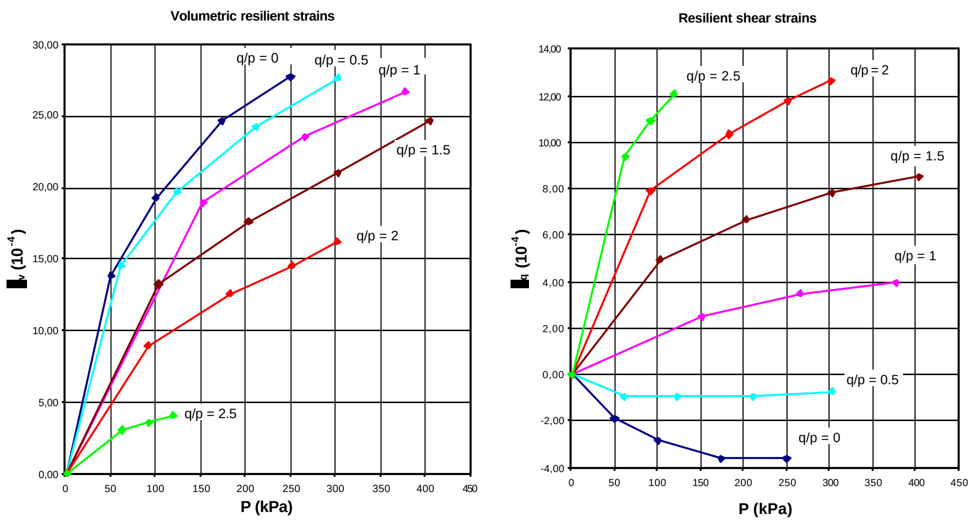


FIGURE 5.1: EXAMPLE OF RESILIENT BEHAVIOUR VARIATION OF ε_V AND ε_Q (ACTUAL MEASURED TEST DATA) WITH MEAN STRESS P AND STRESS RATIO Q/P

Figure 5.2 shows the results of the modelling of these test with the anisotropic Boyce model. It can be seen that the model is able to reproduce the characteristics of the experimental behaviour: non-linear increase of strains with p, the effect of stress path q/p and anisotropy. The quality of the modelling is discussed in Section 6.4.

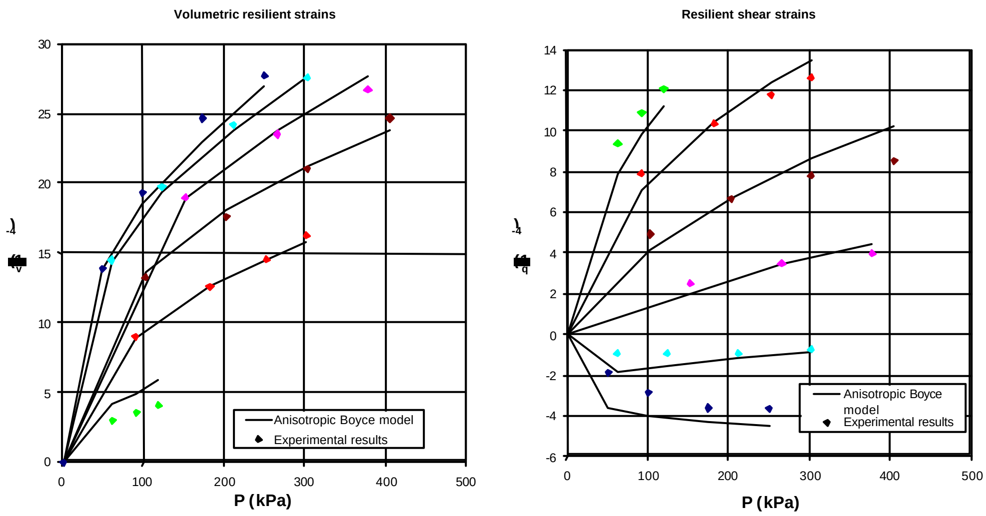


FIGURE 5.2: EXAMPLE OF FIT OF CALCULATED EXPERIMENTAL RESULTS WITH THE ANISOTROPIC BOYCE MODEL

5.2.6 Permanent Strain Model

One model used in COURAGE to model the permanent strain behaviour of the material was that developed at LCPC (Paute, et. al, 1993). The model has the form:

$$e_{1p}*(N) = A_1\left[1-\left(\frac{N}{100}\right)^{-B}\right]$$

with:

A_1	[ ]	"strain rate-type" parameter
B	[ ]	parameter
ε_{1p}*	[ ]	axial permanent strain, with removal of first 100 cycles
N	[ ]	number of cycles

5.2.7 Permanent Strain Rate Model

The other model used in COURAGE to model the permanent strain behaviour of the material was the 'VESYS' model, which has the form:

$$e_{1p}(N) = e_r \frac{m}{a} N^a$$

with:

μ and α	[ ]	parameters
ε_{1p}	[ ]	axial permanent strain, with removal of first 100 cycles
ε_{1r}	[ ]	axial resilient strain
N	[ ]	number of cycles

Taking the derivative of the model, with respect to the number of cycles, yields the rate of permanent strain parameter at a selected cycle number (Mundy, September 1992).

$$\frac{de_{1p}}{dN}(N) = e_r m N^{a-1}$$

with: $\frac{de_{1p}}{dN}$ [%/cycle] strain rate parameter

This model, and the LCPC model, are stress-dependent. In addition, the models both neglect the first 100 cycles of measured strains due to effects of bedding, etc.

6. LABORATORY TEST RESULTS FOR UNBOUND GRANULAR MATERIALS

6.1 Petrographic Descriptions of Materials

All materials were subjected to a petrographic description to identify mineral characteristics which may impact on material behaviour.

Gneiss

The gneiss has an anatomising quartz and mica fabric enclosing porphyroblasts of alkali feldspar. The porphyroblasts range in size from 1 to 5mm and the enclosing quartz grains in the matrix are less than 0.5mm in diameter. There is a well-defined quartz/mica mineral fabric development within the strongly anisotropic rock. Preferential shear could occur along the mica/quartz fabric but the feldspar porphyroblasts and quartz minerals may provide shearing resistance. The resistance to shearing will depend upon the amount and distribution of mica.

The proportion of the constituent minerals is approximately 70% quartz, 15% mica, 10% alkali feldspar and 5% additional minor minerals.

Granite

The rock appears to be of good quality, with the minerals appearing quite fresh. It is very coarse grained, with grains ranging in size from 2 to 12mm in diameter.

The proportion of the constituent minerals is approximately 45% quartz, 45% plagioclase and 10% alkali feldspar (with 75% being plagioclase feldspar and 25% being alkali feldspar), 5% biotite and less than 5% muscovite.

The biotite mineral is prone to weathering to produce iron oxide, but the mineral in the specimen appears quite fresh at this stage. However, this proportion of biotite would not be expected to affect the overall strength of the rock. Some radioactive zircons exist within the biotite, which destroy the crystal structure around the radioactive zircons. This is visible as dark rings surrounding small zircon inclusions.

Limestone

In general, all samples of the limestone consist of 0.5-1mm anhedral and rhombohedral carbonate crystals (possibly dolomite) in a fine-grained carbonate matrix. The fine-grained matrix appears to be associated with biogenic structures

Slide 1:

Evidence of open space filling and recrystalisation associated with biogenic structures. No evidence of straining.

Slide 2:

The carbonate shows no strained crystals. There are two irregular carbonate veins containing some iron oxides. The rock is isotropic; however, the veining may give rise to preferential fracture planes.

Slide 3:

Traces of very coarse-grained carbonate filling biogenic remains are common, these showing open space fill textures. Some cases carbonate crystals appear to have been strained

Recycled Crushed Concrete and Asphalt Plannings

The recycled mix consists of coarse aggregate and sand within a fine cement paste. The coarse aggregate is primarily fresh quartzite with very little internal fracture planes. Some tourmaline exists in one course aggregate particle, possibly up to 10%. The coarse aggregate is a mixture of sub-angular to well-rounded particles to at least 2cm diameter.

The sand fraction in the aggregate is rounded to sub-angular and consists of mainly of quartz with some traces of carbonates. A reasonable level of porosity, of the order of 5 to 7% may be seen as rounded bubbles within the fine aggregate matrix. Brick was identified as being red with a fine grained internal structure. Individual brick particles are coarse-grained material and discrete. A number of fracture planes are visible within the separate brick particles. Some organic material was present and may be asphalt.

The proportion of the constituent minerals is approximately 60% quartz, 35% cement paste and 5% porosity.

None of the grains seem to be touching each other (no mechanical interlock), thus, the strength of the material may be that of the strength of the cement paste itself.

'Flaky Gravel'

The flaky gravel is a natural river gravel from Krapje, Slovenia. It is like a concrete-type aggregate arrangement with a coarse and fine particle matrix. The coarser fraction consists of sub-angular to well rounded river gravel, which makes up to 40 to 50% of the aggregate. Elongated particles are present, but not in large proportions.

The material appears to be metamorphic in origin, with most particles composed of medium to fine-grained quartzite, some with a well-developed mica fabric. Highly strained quartzite, approaching a "ribbon texture", is indicative of high temperatures deformation. The mica and quartz fabrics will create planes of weakness within the individual aggregate particles. One aggregate particle contained a well-developed fabric of calcsilicates (actinolite and clinozoisite). These minerals are more prone to weathering.

One individual quartzite particle with sutured contacts between the quartz grains was observed. Particles of this type will be very strong.

The proportion of the constituent minerals of the coarse aggregate is approximately 95% quartz with less than 5% calcium silicates.

The fine aggregate is predominately quartz and weak muscovite mica, with broken remnants of coarse clasts. The fine matrix is angular to sub-rounded sand, which is well-sorted (similar size range), and mica fragments, which are smaller than the quartz fragments.

The proportion of the constituent minerals of the fine aggregate is approximately 95% quartz, with minor feldspar and 5% mica.

6.2 Simple Tests

Table 6.1 shows the results of the simple laboratory tests performed for the range of materials investigated.

Test	Material				
	Gneiss	Granite	Limestone	RCC&A	Flaky Gravel
General Material Assessment:					
Flakiness Index (Shape) - B.S. (%)	30	7	17	11	13
- CEN (%)	24	3	11	6	9
Plasticity - LL (%)	29	NP	16	34	NP
- PL (%)	20	NP	NP	NP	NP
- PI (%)	9	NP	NP	NP	NP
Compaction - MDD (reference grading) (kg/m^3)	2200	2311	2305	1944	-
- OMC (reference grading) (kg/m^3)	6.0	5.9	4.9	12.0	-
- MDD (fine grading) (kg/m^3)	2220	2320	2355	-	-
- OMC (fine grading) (kg/m^3)	5.5	5.7	5.4	-	-
- MDD (coarse grading) (kg/m^3)	2190	2289	2205	-	-
- OMC (coarse grading) (kg/m^3)	6.2	5.4	4.1	-	-
- MDD (supply grading) (kg/m^3)	-	-	-	2010	2195
- OMC (supply grading) (kg/m^3)	-	-	-	11.0	6.0
Specific Gravity	2.62	2.75	2.71	2.53	2.71
Fragmentation / Abrasion Assessment:					
Los Angeles (LA) (%)	16.0	28.0	19.4	30.3	# 35.3
Modified Bg Index (MBg) (%)	8	9	7	9	7
Dutch Static Compression (DSC) - 5mm (%)	35.2	37.5	43.3	33.7	40.3
- 1.6mm (%)	14.0	18.9	18.5	15.9	19.2
Wet/Dry Strength (10% Fines) - dry condition (kN)	-	-	-	132.6	-
- wet condition (kN)	-	-	-	111.5	-
Micro Deval (MDE) (%)	9.8		12.2	-	-

TABLE 6.1: RESULTS OF SIMPLE TESTS FOR THE SELECTED MATERIALS

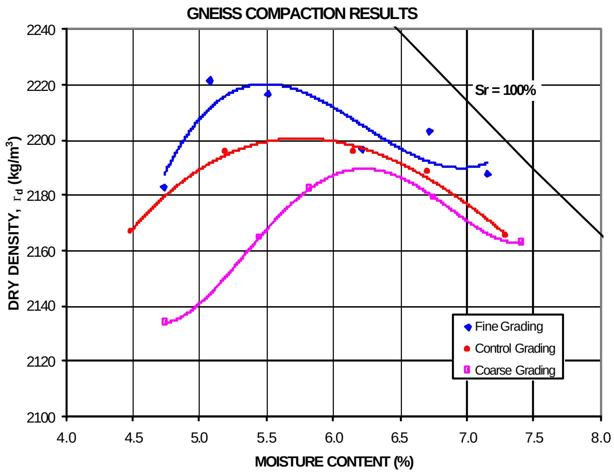
 # There was an insufficient quantity of material sent to Iceland to perform the LA-test on Flaky Slovenian gravel, hence, result shown was for test conducted in Slovenia according to their national standard test procedure, JUS B.B8.045.

The DSC values in Table 6.1 indicated the RCC&A material to be strong in compression, producing the lowest values. This could well be expected of an ex-concrete product. The 'soft' nature of the limestone rock would be expected to contribute to higher plastic deformations in the permanent deformation part of the RLT test.

6.2.1 Compaction and CBR

An increase in density with an increase in fines content was observed for the gneiss, limestone and recycled crushed concrete and asphalt materials (Figures 6.1a, 6.3 and 6.4). It is interesting to observe the completely different behaviour of the gneiss and RCC&A compared with the limestone compaction curves. For the gneiss and RCC&A, the optimum moisture content decreases with an increase in the fines content; contrary to the limestone material where the optimum moisture content increases with increasing fines content. The different behaviour may be attributed to the different mineralogy of particles and the ability of limestone materials to accommodate more water in the fine and slightly cementious material. For the granitic material, which has no plastic fines, the influence of fines content is less marked in the compaction curves; the material for low fines content (3%) is self-drained after a certain moisture content (Figure 6.2a).

The strength, evaluated by CBR, shows the gneiss to decrease with increasing fines content (Figure 6.1b). The plastic fines existing in this material could explain this behaviour; very low values of CBR are observed for the mixture with high fine content. Another important aspect of CBR results is the different sensitivities of the materials to moisture content as a function of fines content. The mixtures with low and medium fines content exhibit the highest sensitivity to moisture content. For granite, the variation of CBR is less significant; a decrease of CBR with increasing moisture content is only observed for the high fines content. These results may indicate why various road authorities use slightly different specifications based on regional knowledge and/or construction practice.



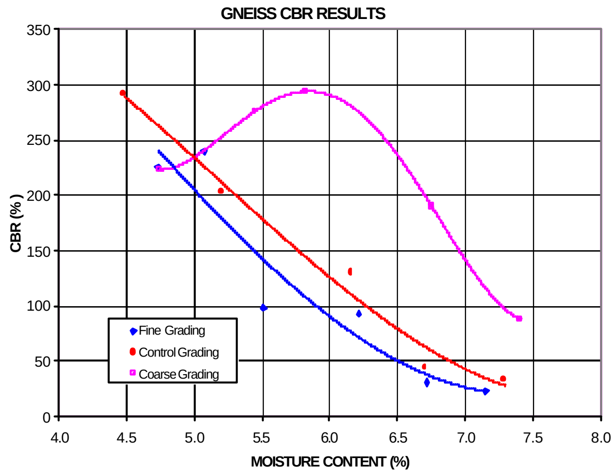
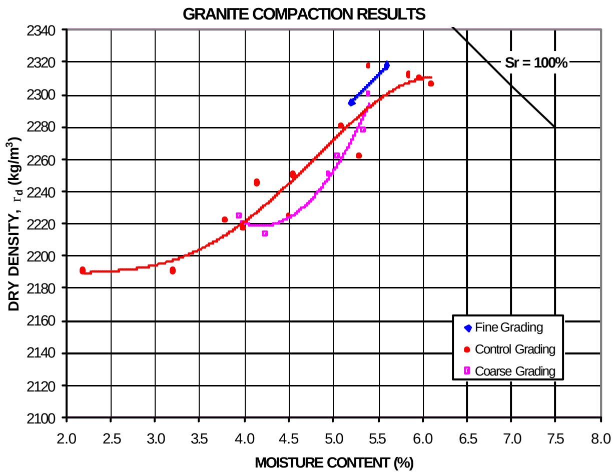


FIGURE 6.1A & B: COMPACTION AND CBR CURVES FOR GNEISS MATERIAL



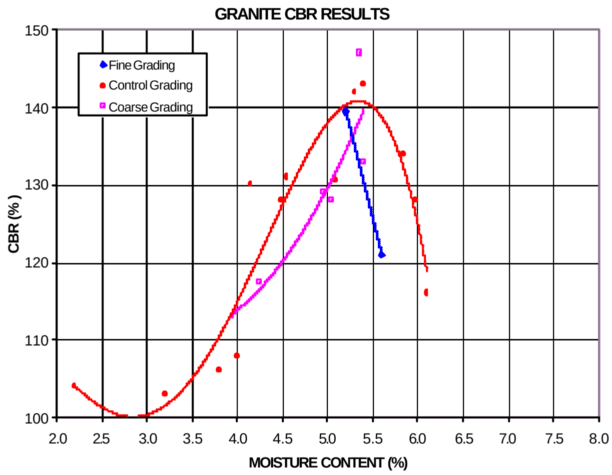


FIGURE 6.2A & B: COMPACTION AND CBR CURVES FOR GRANITE MATERIAL

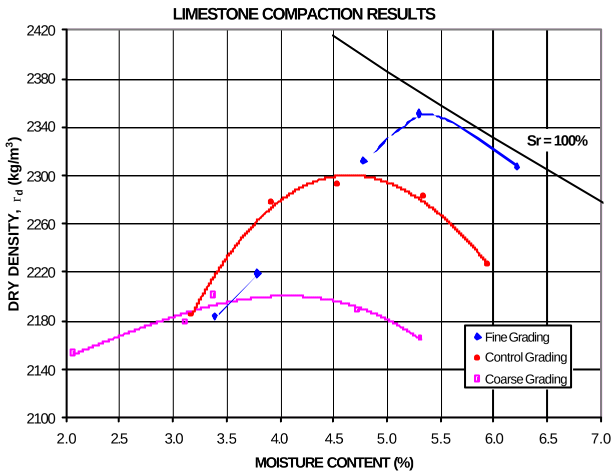


FIGURE 6.3: COMPACTION CURVES FOR LIMESTONE MATERIAL

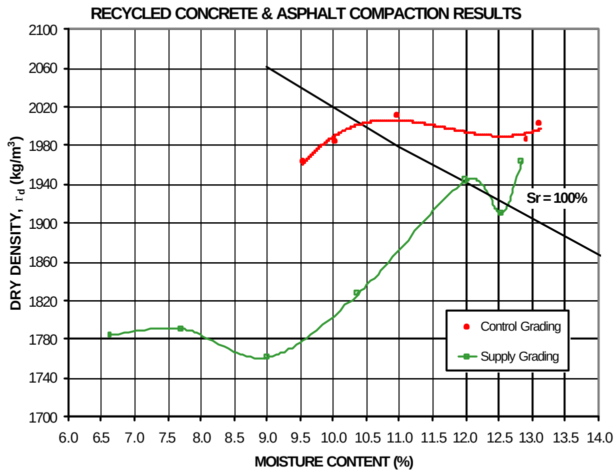


FIGURE 6.4: COMPACTION CURVES FOR RCC&A MATERIAL

6.2.2 Gyratory Compaction

The gyratory compaction test was performed for a range of gyrations, up to a maximum of 300, under a compressive compactive force of 600kPa. This force allowed materials to the compacted to densities comparable to those required in the field. The graphical results (presented in Figures 6.5 to 6.8) for the individual materials illustrate the increase in the percentage of material passing the selected sieve sizes above the original reference grading values.

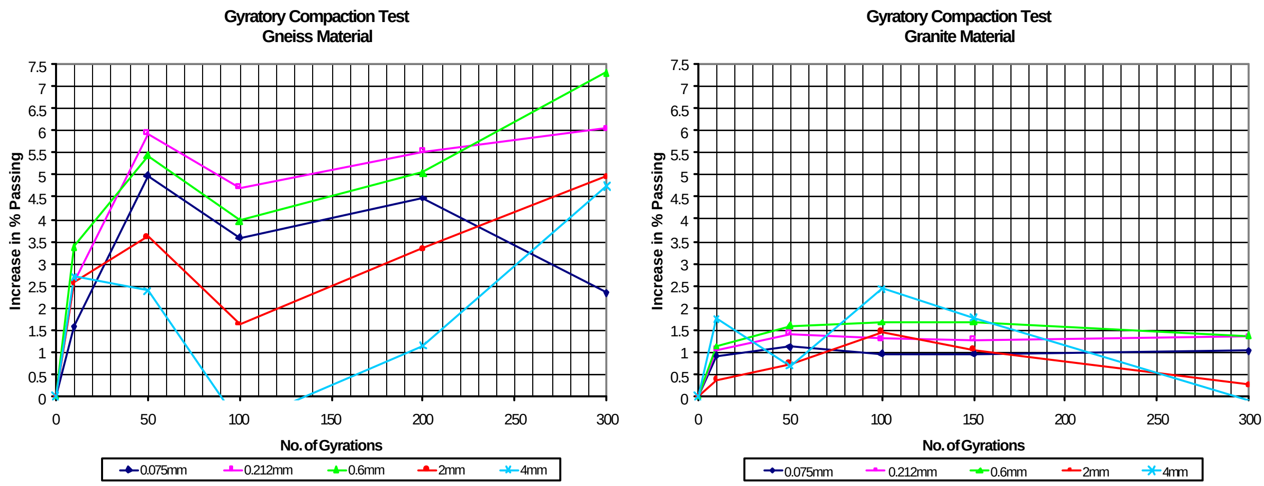


FIGURE 6.5 & 6.6: GYRATORY COMPACTION RESULTS FOR GNEISS AND GRANITE MATERIALS

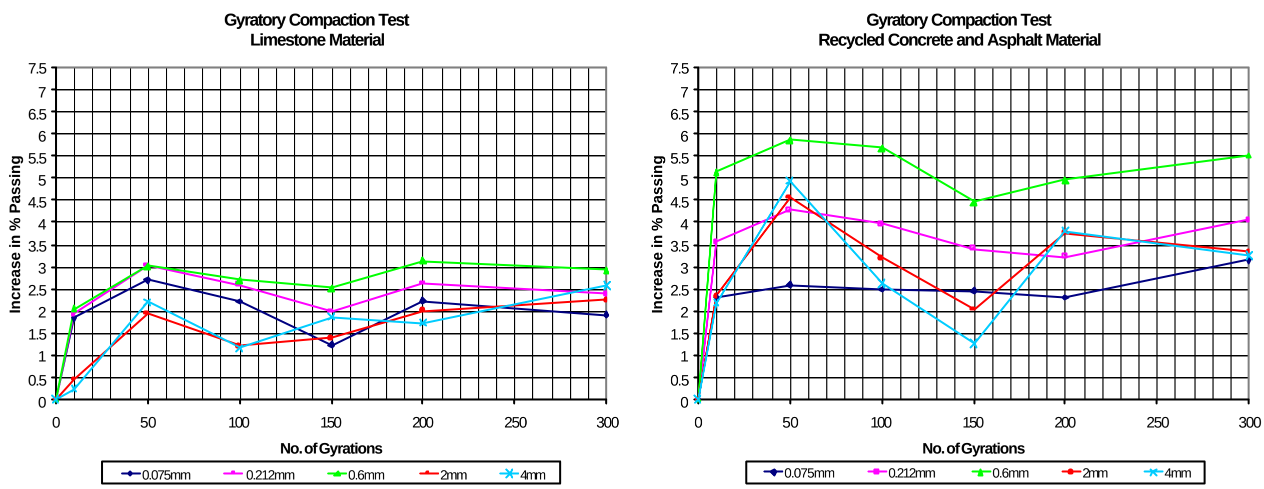


FIGURE 6.7 & 6.8: GYRATORY COMPACTION RESULTS FOR LIMESTONE AND RCC&A MATERIALS

The gyratory compaction test method is presented in Annex 4.3. The results for all materials tested are summarised in a resultant graph below, see Figure 6.9, by averaging the results obtained for either the 150 or 200 gyrations with the 300 gyrations results.

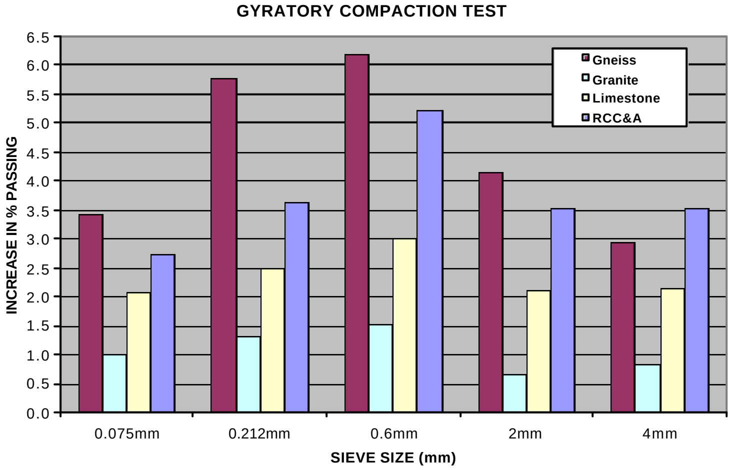


FIGURE 6.9: SUMMARY GYRATORY COMPACTION RESULTS FOR ALL MATERIALS TESTED

This result shows that all the material undergoing the greatest amount of breakdown at the 0.600mm sieve size. The gneiss material was found to produce the highest level of breakdown at all sieve sizes, followed by the recycled crushed concrete and asphalt material, the limestone material and the granite. The gneiss material was quite flaky, particularly at a grain size below 4mm. The material was found to be very difficult to test and consequently achieve very high quality results. This was due to the fact that the fines of this material, passing approximately the 0.600mm sieve, tended to strongly coagulate into small 'powdery' lumps after the washing, and subsequent drying, processes. This action could be closely associated with the high plasticity (PI = 9%) of the fine material determined from testing. The lumps tended to be approximately 4-6mm in diameter, on average. Great care was required to separate the individual grain-size particles from the lumps, avoiding further material breakdown in the process.

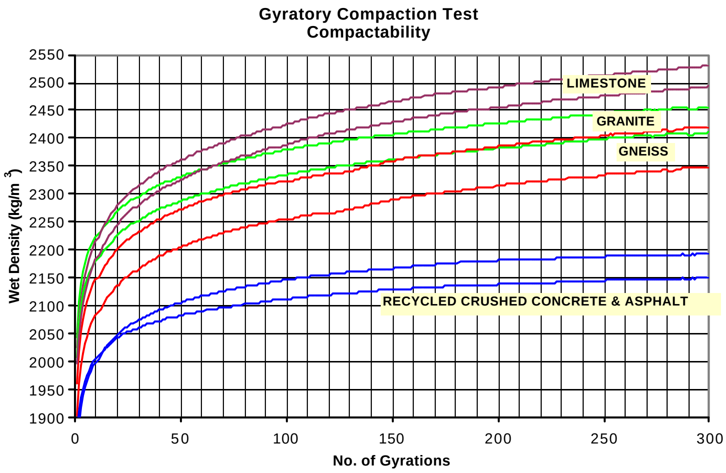


FIGURE 6.10: "COMPACTABILITY CURVES" FOR EACH GYRATORY COMPACTION MATERIAL

The materials tested using the gyratory apparatus were found to yield densities, which lay within the bands illustrated in Figure 7.10, according to their geolgical nature. It should be noted that the gyratory compaction test was found to be quite time-consuming due to the fact that the material grading needed to be carefully manufactured for each sample and, following testing, a wet sieve was required for all materials, prior to dry sieving.

As mentioned the gyratory tests were found to produce the maximum breakdown for all materials at the 0.600mm sieve. If we compare the RCC&A gyratory test result (Figure 6.8) with the pre-test supply grading and post-test grading result of the RLT test specimen (Figure 6.11), the particle breakdown is relatively closely matched; although the RLT specimen was prepared to a slightly coarser grading below the 0.6mm sieve size.

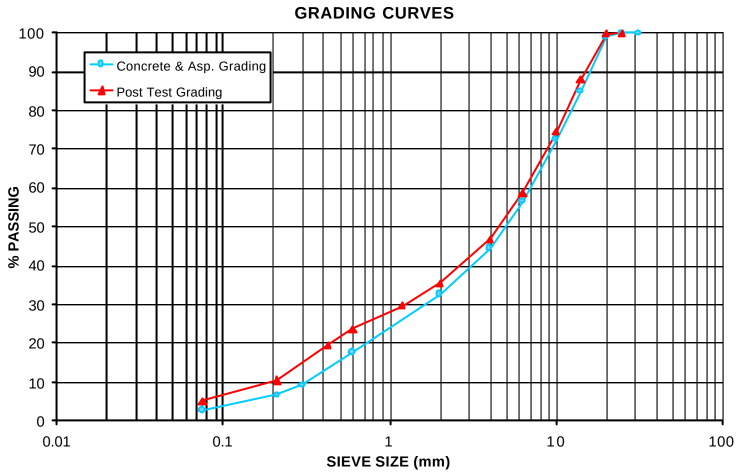


FIGURE 6.11: RLT TEST PRE- AND POST-GRADING CURVE FOR RECYCLED CRUSHED CONCRETE & ASPHALT MATERIAL

6.2.3 Discussion of Simple Tests

The results of the gyratory test would be expected to be somewhat aligned with those of the modified Bg index (MBg) test, given that the prepared sample has the same grading and the nature of the compaction process is somewhat similar. The modified Bg index test did not produce the spread of results that one might expect, with the variation ranging between 7 to 9 units. The granite and recycled crushed concrete and asphalt underwent the highest fragmentation with the limestone and flaky gravel the least. It was surprising that the granitic material in particular, which exhibited low fragmentation in the gyratory compaction test, produced a result as high as it did in the MBg test. However, the result is still moderate when compared to other materials which have been tested using the modified Bg index test such as a fresh, porous Icelandic basalt which yielded a result of 14 (Bjarnason et al, 1999). With reference to this report and the correlations obtained between different test procedures, the following material aspects were apparent:

- the DSC test shows the RCC&A material to be strong against fragmentation, however, in an abrasion test (such as the LA test) the material would be considered unsatisfactory for use as a basecourse material. In the LA test, however, the cement concrete-binding fines tend to be more readily abraded than the parent aggregate, however, this can have a positive effect if the cement fines can hydrate, bind and improve the overall strength and stiffness of the material.
- the LA test shows the Gneiss to perform very well, but due to the limited material size fraction required by the LA test, fails to include the 0.212mm to 2mm fractions which contains a lot of very flaky particles which fragment readily in the Gyratory test.

6.3 Strength Assessment

6.3.1 Static Triaxial Tests

Only two of the COURAGE materials, namely the gneiss and the recycled concrete, have been tested due to limitations in the testing equipment at the other two laboratories. As the recycled concrete was considered to be a more marginal material, a range of test conditions (see Table 6.2) was used to highlight the more limited practical use of such a material in the pavement.

Material and Condition	Reference
Gneiss, DDR=97%, RMC=42%, control grading	A
Gneiss, DDR=97%, RMC=80%, control grading	B
Recycled Concrete & Asphalt, DDR=95%, RMC=88%, supply grading	C
Recycled Concrete & Asphalt, DDR=96.7%, RMC=48%, supply grading	D
Recycled Concrete & Asphalt, DDR=98.6%, RMC=82%, supply grading	E

TABLE 6.2: MATERIALS & CONDITIONS FOR STATIC TRIAXIAL TESTS

The results of the testing are presented in p-q space (as shown in Figure 6.12) and compared with the CEN Method A stress sequence range. It can be seen that the gneiss material, even at a high moisture content, has a shear strength well beyond the stress levels using for CEN Method A RLT testing. Strength-wise the gneiss material is very good indeed. Ref. C material will clearly fail at preconditioning in the RLT test. This was in fact found to be the case so no DDR=95% tests could be performed on this material. In addition, Ref. D material, although possessing sufficient shear strength initially, once this strength is slightly exceeded, was not found to have sufficient cohesion to withstand subsequent stages of the test to expected strength levels. However, the material at these conditions could be expected to withstand the preconditioning load of the CEN VCP test if the first shear strength point is used as a guide, given the "slope" of the other failure lines. Even the strength of Ref. E material is marginal at the q/p=2.5 stress path. Thus, a material of this type is considered only satisfactory as possibly a sub-base or as a capping material. For performance evaluation, the CEN RLT test procedure would need to be reworked to allow for the testing of these more marginal materials at a range of stress conditions, which would be found in-situ, in keeping with the pavement layer of application.

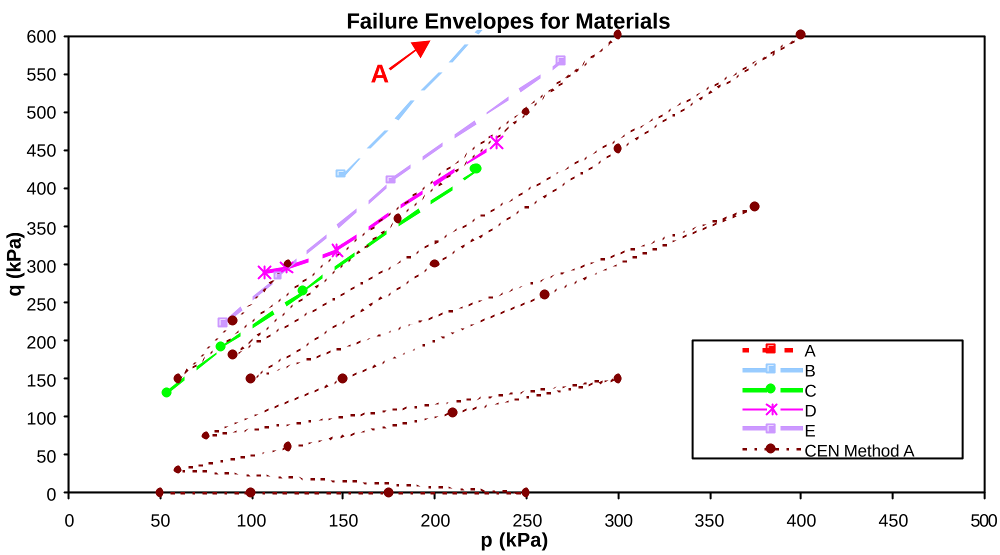


FIGURE 6.12: FAILURE ENVELOPES FOR MATERIALS

6.3.2 *Ultrasonic Wave Velocity Tests*

The results of Ultrasonic Wave Velocity tests on the gneiss material are presented in Figures 6.13 (a) and (b). These results demonstrate that for stiffer specimens, given by a higher resilient modulus, the ultrasonic wave pulse travels more rapidly through the specimen (Figure 6.13 (a)). The density of the specimen does not seem to have a significant bearing on the speed of propagation of the pulse. In addition, Figure 6.13 (b) illustrates that an increase in a material's moisture content slows the speed of transmission of the pulse.

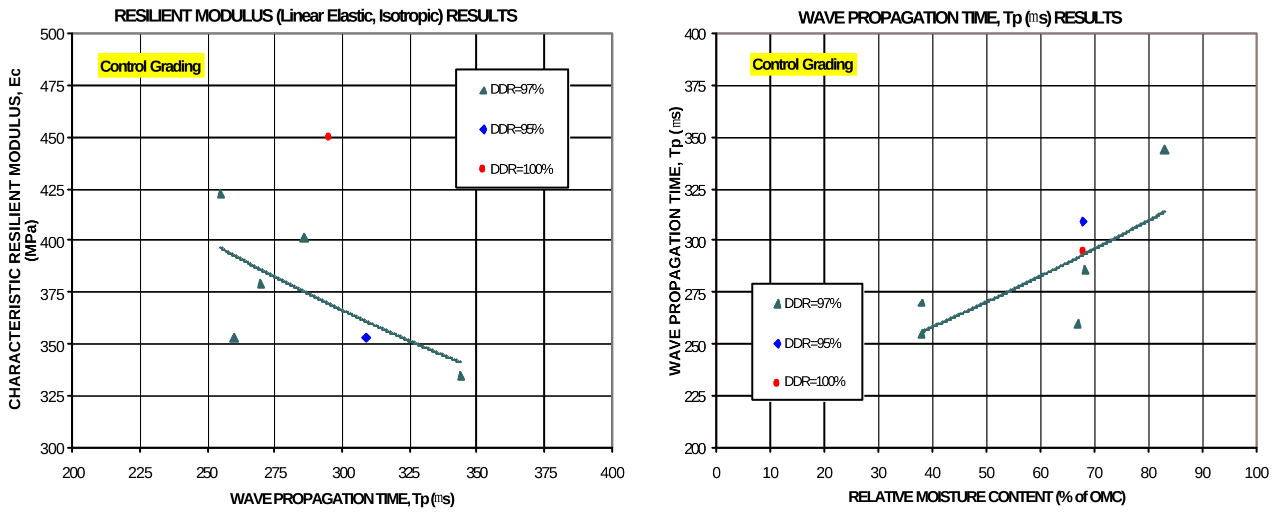


FIGURE 6.13 (A) & (B): ULTRASONIC WAVE PULSE PROPAGATION TIME FOR GNEISS RELATED TO RESILIENT MODULUS AND MATERIAL MOISTURE CONTENT

6.4 Fundamental Tests

The fundamental tests represent more complex tests aimed at replicating field conditions. Two such tests have been used to achieve this, namely:

(i) Repeated Load Triaxial test
(ii) Wheel Tracking test

The results derived from the RLT testing can be separated into two components, namely, those derived from permanent strain analysis and those resulting from the stress stage analysis. The results obtained from testing the COURAGE materials are presented in §6.4.2 and §6.4.3. Prior to RLT testing of the individual materials selected for COURAGE by the different laboratories, RLT round robin testing was performed in order to compare the results obtained from the different laboratories when a selected sample of one material was tested.

6.4.1 Round Robin Tests

The material chosen for the round robin test was the limestone aggregate with the control grading curve (Table 4.3) containing 6.3% of fines (< 0.075mm). The specimen was prepared to a dry density of 97% MDD and a moisture of $w = w_{OPM} - 2\%$ (2 parallel samples). The compaction techniques used by the laboratories were different, with LRPC St. Brieuc using vibro-compression whilst IST and ZAG used a vibrating hammer. The stress paths applied to the specimen during the RLT test conformed to CEN procedure – Method A.

The test results show a very different behaviour of the material related with the permanent deformations obtained during the preconditioning of the material. This difference may be a consequence of the different compaction methods used by LRPC St. Brieuc (vibro-compression) and IST and ZAG (vibrating hammer). However, the resilient behaviour was much closer between the different laboratories. These results were also observed in a RLT robin test realised during another European project “Science” (Galjaard et al., 1996).

6.4.2 Permanent Strain

A typical result for permanent axial and radial strains during the preconditioning test is shown in Figures 6.14 and 6.15. Generally, the plastic strain develops rapidly during the first few applications of loading and the resilient behaviour stabilises as the material approaches a steady state condition. However, for the granitic material with high fines and high moisture content, no steady state condition is observed (see Figures 6.14 and 6.15). As a result of the COURAGE testing, it is clear that some materials can support the 'heavy' preconditioning load applied (σ_1 = 700kPa and σ_3 = 100kPa) and some cannot. Therefore, materials need to be assessed to determine whether they are suitable for use in a pavement as a basecourse, sub-base or capping layer. Only, the higher quality basecourse-type materials will be able to be assessed at the preconditioning and resilient modulus stress stage paths currently presented in the CEN procedure. A separate set of preconditioning and stress stage paths need to be specified for more marginal materials, in keeping with the capability of the material to support such stresses which are aligned with those experienced in the pavement.

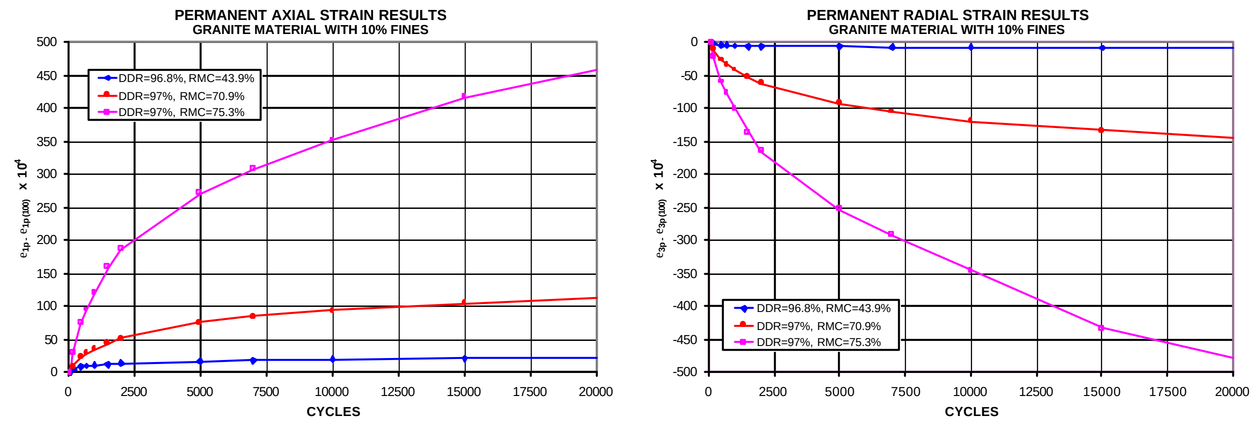


FIGURE 6.14 & 6.15: PERMANENT AXIAL AND RADIAL STRAINS VS LOAD CYCLES

Two forms of modelling analysis were carried out as discussed previously in §5.3.1 and §5.3.2. The results of the high-grade aggregates and RCC&A have been presented in Figures 6.16 and 6.17 for the LCPC strain model (using parameter A_1) and for the strain rate model (using parameter $d\varepsilon_p/dN$), respectively.

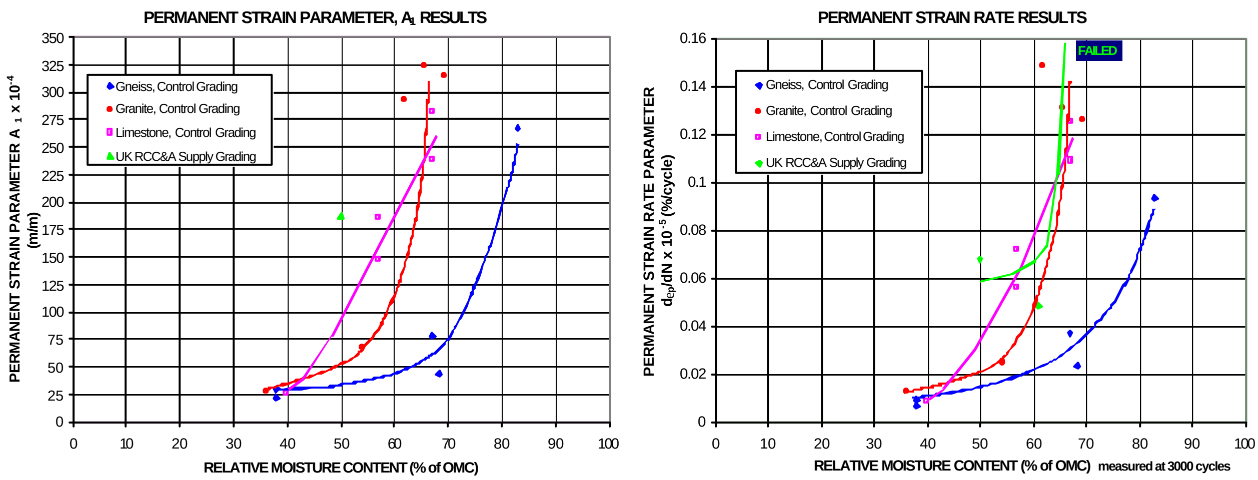


FIGURE 6.16 & 6.17: PERMANENT STRAIN PARAMETER A_1 AND STRAIN RATE $d\varepsilon_p/dN$ VS RMC AT DDR=97%, CONTROL GRADING

N.B. The Recycled Crushed Concrete & Asphalt material was tested at the Supply grading, using both CCP and VCP.

It can be seen that both of these figures illustrate virtually identical strain susceptibility of the four aggregates. All materials undergo rapid strain increases once a moisture content of approximately 60% of optimum is reached. The limestone and RCC&A materials can be seen to undergo higher levels of strain than the other two materials at a condition of lower moisture content. This may well be indicative of the slightly 'softer' nature of the materials. The recycled crushed concrete and asphalt material, possibly along with the granitic material, were completely failing at 67% of OMC. With reference to Figure 6.12, the shear strength of the RCC&A material at this condition is equivalent to the preconditioning stress levels applied to the specimen, *thus the material is unable to support the applied load*. At a moisture content less than 60% of OMC, the material seemed to perform satisfactorily and produced a strain rate, obtained under a both the VCP and CCP tests, comparable with the granite. These results show that for all the materials it is possible to deduce an asymptotic relative moisture content at which the permanent strain rate increases rapidly. The gneiss seems to be the material with the highest asymptotic relative moisture content.

Further, the project also included the assessment of a number of aggregates from Ireland which were described in §4.2.1. The Irish materials assessed for strain susceptibility used the LCPC model (refer to Figure 6.18). It can be seen that the dark argillaceous and carboniferous limestones exhibited relatively high rates of strain, as did the greywacke and crushed granodiorite. The white reef or crushed limestone, ryolite and basalt/dolerite materials performed well at around 50% of their optimum moisture content.

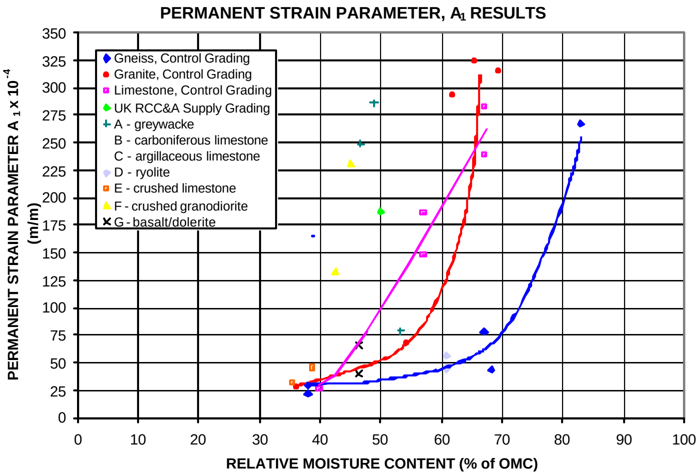


FIGURE 6.18: STRAIN PARAMETER A_1 VS RMC, ALL MATERIALS, DDR=97%, CONTROL GRADING

In addition to the influence of moisture content on the materials, the state of density in known to also significantly affect the rate of permanent strain of an unbound granular pavement material. The two strain model parameters illustrate the effect of density on the permanent strain characteristics of the COURAGE materials (see Figures 6.19 and 6.20). Only, two of the materials were tested at varying density conditions for the same 'reference' level of moisture content. Again, these figures illustrate virtually identical strain susceptibility of the three aggregates with decreasing density. The granitic material undergoes a rapid strain increase with decreasing levels of density.

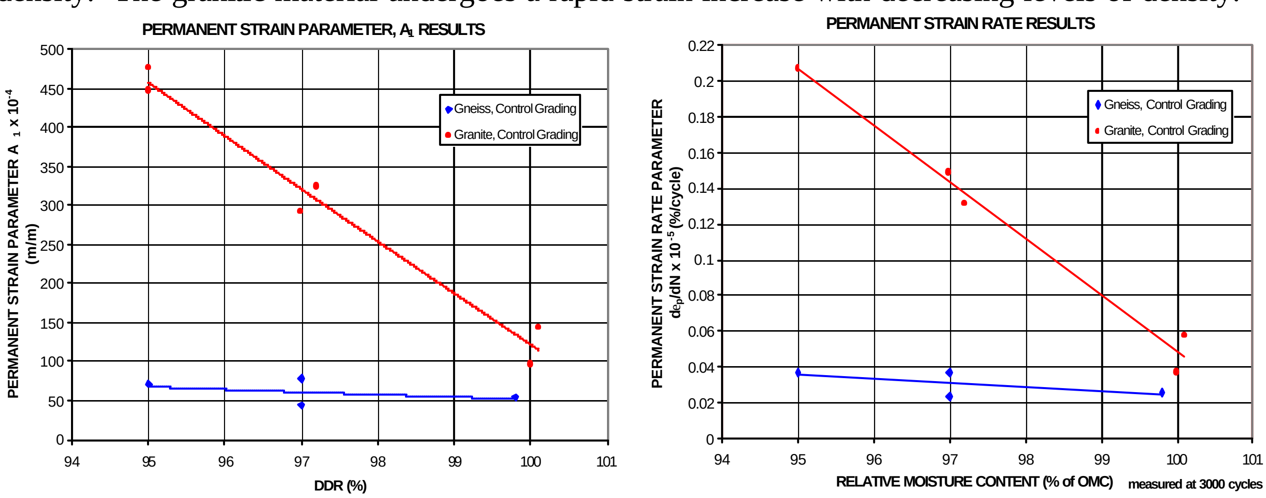


FIGURE 6.19 & 6.20: PERMANENT STRAIN PARAMETER A_1 AND STRAIN RATE $d\varepsilon_p/dN$ VS DDR, RMC=60-65%, CONTROL GRADING

6.4.3 Resilient Modulus

The results of the COURAGE materials have been assessed using the Hooke's Law derived 3-D resilient modulus (using the equation presented in §5.2.1) and the results are illustrated in Figure 6.21. As mentioned in Annex 4.1, the 'characteristic' stress condition used for characterising the materials for comparison was for p = 250kPa and q = 500kPa (σ_1 = 583kPa and σ_3 = 83kPa).

If the materials are somewhat anisotropic in nature, which is influenced by moisture content and density, then the Hooke law is not strictly true. However, this equation is internationally used for presenting the results of variable confining pressure RLT tests. In addition, the isotropic Boyce resilient modulus (with all stress ratios q/p modelled) has been determined at the same stress conditions (as those of the Hooke Law) and also illustrated in Figure 6.21.

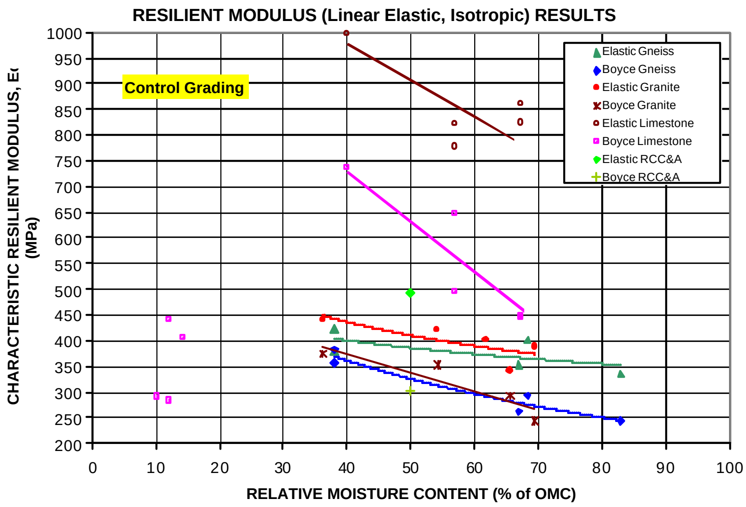


FIGURE 6.21: 'CHARACTERISTIC' RESILIENT MODULUS VS RMC, DDR=97%, CONTROL GRADING

It shall be noted that some differences exist between the Hooke and isotropic Boyce determined moduli. The Boyce moduli are determined in accordance with the theory presented in Annex 5. In essence, the Boyce moduli are calculated using the Boyce predicted (or model-calculated) elastic strains for the 'characteristic' stress condition at which measurement occurs. The Hooke moduli are calculated at the same stress conditions as the Boyce, but use the truly measured elastic strains from the test. A difference in the results between the elastic moduli, determined by Hooke, and the moduli formulation, which is used by Boyce, is noted. In general, the isotropic Boyce-determined values are, on average, 10 to 30% lower than those of the Hooke modulus, however, when using the anisotropic Boyce model to determine moduli, the differences can be greater still.

As can be seen from the permanent strain results, the recycled crushed concrete and asphalt material was failing at 80% of OMC, with still relatively high strain rates and 50 and 60% of OMC. At a moisture content 50% of OMC, the material seemed to perform satisfactorily and the 'characteristic' resilient modulus obtained under a VCP test was quite high around 490MPa. High moduli often result after considered permanent deformation has been induced in the material. This also was the case for the limestone material.

Looking at the aggregates from Ireland, as reported in §4.2.1, their Hooke-determined moduli were compared with those of the other COURAGE materials (refer to Figure 6.22). All of the limestone materials were found to have high 'characteristic' moduli at the stress level used for comparison. This is a common feature of most materials of this geology. The white limestone samples (B and E) would be considered to be softer than most rock types, but are generally recognised as being the most successful of all the rock types when used as an UGM in pavements in Ireland. All of the materials yielded at modulus of at least 450MPa for a moisture content at around 50% of their optimum moisture content.

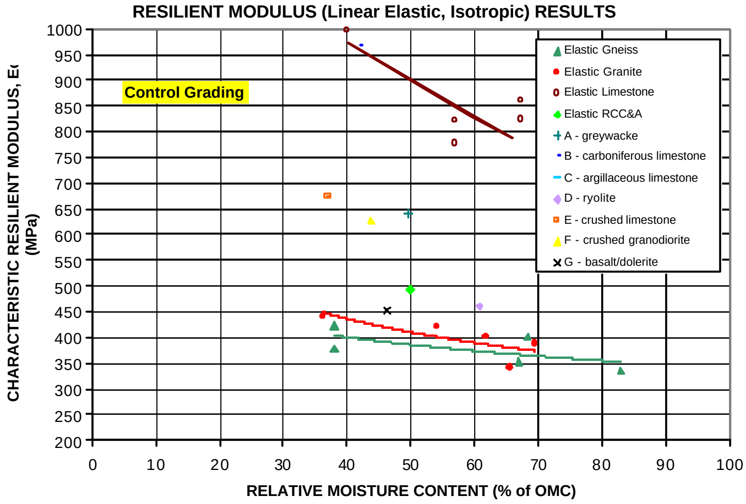


FIGURE 6.22: 'CHARACTERISTIC' RESILIENT MODULUS VS RMC - ALL MATERIALS, DDR=97%

In addition to the influence of moisture content on the materials, the state of density was examined to determine its effect on the performance of the COURAGE materials (see Figure 6.23). Only, two of the materials were tested at varying density conditions for the same 'reference' level of moisture content. The gneiss material undergoes a significant modulus decrease with decreasing levels of density, although the granite does not. The isotropic Boyce model results seem to indicate the opposite trends to the elastic Hooke model.

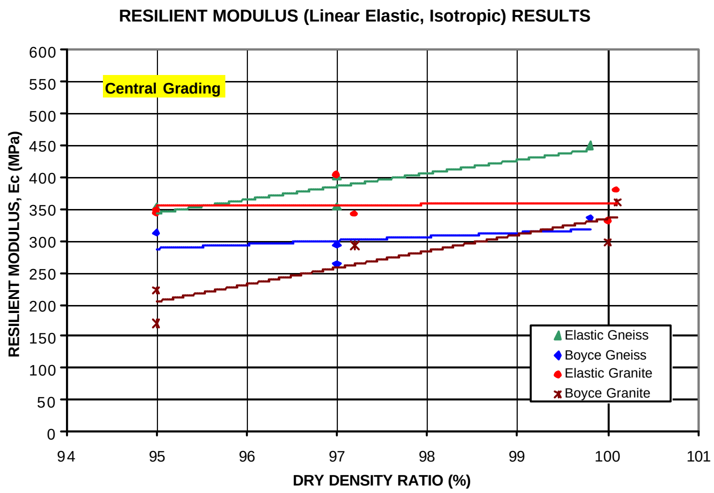


FIGURE 6.23: 'CHARACTERISTIC' RESILIENT MODULUS VS DDR, RMC=60-65%, CONTROL GRADING

The Hooke Law moduli have been plotted in the form of "Resilient Modulus Design Charts" (Mundy, 1992) for the three primary COURAGE aggregates. The charts appear in Annex 6, §A6.2.

6.4.4 Wheel Tracking

As mentioned in §4.3.2, the estimated stress at the top of the granular material layer (with a 30mm asphaltic surfacing) directly under the wheel load is approximately σ_1 = 600kPa. The confining stress determined at this point is approximately σ_3 = 85kPa. This stress combination results in approximately p = 255kPa and q = 510kPa, compared with the RLT test preconditioning load of p = 300kPa and q = 600kPa (σ_1 = 700kPa and σ_3 = 100kPa). The permanent and elastic deformations for the pavement structures tested are tabled below (Tables 6.3 to 6.4).

Loading cycle range	Granite (material A) IST Portugal			Gneiss (material B) LCPC France			Limestone (material C) ZAG Slovenia		
	Part of the base course			part of the base course			part of the base course		
	Whole	upper	lower	whole	upper	lower	whole	upper	lower
0 – 1 000	1.731	1.543	0.188	1.080	0.897	0.182	0.971	0.815	0.156
1 000 – 5 000	1.119	1.080	0.039	0.308	0.250	0.058	0.380	0.344	0.036
5 000 – 10 000	0.506	0.482	0.024	0.207	0.159	0.047	0.197	0.164	0.034
10 000 – 20 000	0.375	0.344	0.031	0.292	0.214	0.078	0.214	0.181	0.033

TABLE 6.3: PERMANENT DEFORMATIONS [MM]

Loading cycle range	Granite (material A) IST Portugal			Gneiss (material B) LCPC France			Limestone (material C) ZAG Slovenia		
	Part of the base course			part of the base course			part of the base course		
	Whole	upper	lower	whole	upper	lower	whole	upper	lower
0 – 1 000	361	209	152	391	229	162	218	149	69
1 000 – 5 000	376	243	133	386	232	154	205	146	59
5 000 – 10 000	374	240	134	389	237	152	212	148	64
10 000 – 20 000	382	229	153	400	241	159	202	146	57

TABLE 6.4: AVERAGE ELASTIC DEFORMATIONS [μM]

With all of the unbound granular materials, most of the basecourse permanent deformations developed in the upper part of the layer. Permanent deformations in the upper half of the layer were in the granite material (A) 97%, gneiss material (B) 73% and limestone material (C) 85% of the whole base course permanent deformations. The permanent deformations that develop during the first 1000 load applications can probably be considered as being *partly* due to the test arrangement, including the thin bituminous layer and the fact that there was no traffic on the surface of the structure during the construction.

Elastic behaviour of the unbound base courses remained quite uniform in spite of the increasing loading cycles (Table 6.4). In general reversible part of deformations appeared to be the least in the limestone material (C) basecourse. In the upper half of the basecourse, the elastic deformations of the granite (A) and gneiss (B) materials observed to be 1.4 – 1.7 times the deformation of the limestone material. It can be supposed that reason why greatest elastic strains appear in the upper half of the basecourse is due to the stress state which is significantly greater there than in the lower half. Therefore it should be most reasonable to pay the biggest attention to this uppermost part of the basecourse. As a result, the permanent and elastic deformations measured in the wheel tracking test over the upper 150mm of the structure have been used to calculate the strain rate, ignoring the first 200 cycles of load. This will allow comparison with the strain rates computed from the RLT preconditioning test.

The results of Figure 6.24 show that the strain rate of the granitic material is markedly higher than that of the gneiss and limestone materials. If the true moisture contents of all the tests are correct, then it could be said that the wheel tracking results are in agreement with those of the RLT tests. Although, the granitic material in the wheel tracking test exhibited a very high rate of strain at a slightly lower moisture content (only 4 to 5% lower) than in the RLT test. It must also be remembered that the applied stress levels in the RLT test were higher, however, the confining pressure in this test is uniform over the length of the specimen tested, as opposed to a reduced confining pressure with depth in the pavement tested by wheel tracking. Ideally, a summation of permanent deformation in the RLT test needs to occur for a number of different specimen elements over the depth of the basecourse layer, with each tested at the calculated stress conditions applicable to each element.

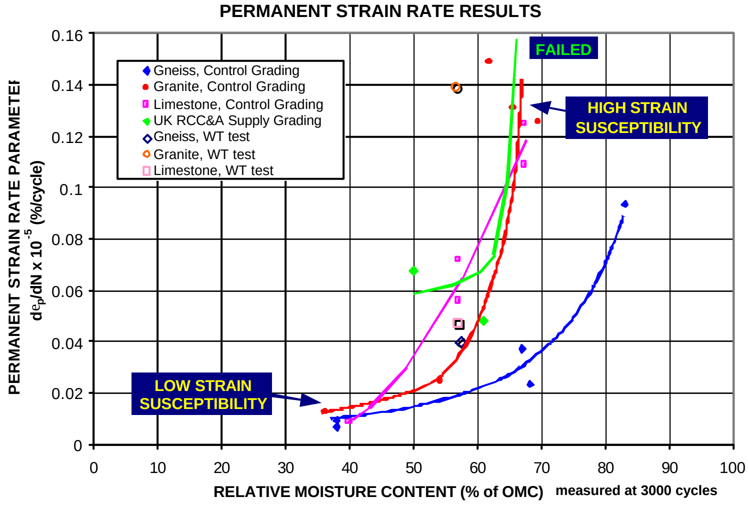


FIGURE 6.24: PERMANENT STRAIN RATE $d\varepsilon_p/dN$ AGAINST RMC FROM RLT & WHEEL TRACKING TESTS

6.4.5 Mechanical Classification

Paute (Paute, et al, 1994) proposed a classification system for unbound granular materials using state conditions representative in the field, which were assumed: $w = w_{OPM} - 2\%$ and DDR = 97%. This system has been enhanced to illustrate the materials' sensitivity to moisture content using the control grading results of RLT tests (see Figure. 6.25). The system defines four classes of material from C1 (high quality) to C3 (marginal) and C4 (unacceptable). A high quality material has a high resilient modulus and a low susceptibility to permanent deformation. As the stiffness of unbound granular materials are very stress dependent, the reference stresses used for resilient modulus characterisation E_c calculations were p = 250kPa and q = 500kPa as above. For permanent deformation, the characteristic value chosen was A_1 as presented in §5.2.6; deducted by fitting the conditioning data for the stress path (p = 300kPa and q = 600kPa).

For the pre-defined conditions, the limestone is the stiffer material ($E_c \cong$ 825MPa), whilst the granite has a stiffness around 425MPa, and gneiss around 375MPa. Stiffness is the mechanical property of an unbound granular material responsible for a material's loading spreading ability. In this respect, the limestone material is expected to have the best performance in a pavement structure.

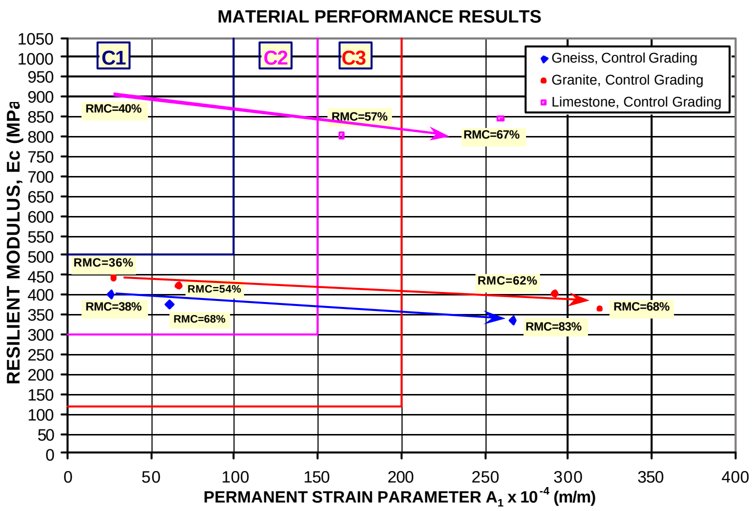
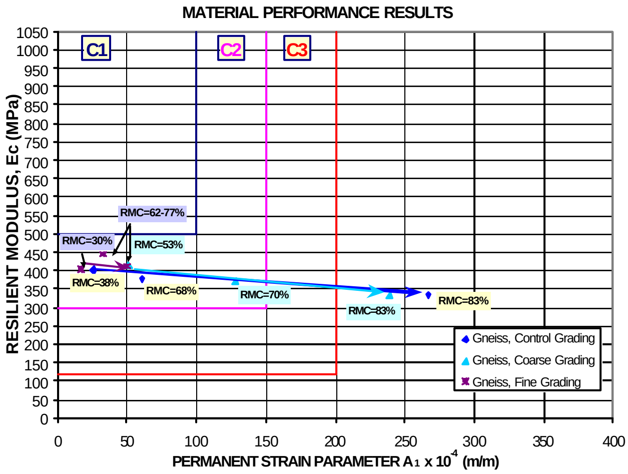


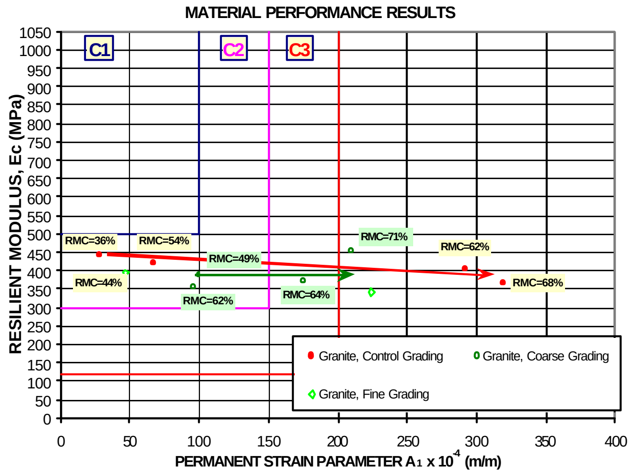
FIGURE 6.25: CLASSIFICATION SYSTEM FOR COURAGE MATERIALS, DDR=97%, CONTROL GRADING

It should be noted that the 'Flaky Gravel' material was unable to survive the preconditioning stage of the test, in fact the material was unable to with stand more than 1700 load cycles. At this stage, the A_1 parameter was determined to be 395 and the resilient modulus was 414MPa, however, remembering that the preconditioning level is higher than that used for characterisation and the material strain response had not stabilised due to the gradual failure.

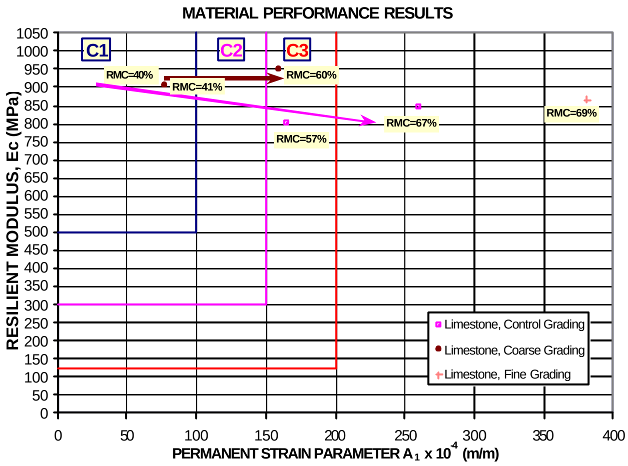
The percentage of fines does not have a significant influence on the characteristic resilient modulus or permanent strain characteristics. However, the effect of moisture content applied to a given material for a particular grading does effect the material performance characteristics as can be seen in Figures 6.26 (a) to (c). The only material not significantly effected in this way is the finely-graded gneiss, which is relatively insensitive to moisture changes.



(a) GNEISS



(b) GRANITE



(c) LIMESTONE

FIGURE 6.26: CLASSIFICATION SYSTEM OF COURAGE MATERIALS, DDR=97%, ALL GRADINGS

Concerning permanent deformation, the relative comparison of materials performance is made slightly difficult because the compaction technique influence the results obtained. From a previous study (Galjaard et al, 1996) it was assumed that the vibro-compression technique provides higher permanent strains during conditioning. Based in this assumption, the gneiss with 10% fines can be considered the material exhibiting the best resistance to permanent deformation. The gneiss and granite with a low fines grading produce a similar behaviour. The results obtained for the granite with 5% fines were the worst.

6.4.6 Overall Material Classification

An attempt at an overall material classification could be made if, as a result of all the simple and functional tests, we aim to place each material into a quality "class". This "class" can be based on established CEN and national standards for the simple tests and also on an 'indicative' ranking of general performance assessment considering the results of the functional tests (refer to Table 6.5). It should be noted that in no way do the expressed rankings relate to traffic volumes, but to material quality in general. Also, the rankings provided for the materials based on the MBg test are based solely on Icelandic experience. In addition, the work carried out in the Icelandic fragmentation studies (Bjarnason et al, 1999) was used to interpret the results of the LA, DSC and MDE tests.

Test	Quality 'Class' Ranking [a]				
	Material				
	Gneiss	Granite	Limestone	RCC&A	Flaky Gravel
General Material Assessment:					
Petrography	2	1		2	
Flakiness Index (Shape)	2	1	1	1	1
Plasticity - LL	2	1	1	3	-
- PI	3	1 [b]	1 [b]	1 [b]	1 [b]
Fragmentation Assessment:					
Los Angeles (LA)	1	1 [c]	1	2	3 [c]
Modified Bg Index (MBg)	1-2	1-2	1	1-2	1
Dutch Static Compression (DSC) - 5mm	2	2	3	2	2
Micro Deval (MDE)	1	-	1	-	-
Gyratory Compaction	2	1	1	2	-
Strength Assessment:					
Static Triaxial	1	-	-	2-3	-
Overall Performance Assessment:					
Permanent Strain [d]	1-2	1-3	1-3	2-3	3
Resilient Modulus (Stiffness) [d]	2	1-2	1	1-2	3

TABLE 6.5: QUALITY "CLASS" CATEGORIES FOR THE COURAGE MATERIALS

NOTE:
a) 1 = basecourse material, 2 = sub-base material, 3 = fill/capping material
b) ranking may be lowered in some countries where specifications do not allow the use of non-plastic materials in the base layer
c) ranking should be lowered for some European countries due to more stringent specification requirements
d) range given due performance over expected in-situ conditions of density and moisture content

Also, the aggregates should be ranked based upon their mineralogy. Although the petrographic descriptions of the COURAGE aggregates were not sufficiently detailed to adequately class the aggregates, the following decisions have been made based upon Icelandic experience.

Granite: The increase of fines <0,063 mm in the test was 2%. The granite would probably be classified as class 1 aggregate even if the MBg result (9) was on the upper limit of the class 1 category. This evaluation was based on the fact that 90% of the material is quartz and feldspars, which usually do not produce harmful fines. The muscovite can cause frost damage, but the amount was less than 5%. The granitic fines are non-plastic.

Gneiss: The increase of fines <0,063 mm in the test was 2%. The gneiss would probably be classified as class 2 material even if the MBg result (8) was lower than the granite result. There was 15% of mica in the material, which was quite high. If this mica was muscovite it can cause frost damage. This classification was supported by the high liquid limit and plastic limits which give an indirect indication that "harmful fines" could be produced when the material is crushed.

RCC&A concrete: The increase of fines <0,063 mm in the test was 1,4 %. The RCA material would probably be classified as a class 2 material, especially since the Bg result (9) was on the upper limit of the class 1 category. The RCC&A had a rather high liquid limit, which could indicate the presence of some unhydrated cement in the fine portion of the material. The petrographic analysis was not detailed enough to support this decision but the presence of brick is detrimental.

The task of assigning a strict "Quality Class" to a material based on the results of all the tests is somewhat complex, given the differing specification requirements and climatic conditions which exist from country to country. However, it is considered that an overall classification would be:

Gneiss - class 2, for reasons of:

- high LL and PI values, making it quite moisture dependent for its performance
- low resilient moduli at expected in-service conditions
- reasonable level of fragmentation at the 0.6mm sieve size under testing leading to the production of more harmful fines, as determined from petrological analysis
- performance in the LCPC experiment with major distress (rutting, surface cracking) after only 400, 000 standard loading cycles (refer to Chapter 7)

Granite - class 2, for reasons of:

- non-plastic nature of material, little apparent cohesion
- very high susceptibility to permanent deformation at mid-range expected in-service conditions

Limestone - class 1, for reasons of:

- very high resilient modulus at mid-range expected in-service conditions
- the non-plastic nature of the material is somewhat overcome by its self-binding properties which develop with time
- however, of concern is the high susceptibility to permanent deformation at mid-range expected in-service conditions. It is expected this would curtail with time as this type of material densifies

RCC&A - class 2 - 3, for reasons of:

- insufficient shear strength at expected in-service conditions
- very high susceptibility to permanent deformation at mid-range expected in-service conditions
- however, is should be noted that many materials of this type, if sourced from higher quality concrete areas, can tend to improve strain and moduli performance with time due to the cementious effects of free, previously unhydrated cement paste. This material did not display these properties.

Flaky Gravel - class 3, for reasons of:

- insufficient shear strength at expected in-service conditions, due to high percentage of rounded particles leaded to poor aggregate matrix interlock
- very high susceptibility to permanent deformation at mid-range expected in-service conditions
- low resilient moduli at expected in-service conditions
- high level of abrasion in the LA test leading to the production of more harmful mica-fines, as determined from petrological analysis

6.5 Modelling Analysis

Extensive modelling of the resilient behaviour of the three principle granular materials studied in COURAGE, the gneiss, the limestone and the granite, was conducted using the anisotropic Boyce and Dresden models (presented in Chapter 5). The objective was to evaluate the ability of the models to describe the resilient behaviour of the materials, and analyse the parameters obtained.

The results of 61 repeated load triaxial tests, performed on the three materials, have been interpreted with the anisotropic Boyce and Dresden models to evaluate the predictions provided by these models. It shall be noted that the non-linear models of Boyce and Dresden cannot be applied to materials where a steady state (permanent strain) condition has not been achieved.

First, the analysis of all the tests showed that the procedures, which involved the use of spreadsheets developed for the determination of the model parameters, worked well. These procedures are now available in all the laboratories participating in the project.

6.5.1 Anisotropic Boyce Model

The modified 'anisotropic' Boyce model, which is a 4-parameter non-linear elastic model, was found to give globally reasonable results for the 3 materials.
Concerning the values of the Boyce model parameters, it was observed that:

- the grading of the limestone material did not significantly affect the model parameters. For the gneiss and granite materials, a significant difference was observed between the finer gradings (7 and 10% fines) with coarser grading (3% fines).
- the moisture content had a significant influence on the model parameters, particularly on parameter G_a. It was also found that the anisotropy increases (γ decreases, and becomes lower than 1) when the moisture content increases.
- the material parameters most sensitive to the moisture content was the limestone, the less sensitive one the granite.
- only the gneiss presented a relatively isotropic behaviour for most conditions; the other materials presented a significant anisotropy, especially at high moisture contents, which confirms the interest of introducing anisotropy in the modelling.

The tests where modelling difficulties were encountered (poor measure of fit, unrealistic values of model parameters) were those performed on very dry specimens (moisture content w_{OPT}-4%) of the gneiss and limestone materials. The main problems were the:

- very small magnitude of the strains measured on these dry, stiff specimens, leading to scatter in the measurements. To avoid this problem, a minimum level of strain under which the measurements are not sufficiently accurate, and cannot be used, should be defined. This should be based upon the maximum error determined over the calibrated range for the strain measuring devices incorporated in the RLT test equipment.
- high values of the parameter γ (values exceeding 1, and up to 1.7) obtained in some of these tests, despite experimental results showing an isotropic response <u>for isotropic loadings</u>. As these values exceeding 1 seem questionable, a possible way of eliminating them would be to impose in the fitting procedure the condition $\gamma \leq 1$, however, this will diminish the level of the fit. Further, investigation needs to be performed to determine whether the parameter γ is truly indicating anisotropy from a material state point of view or whether it is merely an additional parameter which improves model fitting.

6.5.2 Dresden Model

The Dresden model is a non-linear elastic model, expressed in terms of Elastic modulus E and Poisson ratio ν, which has in total 8 parameters. However, the fit is made with 6 parameters, the values of the two others being fixed. This model, contrary to the modified Boyce model, applies to an isotropic material. The levels of fit obtained with the Dresden model were quite satisfactory for the gneiss and the granite, with the measure of fit only slightly lower than those of the modified Boyce model (but with more parameters). The Dresden model did not describe the behaviour of the limestone very well probably due to the higher degree of anisotropy of this material, which the model cannot describe.

6.6 Summary of Key Findings

6.6.1 Summary of Laboratory Testing - Key Findings

The results of the laboratory testing program highlighted the following:

- the CBR results highlighted the different sensitivities of the materials to moisture content as a function of fines content. The mixtures with low and medium fines content exhibited the highest sensitivity to moisture content.
- the MBg index test did not produce the spread of results that one might expect *for the different materials tested within COURAGE*, with the variation ranging between 7 to 9 units. Thus, this test was not found to show the sensitivity necessary to distinguish material quality for the European aggregates assessed. It should be noted, however, that testing of 20 Icelandic basalt materials produced a much greater range (4.3 to 14.1) for evaluation.
- The new gyratory compaction test was found to provide a good means of assessment for the materials examined, however, the procedure was quite labour intensive.
- the simple tests were found to *aid* in material quality assessment, but fail to clearly indicate overall material quality. This can be due to:
    - the specific nature of the test type (eg, fragmentation, weathering or abrasion)
    - the limited portion of the material's parent grading curve used in the simple test
    - the severity of the conditions used in the simple test, with little relationship to real loading conditions
    - the very broad range of specification limits from country-to-country
- static triaxial testing provides a good indication of a material's shear strength, a knowledge of which is required to determine the capacity of the material in the pavement to support the applied traffic induced stresses. This in turn affects the position in which the material may be placed in the pavement structure. The RLT test stress sequences applied to a given material relies heavily on this knowledge.
- the Round-Robin testing of the limestone material indicated that the permanent deformations obtained in the different laboratories exhibited very different material behaviour. This difference may be a consequence of the different compaction methods used by LRPC St. Brieuc (vibro-compression) and IST and ZAG (vibrating hammer).
- some materials could not support the 'heavy' preconditioning load applied (σ_1 = 700kPa and σ_3 = 100kPa), which is more in-keeping with initial heavy compaction traffic loadings. However, these materials may be suitable for use in a pavement as a sub-base or capping layer. At present, only high quality basecourse-type materials are capable of being assessed at the preconditioning and resilient modulus stress stage paths currently specified in the CEN procedure.
- a separate set of preconditioning and stress stage paths need to be specified in the CEN RLT test procedure for more marginal materials, in keeping with the capability of the material to support such stresses, which are aligned with those experienced in the pavement.
- the LCPC permanent strain model (using parameter A_1) and the permanent strain rate model (using parameter $d\varepsilon_p/dN$) illustrated virtually identical strain susceptibility of the

three aggregates examined.

- some differences in the magnitude of resilient moduli results exist between the Hooke and isotropic Boyce determined moduli determined by the appropriate computations. Hooke-derived values are generally 10 to 30% higher. Resilient moduli determined by the 'anisotropic' computations are lower again than those determined by the isotropic Boyce method. These variations require further investigation as they are essential to strain determination computed by pavement design modelling, where resilient modulus is an essential input.
- the wheel tracking tests on a very thinly surfaced unbound granular pavement (using the three COURAGE materials) showed that in the upper half of the basecourse layer:
  - at least 73 to 95% of the total layer permanent deformation resulted
  - the resilient strains were between 1.5 and 2.5 times greater than in the lower half

  due to the higher stress states which exist there.
- the percentage of material fines did not have a significant influence on the characteristic resilient modulus or permanent strain. However, the effect of moisture content applied to a given material for a particular grading does effect the material performance characteristics.

6.6.2 Summary of Modelling - Key Findings

Factors which need to be considered in the modelling analysis are:

- low q/p stress ratios (less than 1.0) should not be used in the modelling of RLT data, given the limits of the strain measuring devices mentioned above.
- the stress ratio of q/p = 2.5 is considered acceptable for modelling purposes *provided* than the mean normal stress does not exceed 125kPa, otherwise significant material shear could result and adversely affect the quality of the modelling results.
- investigation needs to be performed to determine whether the parameter γ in the anisotropic Boyce model is truly indicating anisotropy from a material state point of view or whether it is merely an additional parameter which improves model fit.

7. DESCRIPTION OF EXPERIMENTAL PAVEMENTS

7.1 Introduction

Two full-scale experiments were analysed in COURAGE and used as a basis for the comparisons with the finite element pavement models presented in the next chapter. The two selected experiments were:

- An experiment performed on the LCPC accelerated pavement testing facility, on a low traffic pavement, with an 80mm thick asphalt wearing course and a gneiss granular base.
- An experiment carried out by IST, in Portugal, on an instrumented section of Road IC1, near Porto and referred to as Site PT.1 in Table 3.1. This second experiment involves a much stiffer pavement, with a thickness of 295mm of bituminous materials and a granitic granular sub-base.

Although these experiments were conducted outside of the COURAGE, the exploitation of the results was performed in this project. Both of the granular materials used for this experiment, the gneiss and granite aggregates, were laboratory tested in COURAGE.

7.2 LCPC Full Scale Experiment

7.2.1 The LCPC Fatigue Test Track

The Circular Fatigue Test Track of LCPC, in Nantes (Figure 7.1), allows the testing of 6m wide pavement structures in realistic full scale conditions: real axle load (45 to 75kN), high speed (up to 90 km/hour), and large number of load cycles (up to several million loads). It is an outdoor installation, and it is subject to normal climatic variations. During the experiment, the lateral position of the wheels varies, to simulate accurately the distribution of loads due to real traffic.

Different types of surface measurements (FWD, Deflection, transverse and longitudinal profiles, distress analysis) are also performed during the experiments.

FIGURE 7.1: VIEW OF THE LCPC PAVEMENT TESTING FACILITY

7.2.2 Tested Pavement and Instrumentation

The experiment selected for this project was performed in 1994, to study the fatigue behaviour of four different flexible pavement structures. For the modelling work of COURAGE, only one of the four pavement structures was selected (the one with the most complete instrumentation). This structure is presented in Figure 7.2.

The pavement structure has a length of 28m, a width of 6m and consists of:

- a bituminous wearing course (standard bituminous concrete with 50/70 grade bitumen), with an average thickness of 85mm;
- a granular base (0/20mm crushed gneiss) with an average thickness of 430mm;
- a subgrade with a thickness of 2.5m of micaschist with a low modulus (around 30MPa).

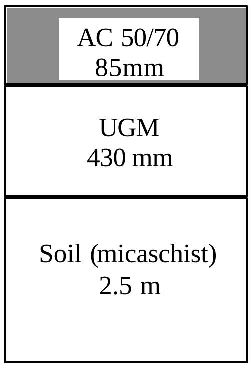


FIGURE 7.2: STRUCTURE OF THE LCPC EXPERIMENTAL PAVEMENT

The instrumentation installed in this experimental structure included:

- strain gauges to measure longitudinal and transversal strains at the base of the asphalt layer,
- a displacement transducer to measure surface deflection,
- displacement transducers to measure the vertical strains in the top 100mm of the granular layer and of the subgrade,
- vertical pressure transducers at the surface of the subgrade,
- thermocouples in the asphalt layer,
- tensiometers, to measure the suction in the granular base and in the soil.

The results of this experiment and associated materials testing are presented in full detail in the reports (Gramsammer et al, 1994 and Hornych et al, 1999).

7.2.3 Pavement Materials

The various materials of the experimental pavement were characterised by laboratory and in-situ tests.

7.2.3.1 Subgrade soil

The soil was a weak micaschist, containing approximately 30% of fines, with a high fines production during compaction. The CBR results indicated that this soil was very sensitive to moisture content, and has a very low CBR at high moisture content: CBR = 10 for w = 10% (average in-situ moisture content, CBR = 2 for w = 14%).

During the construction of the experimental pavement, the upper 500mm of this soil were replaced and recompacted. After the compaction, the bearing capacity of the soil was measured by two methods: dynaplaque tests (dynamic plate loading) and Benkelman beam deflection and the results indicated an average elastic modulus E = 26MPa, with values ranging from 22 to 32MPa.

7.2.3.2 Granular material

The unbound granular material used for this experiment was a 0/20mm crushed gneiss. Two series of repeated load triaxial tests were performed on the material, at a dry density representative of the in-situ conditions, and at several moisture contents; the second series was part of the COURAGE test programme.

During the construction of the pavements, in-situ measurements of density, moisture content and deflection were also performed on the granular base. The results showed a:

- mean moisture content *w* = 5.9%
- mean dry density ρ_d = 2070kg/m^3
- mean deflection : 1.92mm (15 measurements) - standard deviation 0.26mm

It was noted that the moisture content of the granular material decreased during the course of the experiment; the average final moisture content in the UGM at the end of the experiment was only 2.5%.

7.2.3.3 Bituminous concrete

The bituminous material was a standard bituminous concrete with 50/70 grade bitumen; it was subjected to extensive laboratory studies (several types of fatigue tests, complex modulus tests). The behaviour of the bituminous concrete was characterised using the Huet-Sayegh model, which describes the variations of the complex modulus of bituminous mixes with frequency and temperature. This model is presented in detail in the LCPC Full Scale Experiment report (Hornych et al, 1999).

The Huet-Sayegh model was used to calculate values of elastic modulus of the bituminous concrete corresponding to the range of frequencies and temperatures representative of the experiment. These values are given in Table 7.1

	Frequency (Hz)									
T °C)	**0.1**	**0.25**	**0.5**	**1**	**2.5**	**5**	**7.5**	**10**	**12.5**	**15**
6	6269	7977	9333	10725	12598	14025	14859	15449	15906	16278
8	5071	6674	7977	9334	11179	12598	13433	14025	14485	14860
10	4009	5472	6702	8007	9810	11211	12039	12630	13089	13465
12	3099	4393	5525	6759	8499	9871	10689	11274	11729	12103
14	2347	3452	4464	5605	7260	8590	9389	9964	10413	10782

TABLE 7.1: ELASTIC MODULUS (MPA) OF THE BITUMINOUS CONCRETE

7.2.4 Experimental Programme

A total of around 3 million load passes were applied in this experiment. The load applied was the standard French axle load (dual wheel half-axle loaded at 65kN) and the loading speed was 70km/h. The characteristics of this load are shown on Figure 7.3. At the beginning of the experiment, short loading sequences (several hundred cycles) were also applied with:

- different axle loads: 45, 65 and 75kN;
- different speeds: (3 to 70km/h);
- 11 different fixed lateral positions of the wheels (covering a width of 1m), to simulate a realistic load distribution due to traffic.

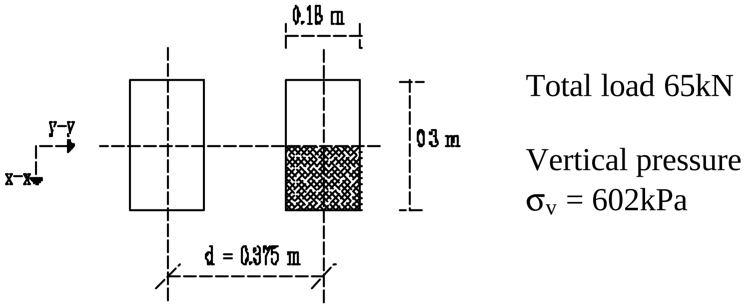


FIGURE 7.3: CHARACTERISTICS OF THE DUAL WHEEL LOAD (65KN LOAD)

7.2.5 Performance of the Pavement

The following parameters were monitored at various stages of the experiment:

- deflection (using the deflectograph), on the completed pavement, at various numbers of load applications,
- transverse profile, to determine the maximum rut depth,
- surface cracking (visual inspection)

The evolution of these parameters with the number of load applications up to the first pavement repair (1,000,500 loadings) is shown in Figures 7.4 to 7.6. The following trends were observed:

- the deflection increased fairly regularly (from 107mm/100 to 179 mm/100 on average) up to 800,000 loadings and then stabilised (see Figure 7.4),
- significant cracking (near 25%) was observed after about 400,000 loadings, and the surface was almost entirely cracked after 800,000 loadings (see Figure 7.5),
- in the evolution of the rut depth 3 different phases were identified: the rapid development of initial permanent deformation during the first loadings (3mm of rut after 5,000 loadings); then a progressive increase of the rut depth up to about 600,000 loadings (average rut depth 16mm), and then a significant acceleration of the rutting towards the end of the life of the pavement once the surface cracking became pronounced. The highest rut depths were observed at the measurement points where the cracking was also the most severe.

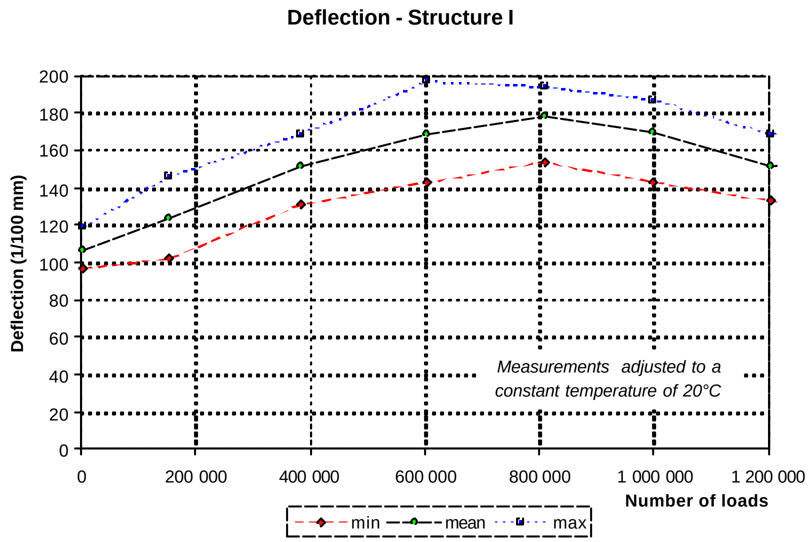


FIGURE 7.4: LCPC EXPERIMENT - EVOLUTION OF DEFLECTION WITH THE NUMBER OF LOADS

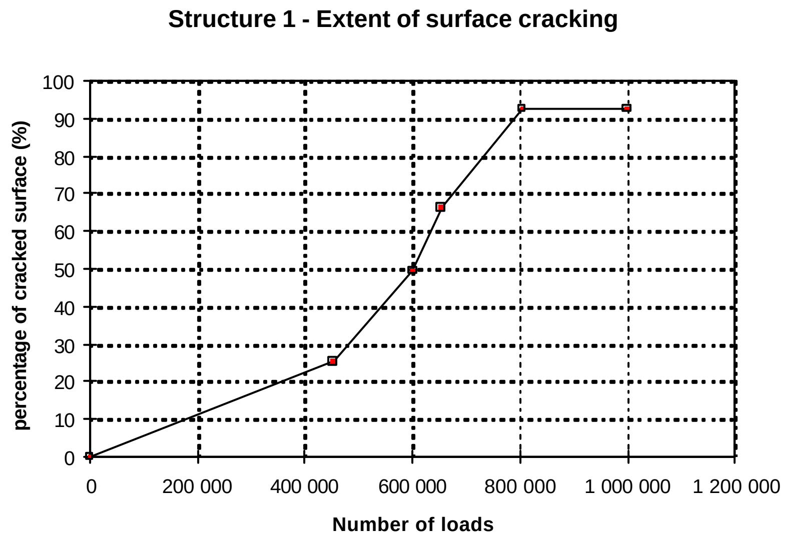


FIGURE 7.5: LCPC EXPERIMENT - EVOLUTION OF SURFACE CRACKING WITH THE NUMBER OF LOADS

7.2.6 Climatic and Moisture Conditions

The climatic and environmental data available for this experiment included:

- measurements of air and pavement temperatures,
- daily rainfall,
- depths of ground water table,
- some suction measurements in the granular layers and in the subgrade soil

The period of main interest was the period of application of the first one million loadings (24 November 1993 to 31 January 1994), where the following observations were made:

- average air temperatures during the testing period were low (in the range of 5°C to 10°C),
- rainfall during the testing period was important, with 120mm of rain in December and 134mm in January. This rain was followed by a fairly dry period with 24mm of rain in November (see Figure 7.6),
- rain caused the water table to rise rapidly in December 1993 from a depth of 1.9m beneath the pavement surface to a depth of 850mm, and remained near that level in January 1994.

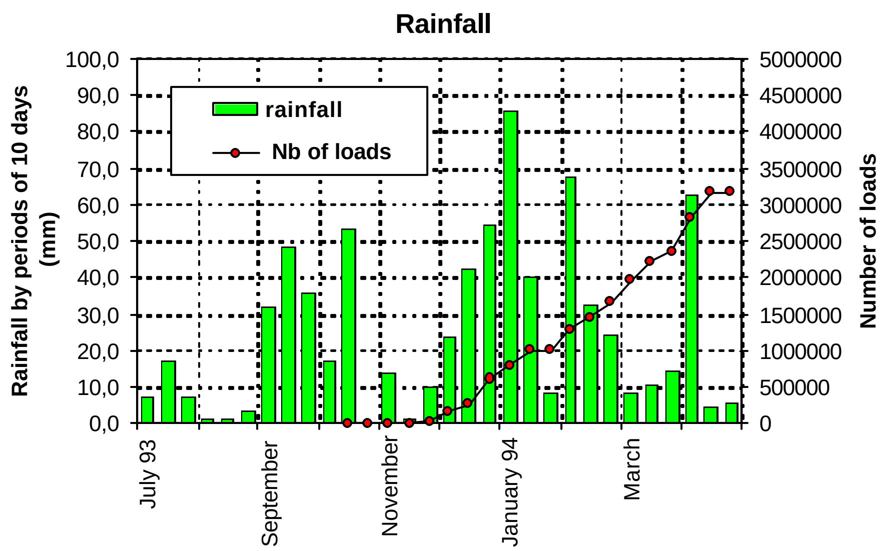


FIGURE 7.6: RAINFALL DURING THE LCPC FULL-SCALE EXPERIMENT

7.2.7 Examples of Data Selected for Modelling

For the purpose of the modelling work, it was necessary to select data from the large database to use for comparison with the models. The following approach was adopted:

1. As the modelling concerns only the resilient behaviour, it was decided to use only measurements made at the beginning of the experiment when the pavement was not damaged (during the first 10,000 load cycles).
2. Only surface deflection and strain measurements (longitudinal and transversal strains at the bottom of the bituminous concrete, vertical strains at the top of the granular layer and the soil) were considered. Vertical pressure measurements, which had scattered results, were not used.
3. Three sets of transducer measurements, corresponding to different loading conditions, were analysed:
   - a series of measurements for **11 different lateral positions of the wheels**, with a constant load of 65kN and a loading speed of 70km/h;
   - a series of measurements for **3 different values of the load (45, 65 and 75kN)**, with the half-axle in the central position, and a loading speed of 70 km/h;
   - a series of measurements for **6 different values of loading speed (3.4, 6.8, 13.6, 27.1, 47.5 and 67.8km/h)** with the half-axle in the central position, and a load of 65kN.

The following notations were used for the measurements:

ε_{xx} longitudinal horizontal strain (direction x), parallel to wheel movement,

ε_{yy} transversal horizontal strain (direction y), perpendicular to wheel movement at the bottom of the bituminous layers,

ε_{zz} - vertical strains (direction z) in the granular layers and subgrade soil,

Stresses and strains are positive when they are in compression.

Detailed analysis of the measurements appears in the COURAGE-WP4 Interim report (Hornych et al, 1999). Only some examples are presented here which concern the first set of data, corresponding to different lateral positions of the wheels (load 65kN, loading speed 70km/h). Figure 7.7 shows maximum values of longitudinal strains, ε_{xx} measured at the bottom of the asphalt layer, for different lateral load positions, varying from y= -0.53 to y= 0.53m (with the gauges in the central position, y = 0). Figure 7.8 shows variations of longitudinal strains, ε_{xx} at the bottom of the bituminous layer with the displacement x of the load, with the load in the central position (y = 0).

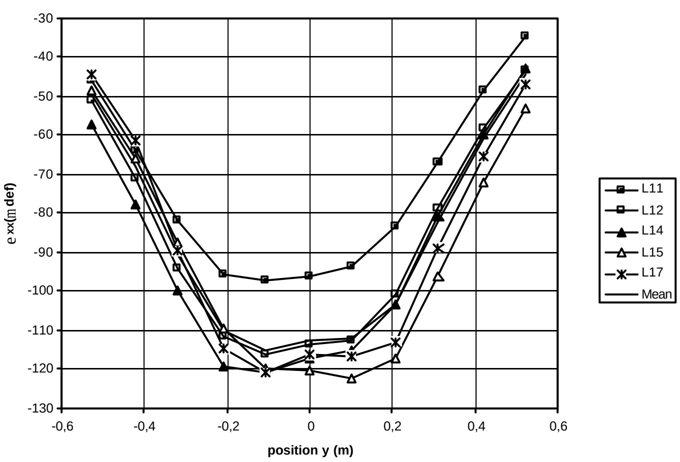


FIGURE 7.7: EXAMPLE OF MEASUREMENTS OF MAXIMUM LONGITUDINAL STRAINS ε_{XX} AT THE BOTTOM OF THE BITUMINOUS CONCRETE FOR DIFFERENT LATERAL POSITIONS OF THE LOAD

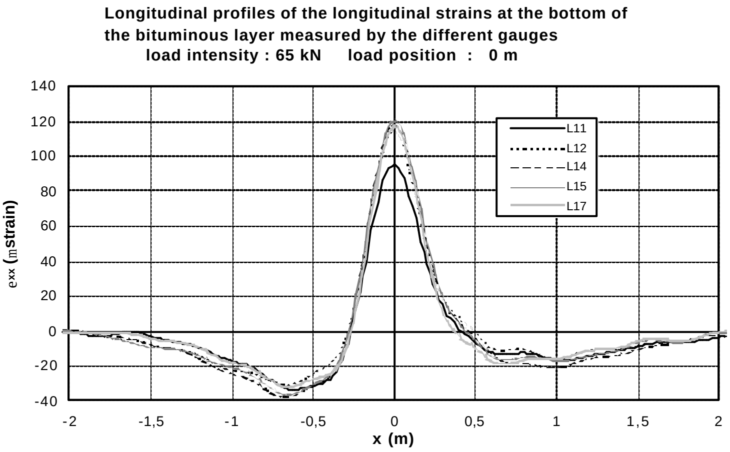


FIGURE 7.8: LONGITUDINAL STRAINS ε_{xx} PROFILE (BOTTOM OF BITUMINOUS LAYER)

Figure 7.9 shows similarly maximum values of vertical strains, ε_{zz} at the top of the granular base, for different lateral positions of the load.

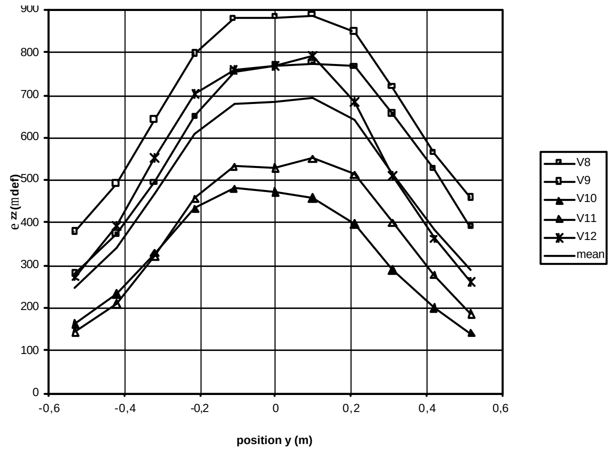


FIGURE 7.9: EXAMPLE OF MEASUREMENTS OF MAXIMUM VERTICAL STRAINS ε_{zz} AT THE BOTTOM OF THE BITUMINOUS CONCRETE FOR DIFFERENT LATERAL POSITIONS OF THE LOAD

7.3 IST Full Scale Experiment

This experimental pavement is a 10m section of a 2x2 lane road. Its structure consists of 295mm of bituminous materials placed on a 500m thick granular base. The pavement was equipped, during its construction, with strain gauges placed at different levels within the pavement structure. This enabled the measurement of strains in the different layers under the effect of a load. In addition, thermocouples were installed in the bituminous layers to measure temperature, and tensiometers and TDR probes in the unbound layers, to measure pore water pressures and moisture contents.

Over a period of almost two years, from February 1997 to November 1998, various tests were performed on this experimental road section (approximately every two months in winter and spring, and every six months in summer) including:

- plate load tests,
- Falling Weight Deflectometer tests,
- truck wheel load tests,

A detailed presentation of this experiment can be found in the report (Salasca, January 1999).

7.3.1 Pavement Structure

The experimental pavement structure is a 10m long section of a 2x2-lane road, called IC1 (Complementary Itinerary) situated north of Porto, Portugal. Three identical pavement sections, spaced 5m apart, were instrumented and successively tested. These three sections will be called Section 1, Section 2 and Section 3, respectively (see Figure 7.11).

The pavement structure consists of 6 layers (see Figure 7.10):

- porous asphalt layer, 40mm thick
- binder layer, 55mm thick
- 2 layers of bitumen macadam, with thicknesses of 95mm and 105mm
- a base layer of unbound granular material, with a mean thickness of 200mm; the 2 granular layers have a thickness of 450mm (Section 1), 535mm (Section 2), and 503mm (Section 3)
- a sub-base layer of unbound granular material, which has a mean thickness of 300mm, and is covered with a leaf of geotextile
- a foundation soil layer, covered with a geotextile

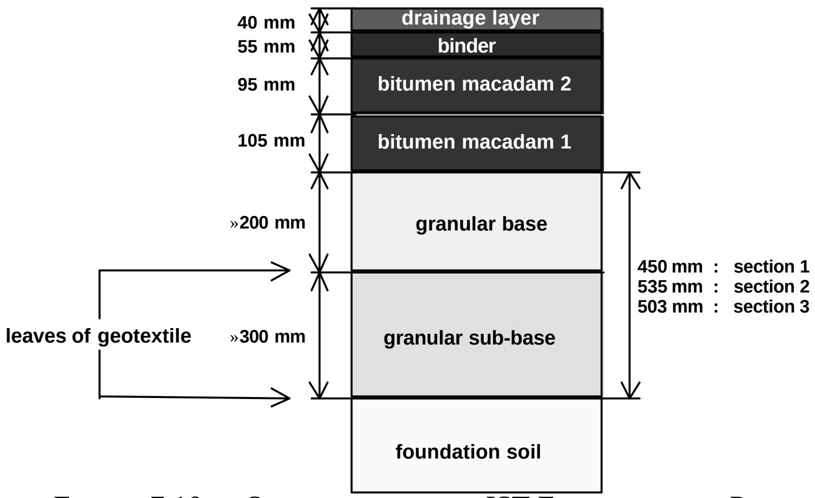


FIGURE 7.10: STRUCTURE OF THE IST EXPERIMENTAL PAVEMENT

7.3.2 Pavement Materials

7.3.2.1 Subgrade soil

The subgrade soil present on the site was a silty-sand, resulting from weathering of granite. The depth of this soil varied between 1.5m and 3m. The in-situ modulus of the soil was determined by plate load tests for three levels of load (vertical pressures of 50, 150, and 250kPa). The resilient deformability modulus, M_r of the soil was close to 30MPa.

Date of Test	Pressure (kPa)	w_r (mm)	M_r (MPa)
05/02/97	50	0.87	32
	150	2.73	30
	250	4.46	31
05/02/97 *	25	0.89	15
	50	1.32	21

*this test has not been rigorously done according to ASTM standard method

TABLE 7.2: PLATE LOAD TESTS ON THE SOIL - RESILIENT DISPLACEMENT AND MODULUS OF SOIL

In-situ density and moisture content measurements were conducted on the top layer of the subgrade soil using in-situ nuclear method (on 5/2/97). The results obtained are presented in Table 7.3. The moisture content of the soil was rather variable, with values between 13% (w-w_{opt}=0.2%) at Section2 to 17.7% (w-w_{opt}=3.9%) at Section 3. The density of the soil was homogeneous.

	Section 1			**Section 2**			**Section 3**		
Test	test 1	test 2	mean	test 1	test 2	mean	test 1	test 2	mean
w (%)	16.2	14.1	15.2	13	14	13.5	17.7	16.7	17.2
r$_d$ (kg/cm^3)	1710	1850	1780	1820	1770	1790	1790	1800	1790
compaction rate (%)	90.2	97.4	93.8	95.7	93.4	94.5	94.3	94.5	94.4

TABLE 7.3: MEASUREMENTS OF MOISTURE CONTENT AND DENSITY AT THE TOP OF THE FOUNDATION SOIL

7.3.2.2 Unbound granular material

The unbound granular material used for the base and sub-base was a 0/20mm crushed granite as tested in the COURAGE program. The laboratory test results obtained for this material are presented in Chapter 7 of this report. Various in-situ tests have been performed on the granular base including plate load tests (according to ASTM methodology) and in-situ measurements of moisture content and density, by a nuclear method. Table 7.4 shows the resilient surface deflections and associated moduli for each load level applied.

Test	Date	12/02/97		13/02/97		14/02/97	
	Place	Section 3		Section 2		Section 3	
Pressure (kPa)		w_{rs} mm	M_{req} MPa	w_{rs} mm	M_{req} MPa	w_{rs} mm	M_{req} MPa
25		0.48	29	0.41	34	0.35	40
50		0.86	32	0.82	34	0.71	39
100		1.41	39	1.56	35		
150		1.96	42	2.11	39		
200				2.49	44		
250				2.92	47		

TABLE 7.4: PLATE LOAD TESTS ON THE GRANULAR BASE - RESILIENT SURFACE DEFLECTIONS AND EQUIVALENT MODULI OF THE GRANULAR LAYERS AND UNDERLYING SOIL

The results of the 3 tests indicate an increase of the modulus of the foundation (granular layers + soil) with the level of stress.

Density and moisture content measurements of the granular base gave the following average values:

- average dry density: ρ_d = 2150kg/m^3 for all sections (95% of modified Proctor optimum)
- moisture content ranges of: w = 1.8 to 2.5% (Section 1), w = 3.1 to 3.4% (Section 2), w = 1.5 to 2.7% (Section 3).

The moisture contents were fairly low in Sections 1 and 3 (with values more than 3% lower than optimum) and about 1% higher in Section 2.

7.3.3 Pavement Instrumentation

During the construction of the pavement, in February 1997, the following transducers were installed in the pavement layers:

- strain gauges (type KYOWA) for the measurement of strains in the different layers,
- thermocouples (type K), for the measurement of temperatures in the bituminous layers,
- TDR probes for the measurement of moisture contents in the soil and granular materials,
- tensiometers for the measurement of negative pore pressures in the soil.

The instrumentation of the experimental pavement is shown in Figure 7.11. In this figure, the x-axis is the longitudinal axis of the road, and each Section is referenced by a vertical axis z_i, i = 1 to 3.

Concerning the strain gauges, whose measurements will be used for the modelling work, 7 have been installed in each of the 3 test Sections:

- one vertical strain gauge is placed at the top of the soil layer, named V1, V4 and V7 in Sections 1, 2 and 3, respectively.
- two vertical strain gauges are situated at the top of the granular base layer, named V2 and V3, V5 and V6, and V8 and V9, in Sections 1, 2 and 3, respectively. Three gauges (V2, V5 and V8) are placed on the vertical axis z_i of each section, the other ones (V3, V6, V9) at a distance of approximately 0.40 m from this axis.
- four horizontal strain gauges are placed at the bottom of the first layer of bitumen macadam; situated on the two sides of the vertical axes z_i, they consist of:
  - ➢ two longitudinal strain gauges, named L11 and L12, L13 and L14, and L16 and L17, in Sections 1, 2 and 3 respectively
  - ➢ two transversal strain gauges, named T11 and T12, T13 and T14, and T16 and T17, in Sections 1, 2 and 3, respectively

From the twelve gauges, four did not survive the construction of the pavement: T11 in Section 1, L13 and L14 in Section 2, and L17 in Section 3. Besides, further tests have shown that gauge L12 does not give correct strain values and must not be taken into consideration.

The following notations were used for the measurements (with stresses and strains being positive in compression):

w - surface deflections (vertical displacements),

ε_{xx} longitudinal strain (direction x), parallel to wheel movement,

ε_{yy} transversal horizontal strains (direction y), perpendicular to wheel movement,

ε_{zz} - vertical strains (direction z) in the granular layers and subgrade soil,

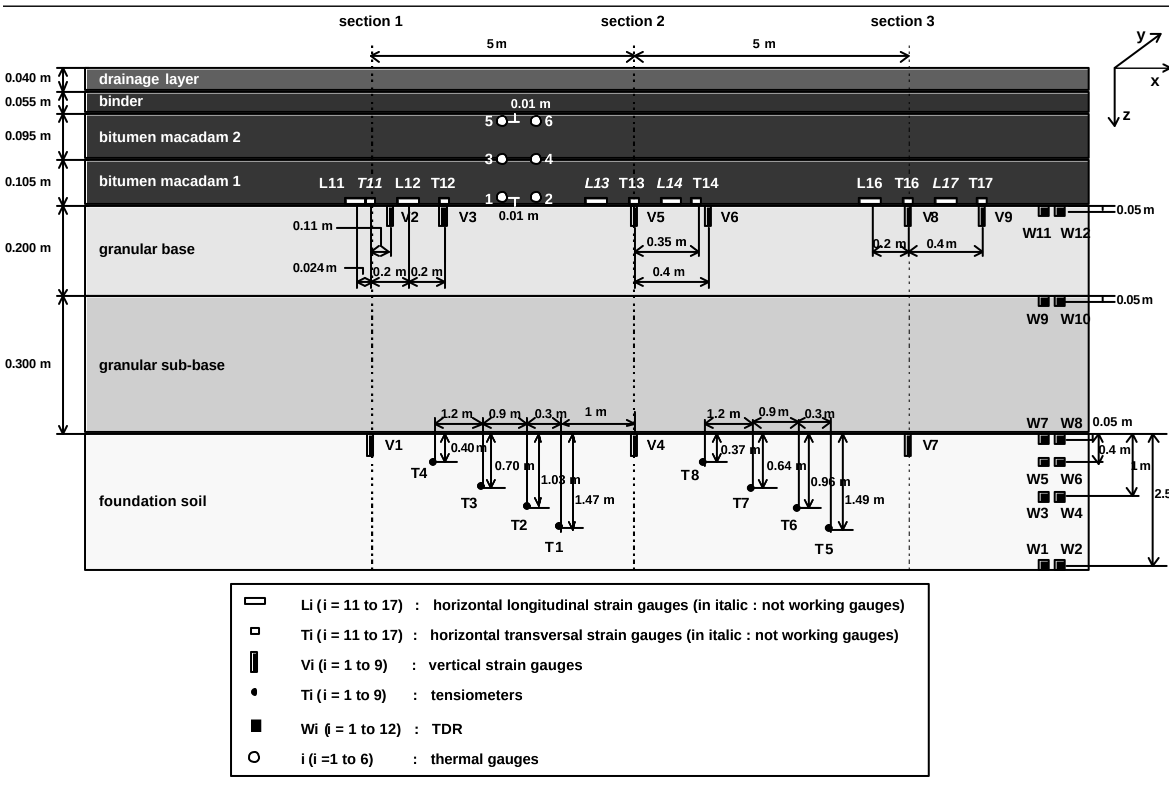


FIGURE 7.11: INSTRUMENTATION OF THE IST EXPERIMENTAL PAVEMENT

7.3.4 Experimental Results Selected for Modelling

A number of in-situ tests were performed on this pavement as reported by Salasca (Salasca, January 1999). The results of some selected tests are presented hereafter.

7.3.4.1 Plate load tests

The results of the plate load tests on the granular base (on 12-14/2/97) show variations of measured strains and surface deflections with applied load (see Figure 7.12). A large difference between the measurements of soil gauges V4 (Section 2) and V7 (Section 3), measured on different dates is noted and also between granular layer gauges V8 and V9 (both Section 3). These differences are probably due to the moisture variations of the soil, which was already noted.

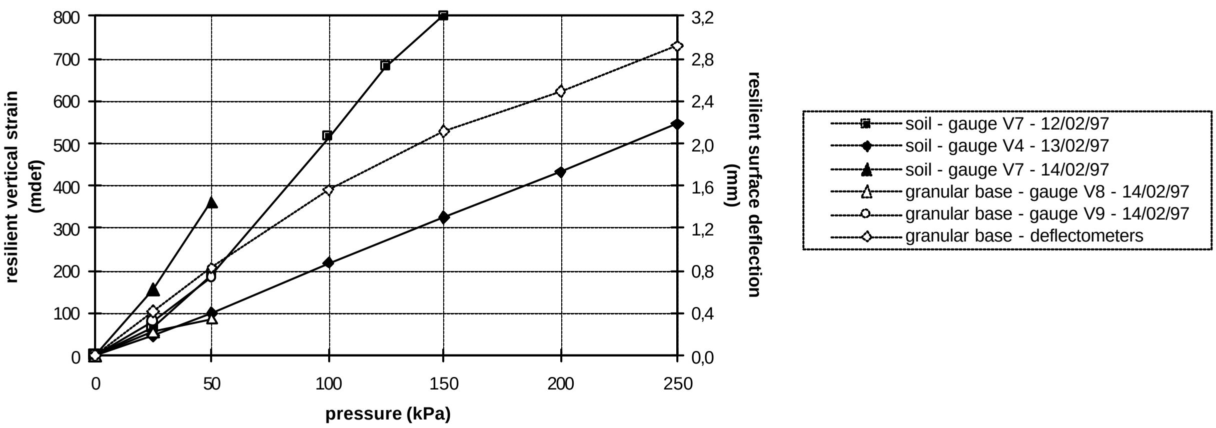


FIGURE 7.12: PLATE LOAD TESTS ON THE GRANULAR BASE - RESPONSE OF SOIL AND GRANULAR BASE

For the soil, the variations of the vertical strain with applied load (see Figure 7.12) indicate that the stiffness of the soil decreases when the applied load increases. For the granular material, the variation of the surface deflection with load level (dotted curve) indicates a non-linear response, but here it is an increase of the stiffness of the granular material with load level which is observed.

7.3.4.2 FWD tests

FWD tests were performed on the pavement wearing course (on 19/10/98), included four load levels (25, 45, 65 and 75kN) on each of the 3 Test Sections. The lowest deflections were obtained in Section 1 and the highest in Section 2, where it was observed that the moisture content of the granular material was the highest. To illustrate these deflection measurements, Figure 7.13 represents the deflections bowls obtained in Section 2 for the four load levels.

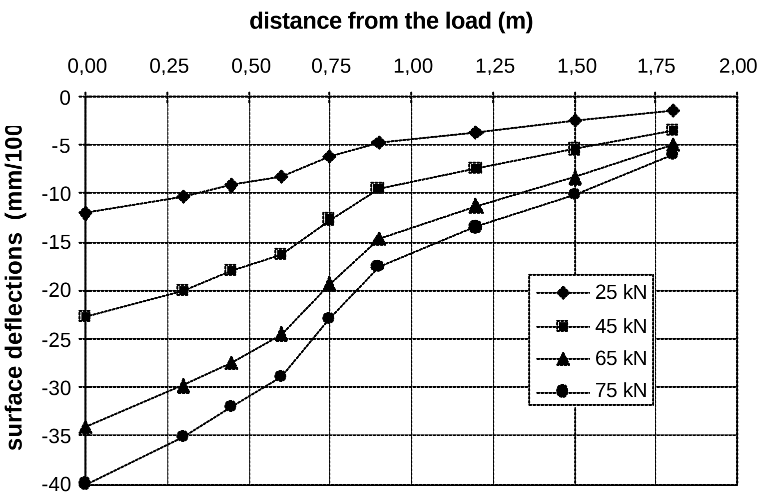


FIGURE 7.13: FWD TESTS ON THE WEARING COURSE (19/10/98) - DEFLECTION BOWLS IN SECTION 2

Figure 7.14 compares the deflection bowls obtained for the three Sections for the load of 65kN. As the values of deflection obtained far from the load are influenced mainly by the behaviour of the soil, it seems that the stiffness of the soil is lower in Section 2 than in the two other sections, where the response is similar. This also means that the difference in deflection between Sections 1 and 3, near the applied load, is due to differences in the stiffness of the upper layers (probably the granular layer, which seems to have a lower modulus in Section 3).

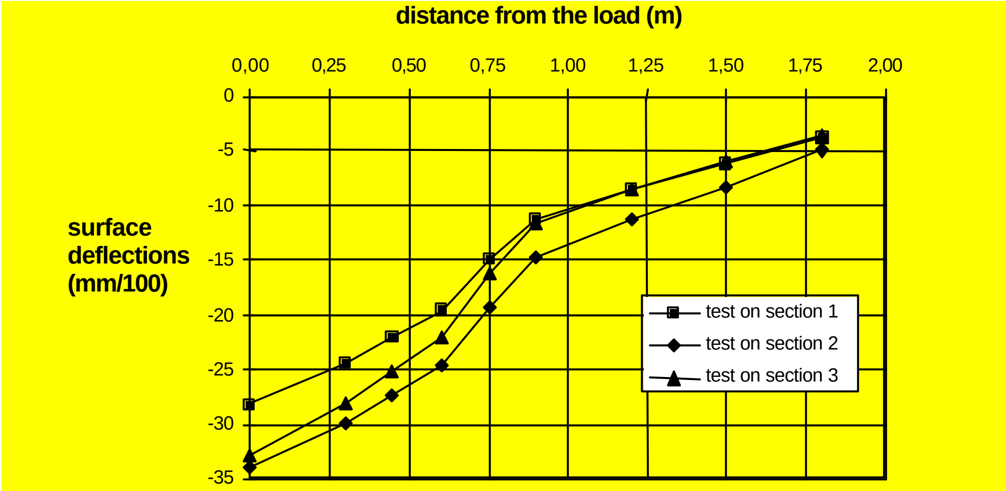


FIGURE 7.14: FWD TESTS ON THE WEARING COURSE (19/10/98) - DEFLECTION BOWLS IN THE 3 SECTIONS - LOAD: 65 KN

Figure 7.15 presents the variations of the maximum deflections under the centre of the load, with the level of the load, for the three Sections. This figure shows that the increase of deflections with load is clearly linear; a result common to all the FWD tests.

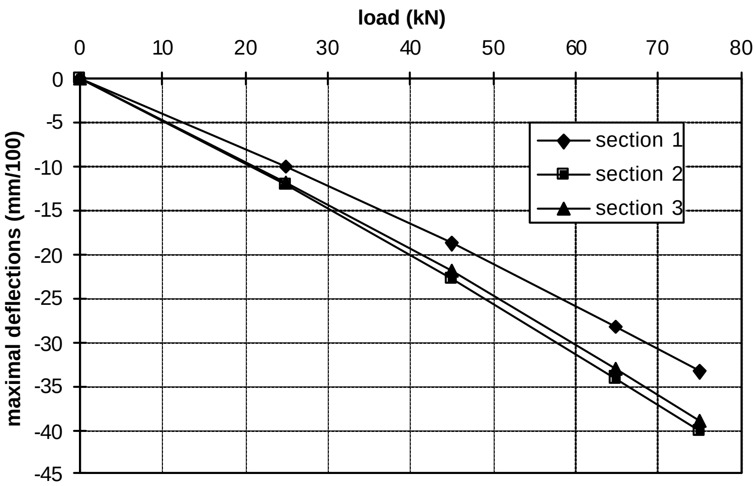


FIGURE 7.15: FWD TESTS ON THE WEARING COURSE - 19/10/98 VARIATION OF MAXIMAL DEFLECTIONS WITH LOAD LEVEL

7.3.4.3 Wheel load tests

The wheel load tests selected for the modelling were performed on the 23/11/98. The tests consisted in moving a heavy vehicle on the instrumented pavement, at several different speeds, and measuring the response of the gauges placed in the pavement under this loading. The geometry of the wheels of the heavy vehicle used for these tests is shown in Figure 7.16. The vehicle has one front axle with two wheels (load on this axle is 6.44kN) and one back axle, with 4 wheels (load on this axle is 12.26kN). The loadings were performed with two twin wheels of the back axle centered on the longitudinal axis where the strain gauges are situated.

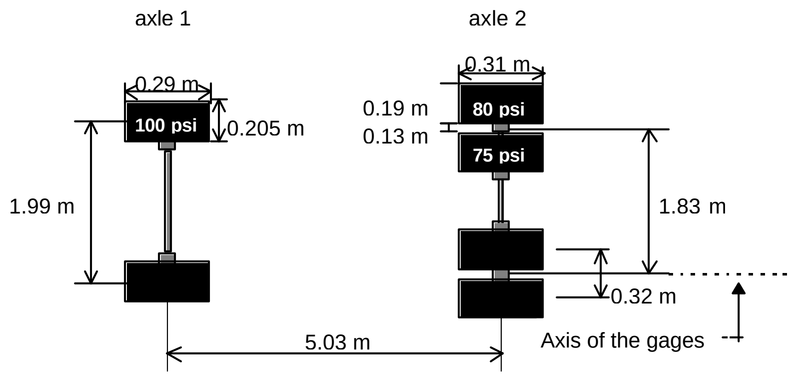


FIGURE 7.16: GEOMETRY OF THE WHEELS OF THE VEHICLE USED FOR THE WHEEL LOAD TEST

7.3.5 Main Conclusions of the IST Experiment

To conclude the presentation of the IST experiment, the following main conclusions can be made concerning the performance of the various materials and the results of the different types of tests.

7.3.5.1 Seasonal variations of deflection

From the FWD tests performed at different times of the year, it is possible to obtain information on the seasonal variations of the pavement deflections and of the stiffness of the various pavement layers. Figure 7.17 presents the evolution of the maximum surface deflections in Test Section 3, for all the FWD tests performed on the wearing course of this Section. The evolution is similar in the two other Sections. It is noted that the surface deflection increases by about 20% between January and April 1998, and decreases again in October and November 1998. This result is in agreement with the Portuguese practice, which considers the spring period as the most unfavourable period, with the highest moisture content in the unbound materials.

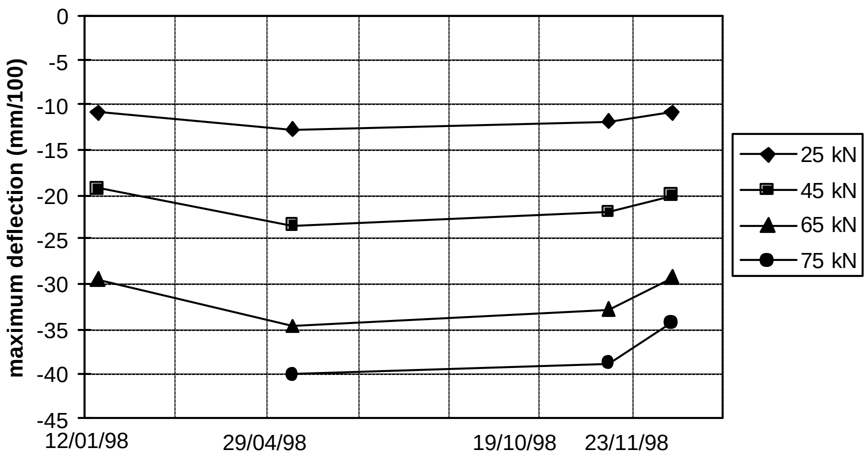


FIGURE 7.17: VARIATIONS OF MAXIMUM SURFACE DEFLECTIONS IN SECTION 3, FOR THE FWD TESTS

The seasonal variations of the vertical strains in the soil follow a rather similar pattern (see Figure 7.18). The strains decrease between June 1997 and January 1998, and then increase, before reaching their highest value in April 1998. After that, the measurements of gauge V7 (the only one still working) show again a decrease of strains over the summer. The amplitude of the variations is of the order of 20% to 30%.

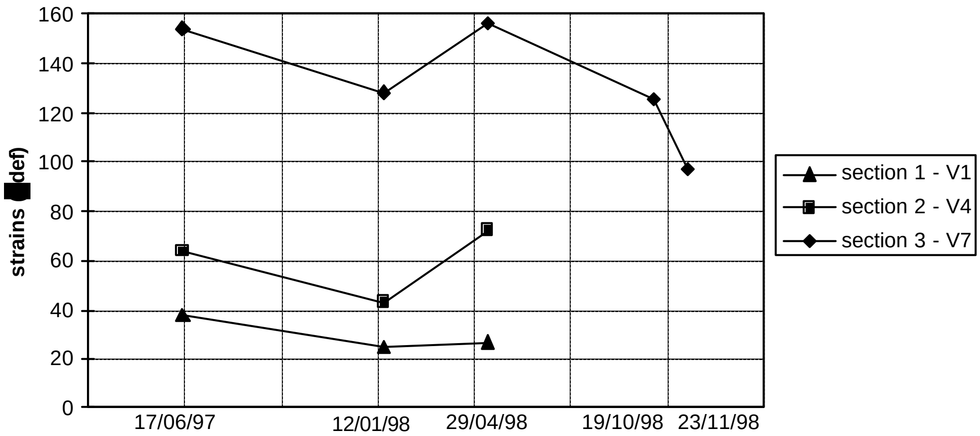


FIGURE 7.18: VARIATIONS OF THE VERTICAL STRAINS ε_{zz} IN THE SOIL MEASURED DURING THE FWD TESTS

The variation of the vertical strains in the granular base show a substantial decrease between June 1997 and January 1998 (see Figure 7.19). This decrease could be due to several effects such as moisture variations and temperature variations (change of stiffness of the bituminous layers), but also due to a densification of the granular layer under traffic, during the first months of the life of the pavement. After that, the values of strain increase again. The amplitude of the variations is important (more than a factor of 2), but the measured values are low (between 10 and 50 µstrains).

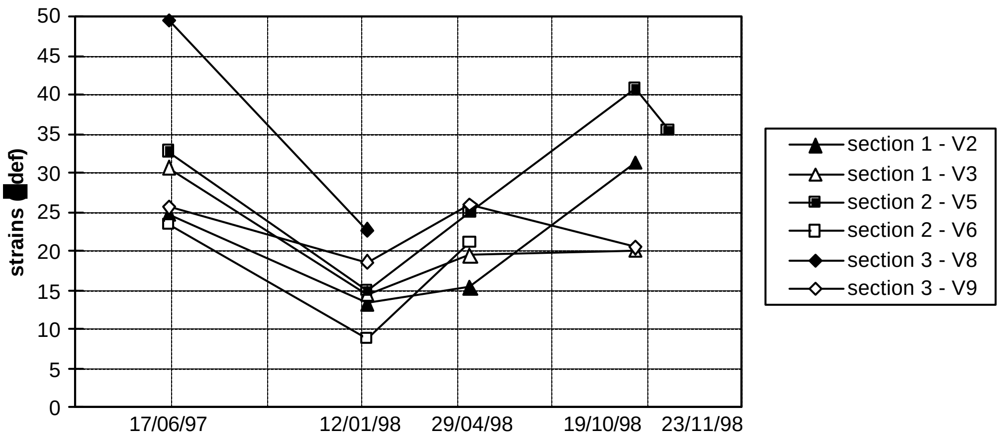


FIGURE 7.19: VARIATIONS OF VERTICAL STRAINS ε_{zz} IN THE UGM BASE MEASURED FOR THE FWD TESTS

7.3.5.2 Effect of loading speed

From the results of the wheel load tests carried out on the 23/11/98 at speeds varying between 0 (static loading) and 40 km/h, it is possible to appreciate the effect of loading speed on the strains in the various pavement layers. The results obtained for Section 1 are presented in Figure 7.20. It can be seen that for this thickness of bituminous pavement (295mm of bituminous materials) that the:

- effect of speed is most significant between 0 and 3km/h, where the measured strains (longitudinal strains at the bottom of the bituminous layers, vertical strains at the top of the granular layer) decrease by more than 50 % ,
- effect of speed is negligible above 15km/h.

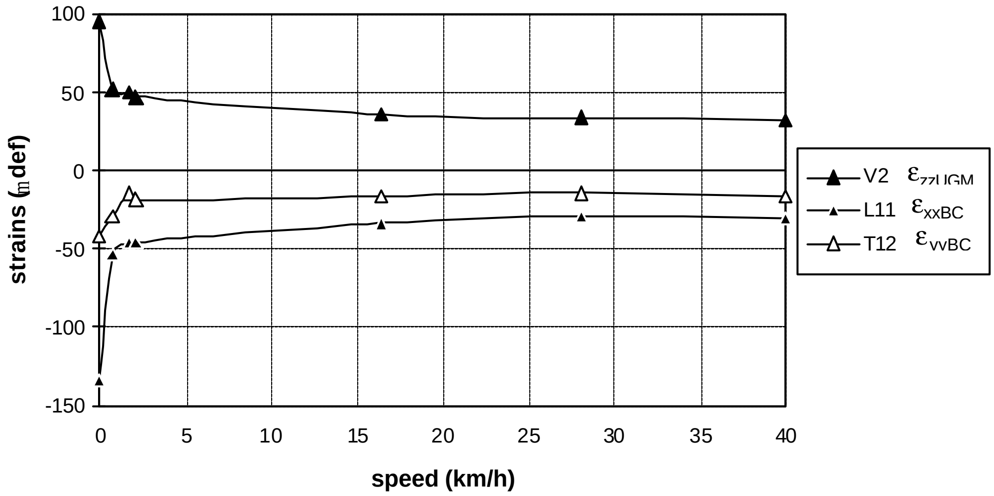


FIGURE 7.20: EFFECT OF LOADING SPEED ON BITUMINOUS CONCRETE AND UGM STRAINS (SECTION 1)

7.3.5.3 Comparison between FWD tests and wheel load tests

As it was observed that the effect of loading speed becomes very small above 15km/h, a comparison of the measured pavement layer strains during the wheel loading (at 40km/h) has been made with the strains obtained under FWD loading. Approximately the same level of load for each test applies (65kN in the FWD tests, 61.3kN in the wheel load tests). Figure 7.21 compares the strains measured in Section 1 for the two types of tests. Even if there are only three points of comparison, the agreement between the strains obtained in the two types of tests is very satisfactory. This suggests that the FWD loading is very similar to the loading produced by a heavy vehicle at a relatively high speed.

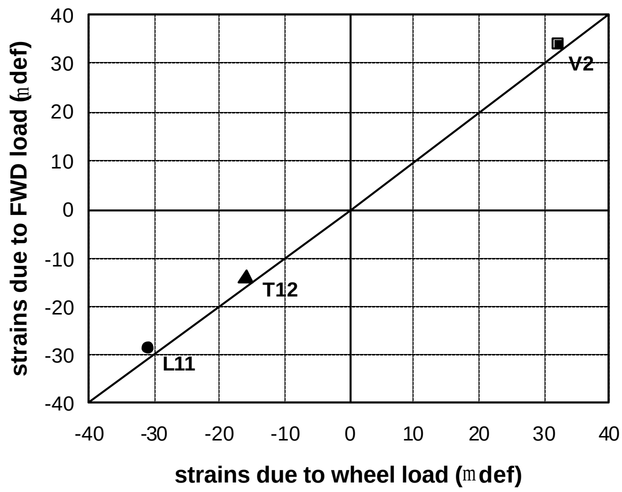


FIGURE 7.21: STRAINS MEASURED BY FWD TESTS AND WHEEL LOAD TESTS AT 40 KM/H

7.4 Comparison of the Two Pavement Experiments

The difference between the response of the two pavements is clearly illustrated in Table 7.5, which compares levels of strain obtained for similar loading conditions. The difference in strain levels is particularly important for the granular materials (more than a factor of 10).

For the LCPC experiment, it is possible to compare the real life of the pavement, observed in the experiment (approximately 800,000 standard axle loads) with design predictions. The French pavement design method, for this type of low traffic pavement, uses the design criteria for the maximum tensile strain ε_t at the bottom of the asphalt and the maximum vertical strain ε_{zz} at the top of the granular layer and at the top of the subgrade.

	LCPC Wheel loading 65kN, 70km/h	IST FWD loading 65kN	IST Wheel loading 65kN, 2km/h
Deflection w (mm/100)	88	32	50
ε_{xx} BC (μstrains)	113	15	30
ε_{ZZ} UGM (μstrains)	684	30 - 40	50
ε_{ZZ} SOIL (μstrains)	600	100 - 120	230

TABLE 7.5: AVERAGE STRAIN LEVELS OBTAINED IN THE TWO FULL SCALE EXPERIMENTS FOR SIMILAR LOADING CONDITIONS

For each of these design criteria, it is possible either to calculate the *maximum permitted strain* for a given design life (number of standard loads), or inversely the number of equivalent 130kN axle loads leading to a given level of strain. Applying the second approach to the mean levels of strain measured experimentally gives the results indicated in Table 7.6. The theoretical life of the pavement is 475,000 standard loads (the lowest value obtained with the three criteria). This agrees quite well with the experimental observations, which showed that the cracking covered 38% of the surface of the pavement after 500,000 loads and virtually 100% of the surface (complete failure) after 800,000 loads.

Design criterion	Mean measured strain (μstrains)	Estimated design life (number of standard 130kN loads)
ε_t BC	113	475,000
ε_{ZZ} UGM	684	1,470,000
ε_{ZZ} SOIL	600	2,650,000

TABLE 7.6: ESTIMATED LIFE OF LCPC PAVEMENT ACCORDING TO THE FRENCH PAVEMENT DESIGN METHOD

7.5 Summary of Key Findings of the Two Experiments

The two studied experiments were different in nature with the LCPC experiment carried out on a fatigue test track, providing well monitored pavement materials through detailed instrumentation and carefully controlled loading conditions (magnitude, speed and position of the load). This type of experiment also allows the pavement to be tested to failure.

The LCPC pavement structure exhibited:

- relatively high strain levels in the unbound granular base and subgrade soil due to an intermediate thickness of bituminous surfacing (85mm),
- a non-linear, load-dependent response - somewhat "limited" due to the thickness of the bituminous layer (the non-linearity is more important with thinner bituminous layers) and to the cold test conditions (average temperatures generally between 5°C and 10°C) which increases the resilient modulus of the bituminous material.
- a strong strain measurement dependence on the loading speed for speeds up to 40km/h.
- a large decrease in the moisture content of the granular base occurred between the end of the construction (w = 5.9% or RMC = 98% of OMC) and the end of the experiment (w = 2.5% or RMC = 42% of OMC, on average). This dramatic difference would result in much increased strain susceptibility and reduced resilient modulus of the UGM (refer to §6.4.2 and §6.4.3).

The IST pavement structure exhibited:

- low levels of strain in the unbound layers resulting from vehicle and FWD tests, despite the heavy traffic due to the rigidity of the structure (295mm of bituminous materials).
- a very linear response of the pavement to different levels of load due to the thickness of the bituminous layer.
- some non-linear response with load when plate load tests were carried out *directly* on the prepared surface of the granular sub-base.
- greater variability of the strain measurements than in the LCPC experiment. This was probably due to the variable bearing capacity of the subgrade soil, and to the small magnitude of the strains measured (thus, greater error).
- seasonal variations of approximately:
    - 50% for the vertical strains in the granular layer
    - 20% to 30% for the deflections and vertical strains in the soil
- moisture content in the unbound granular sub-base layer of between RMC = 27% to 97% of modified Proctor OMC, with measurement ranges applicable to the period of time shortly after construction.
- equilibrium moisture content in the unbound granular layer of between 2 and 3% (RMC = 33 to 50% of modified Proctor OMC).
- a strain measurement dependence on the loading speed for very low speeds up to 15km/h.
- good agreement between strains measured under wheel loading at high speed (40km/h) and FWD loading (65kN load), indicating that the FWD loading is fairly representative of actual traffic loads for this type of structure.

8. MODELLING OF THE EXPERIMENTAL PAVEMENTS

8.1 Introduction

This chapter deals with the modelling of the two full scale experiments with both linear elastic and finite element calculations. The FE approach used non-linear models for the unbound granular materials (Boyce model and Dresden model) in order to evaluate the ability of these models to predict the behaviour of pavements with unbound granular bases. Of the two full scale experiments analysed, the LCPC experiment proved much easier to model than the IST experiment. In this second experiment, carried out on a thick bituminous pavement, the strain measurements in the unbound granular layers led to very low and fairly scattered strain values, which made comparisons between experimental measurements in the granular layer and modelling results rather inconclusive. For this reason, only the results of the modelling of the LCPC experiment are presented in this general report. Some results of the modelling of the IST experiment can be found in the WP4 report "Modelling of the IST full scale experiment" (Hornych and Salasca, May 1999).

The chapter presents successively:

- the finite element models used and the modelling approach;
- the analysis of the results of the LCPC experiment;
- the analysis of the results of the IST experiment;
- conclusions and recommendations about the models used.

8.1.1 Modelling with CESAR-LCPC (Boyce model)

The finite element program used for this study is the program CESAR - LCPC. It is a general, multi-purpose finite element code, used for various applications (2D or 3D). Two modules of CESAR have been used for this work:

- the module LINE used for linear elastic calculations;
- the module CVCR, used for modelling of pavements under moving wheel loads. This module includes the following models of material behaviour:
    - linear elasticity;
    - the Boyce model (isotropic or anisotropic) for unbound granular materials;
    - the Huet-Sayegh visco-elastic model for bituminous materials.

General hypotheses

The following hypotheses have been used for all the finite element calculations.

- Most calculations have been performed in 3D. This is important in particular to model accurately the geometry of the load (65kN half-axle with twin wheels in most cases, see Figure 7.3), which has an important effect on the results and particularly on the asphalt strains.
- The finite element mesh used for all the calculations where the bituminous concrete was considered linear elastic describes ¼ of the pavement structure (due to the symmetry of the problem) and consists of hexahedral elements with 20 nodes. From previous experience, it is known that this type of 3D mesh produces results of good accuracy.
- For the calculations with the visco-elastic model, in which the movement of the load is taken into account, a larger mesh, describing ½ of the structure, was used (in this case, the longitudinal symmetry is lost).

8.1.2 Modelling with ANSYS (Dresden model)

The second finite element program used is the program ANSYS, used at the University of Hanover. It is also a general multi-purpose finite element code (2D and 3D). For the COURAGE calculations, the models used were the Dresden model and linear elasticity.

A module for the non-linear calculations with the Dresden-model was implemented in ANSYS. Because ANSYS is a commercial program, the University of Hannover was not able to develop this module alone. However, difficulties were encountered with this implementation. The 2D-calculations were working well, but in 3D, there were often convergence problems and also unexpected interruptions of the program. Many tests were performed to find the cause of the problem (hardware-components, computer memory, meshing of the systems, boundary conditions, loads, element type, nodes and so on), but nothing was found. Due to these numerical problems, only a limited number of calculations where the convergence was acceptable could be performed with ANSYS; they concerned only the LCPC experiment.

The following hypotheses have been used in all the ANSYS calculations:

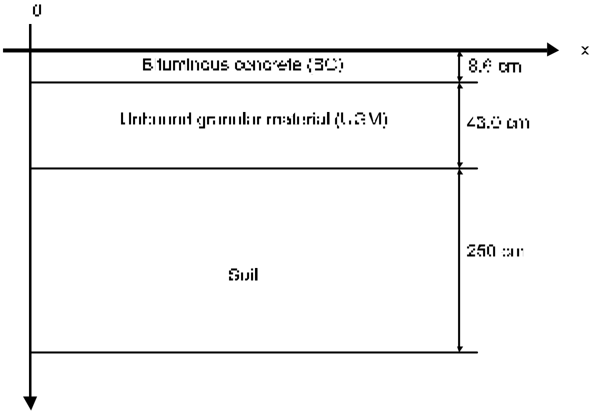
- Only 3D-calculations have been performed, as at LCPC (except for some preliminary comparisons with CESAR-LCPC).
- The mesh was the same for all calculations and described a quarter of the full pavement structure due to the double symmetry. This mesh consisted of 2002 hexagonal elements with 20 nodes.
- The load was represented as in the LCPC calculations. The pressures for these distributed loadings were 602kPa (65kN), 417kPa (45kN) and 694kPa (75kN).

8.1.3 Comparisons between CESAR-LCPC and ANSYS

Before modelling the full scale experiments, some preliminary calculations were performed to compare the two finite element programs CESAR (LCPC) and ANSYS (Uni of Hannover). For these comparison a simple 2D structure (axisymetric) was used, as shown on Figure 8.1.
Four calculations were performed and compared:

- linear elastic calculations with CESAR and with ANSYS,
- non-linear calculations with the Boyce model in CESAR and the Dresden model in ANSYS, using model parameters obtained for the same triaxial test.

Some main results of the comparisons of the two programs are summarised in Table 8.1. Figure 8.2 shows curves of variation along x of the vertical strain ε_y at the top of the granular layer obtained with the two non-linear models.



Applied load : 65kN load

uniformly distributed on a 375mm diameter disc

FIGURE 8.1: 2D-STRUCTURE FOR THE COMPARISON BETWEEN ANSYS AND CESAR

Model	Program	Surface deflection V	Radial strain at bottom of asphalt ε_r	Vertical strain at top of granular layer ε_y	Vertical strain at top of soil ε_y
		[µm]	[µdef]	[µdef]	[µdef]
Linear elastic	CESAR	-935	202.7	-1015	-805
Linear elastic	ANSYS	-927	204.4	-1015.5	-804.6
Boyce	CESAR	-1065	199	-910	-991
Dresden	ANSYS	-1134	220.5	-905.5	-986.4

TABLE 8.1: RESULTS OF COMPARISONS BETWEEN ANSYS AND CESAR (MAXIMUM STRAIN VALUES)

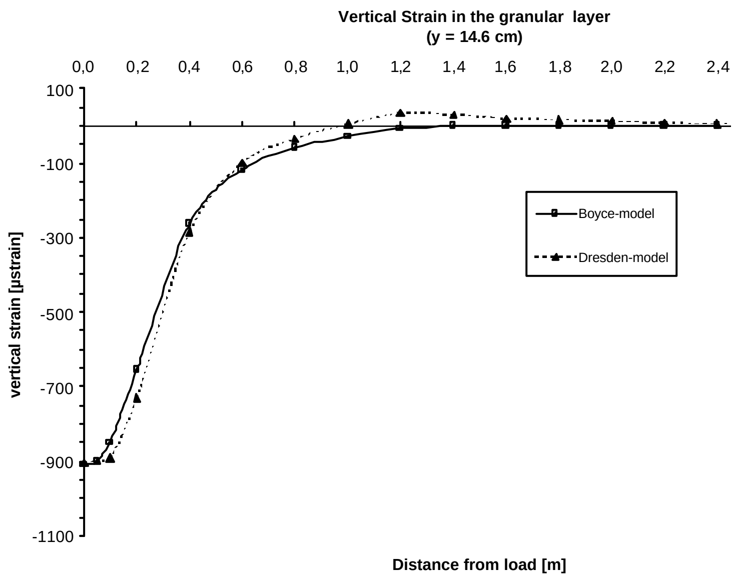


FIGURE 8.2: COMPARISON OF VERTICAL STRAINS ε_Y AT THE TOP OF THE UNBOUND GRANULAR LAYER OBTAINED WITH CESAR-LCPC (BOYCE MODEL) AND ANSYS (DRESDEN MODEL)

The results show that there is a good agreement between the finite element programs ANSYS and CESAR. With linear elasticity the differences are very small, but with the non-linear models, they are a little larger. For the radial strain in the bituminous concrete (Table 8.1), the maximum difference between the two models is 10%; for the vertical strains ε_y near the top of the granular layer, the difference is of the same order (Figure 8.2). But it can be noted that the Dresden-model tends to produce positive vertical strains (heave) at some distance form the load (x ≈ 1.2m) whereas the Boyce-model gives only negative vertical strains.

8.2 Analysis of the LCPC Test Track Experiment

The following approach was adopted for the modelling work for the LCPC full scale experiment:

- first, some linear elastic calculations were performed to test this "reference" model, which is still the most widely used for pavement design calculations, and also to back-calculate values of elastic modulus for the various pavement materials;
- secondly, the Boyce model was introduced for the granular material; a short parametric study was carried out to evaluate the effect of the model parameters on the results;
- finally, to further improve the modelling predictions, the non-linear behaviour of the soil (described by the Boyce model) and the visco-elastic behaviour of the bituminous concrete (described by the Huet-Sayegh model) were also taken into account.

8.2.1 Modelling of the Reference Loading (load q = 65kN, speed v = 67.8km/h)

8.2.1.1 Linear elastic calculations

The first case considered for modelling is the series of loadings with different lateral positions of the wheels (total load Q = 65kN, speed v = 67.8km/h). This case is called the "reference loading".

Firstly, calculations were performed using a *very simplified* linear model for all the materials, in order to test the simple hypothesis and determine values of average elastic modulus for the unbound materials (granular material and subgrade soil). The calculations were performed as follows:

- the value of the elastic modulus of the bituminous concrete was determined from the laboratory tests and considered constant;
- the values of elastic moduli of the granular layer and of the soil were varied, until a satisfactory fit with the experimental results was obtained.

The elastic parameters obtained using this approach are given in Table 8.2.

		bituminous concrete	UGM	soil
modelling		linear elastic	linear elastic	linear elastic
parameters of models	θ_{mBC} = 11.4°C θ_{mBC} = 7.0°C	E = 11700 MPa E = 15200 MPa ν = 0.25	E = 130 MPa ν = 0.35	E = 40 MPa ν = 0.35

TABLE 8.2: REFERENCE LOADING - PARAMETER VALUES FOR LINEAR ELASTIC MODEL

The comparisons between the main measurements (maximum values of strains under the centre of the half-axle and under the centre of one wheel) and the calculations are presented in Table 8.3.

			θ_{mBC} °C	surface deflection w (mm/100)		Maximum strains in the different layers							
						bituminous layer				granular base		soil	
						ε_{xx} (µstrain)		ε_{yy} (µstrain)		ε_{zz} (µstrain)		ε_{zz} (µstrain)	
				0m	0.19m	0m	0.19m	0m	0.19m	0m	0.19m	0m	0.19m
measurements			11.4		-87.9	112.8 ±9.6	110.3 ±8.8	59.3 ±5.5	109.3 ±0.4	-684.0 ±175.3	-641.9 ±184.6		
			7.0									-600.4 ±67.5	-558.7 ±77.3
calcs	BC UGM soil	linear elastic linear elastic linear elastic	11.4	-83.1	-80.3	154.6	154.1	26.2	124.9	-663.4	-699.9		
			7.0									-693.0	-642.9

TABLE 8.3: REFERENCE LOADING - LINEAR ELASTIC CALCULATIONS AND MAIN MEASUREMENTS (VALUES UNDER THE CENTRE OF THE AXLE: Y = 0M, AND UNDER THE CENTRE OF ONE WHEEL: Y = 0.19M)

The linear elastic calculations provided results with relatively good agreement to the experimental measurements, probably due to the relatively thick bituminous layer (86mm):

- the vertical surface deflection w and the vertical strain ε_{zzUGM} at the top of the granular base are well predicted;
- the vertical strain at the top of the soil ε_{zzsoil} is slightly higher than the mean of the measurements;
- the longitudinal strain ε_{xx} at the bottom of the bituminous layer is overestimated by about 35%. However, it should be noted that the value of ε_{xx} is difficult to estimate accurately, because it is very sensitive to several factors such as the exact position of the gauge (horizontal and vertical) and the shape of the tyres. For example, a variation of 10mm in

the vertical position of the gauge changes the measured value by about 20%. For this reason, it was not attempted to modify the modulus of the bituminous material in order to obtain a better adjustment for ε_{xx}, and only the value of modulus obtained from the laboratory tests (E = 11700MPa for θ = 11.4°C) was considered.
- the linear elastic model is not able to reproduce the non-symmetrical shape of the curves of variation of ε_{xx} and ε_{yy} along x, which is due to the viscous behaviour of the bituminous material. However, in this test, the effect of viscosity is fairly small, due to the low temperature (11.4°C) and high loading frequency (13Hz).

8.2.1.2 Calculations with the Boyce model
In a second phase, the Boyce model (isotropic and anisotropic) was introduced to describe the behaviour of the unbound granular material, the other materials remaining linear elastic (with the same parameters as in Table 8.2).

Two series of calculations were performed with the Boyce model to investigate the:
- isotropic Boyce model was applied using data applicable to three different moisture contents (w = 2.3, 3.8 and 4.8%). The results predicted using the Boyce model were not much better than with linear elasticity. Using w= 2.3 % the vertical strain at the top of the granular layer ε_{zzUGM} was greatly underestimated, whilst the vertical strain at the top of soil ε_{zzsoil} and the asphalt strain ε_{xx} were overestimated. The low value obtained for ε_{zzUGM} suggested that for w = 2.3%, the stiffness of the granular layer was too high. However, by considering a higher moisture for the granular material (w = 3.8%, giving a lower stiffness), the value of ε_{zzUGM} was, indeed, much closer to the measured value, but the strains ε_{xx} and $\varepsilon_{zz\ soil}$ increase further beyond the measured values.
- influence of the parameter n, which characterises the degree of non-linearity (or stress-dependency) of the material. Values of n between 0.35 to 0.7 (usual experimental range) were tested. It was found that increasing n improves only slightly the predictions, and so it was concluded that modifying n is not necessary at this stage.
- anisotropic Boyce model, with values of γ in the range of the experimental results (between 0.7 and 1, although some were greater than 1). The results indicated that decreasing γ form 1 to 0.8 caused a significant decrease in the vertical strain at the top of the granular layer ε_{zzUGM} which in turn increased the difference with the experimental results. As a result, it was decided to keep γ = 1 for the rest of the study.

Finally, it was decided to test the effect of a non-linear behaviour of the soil and of a viscoelastic behaviour of the bituminous material (Huet-Sayegh model). For the soil, in the absence of laboratory tests, it was decided to describe it using the Boyce model, which is also applicable to sandy soils, and the model parameters were estimated as follows:
- n and $\beta = (1-n)\dfrac{K_a}{6 \cdot G_a}$ were fixed to n=0.5 and β=0.25, which are usual mean values;
- the value of G_a was adjusted in order to fit the experimental value of ε_{zz} at the top of the soil. This led to : K_a = 92.5 MPa, G_a =30.8 MPa .

The first calculations showed that taking into account the non-linear behaviour of the soil greatly improved the estimation of ε_{zzsoil}, with the other strain values remaining practically unchanged. The introduction of the visco-elastic behaviour of the bituminous concrete, with parameter values obtained form the laboratory tests (see §7.2.2.3) only improved the estimation of the strains at the bottom of the bituminous layer, ε_{xx} and ε_{yy}. This most complete model leads to relatively good results except for the vertical strain at the top of the granular layer, which is still underestimated by about 20% to 25%.

8.2.2 Modelling Using Three Load Levels (load q = 45, 65 and 75kN)

8.2.2.1 Comparison of Four Models

For the analysis of this second series of experimental results, it was decided to compare directly four modelling hypotheses:

1. all materials linear elastic;
2. bituminous concrete and soil linear elastic, granular material described by the Boyce model;
3. bituminous concrete linear elastic, granular material and soil described by the Boyce model;
4. bituminous concrete visco-elastic, granular material and soil described by the Boyce model;

The values of the parameters used for the calculations with the 4 pavement models are given in Table 8.4. For the granular material, the parameters correspond to w = 2.3%.

	bituminous concrete		UGM	soil
Modelling	visco-elastic (Huet-Sayegh model)		non-linear elastic (Boyce)	non-linear elastic (Boyce)
parameters of models θ_{mBC} = 11.4°C	E_∞ = 34000 MPa E_0 = 70 MPa k = 0.22 h = 0.65, δ = 2.8	ν = 0.25 A0 = 1.940247 A1 = -0.373213 A2 = 0.00191198	K_a = 97.1 MPa G_a =135.8 MPa n = 0.473, γ = 1	K_a = 92.5 MPa G_a = 30.8 MPa n = 0.5, γ = 1

TABLE 8.4: TESTS WITH 3 LOAD LEVELS (V = 67.8 KM/H) PARAMETERS OF MODELS USED IN CALCULATIONS

Surface deflection:

Figure 8.3 presents comparisons between the experimental and calculated maximum surface deflections. The experimental results show that the variation of the deflection with load level was slightly non-linear. The four models gave similar predictions:

- the model with all materials linear elastic predicts, as expected, a linear variation of the deflection with load level, but still gave the best predictions;
- the introduction of the non-linear behaviour of the granular material and of the visco-elastic behaviour of the bituminous concrete had little effect;
- taking into account the non-linear behaviour of the soil slightly reduced the deflections, and made their variation with load level non-linear.

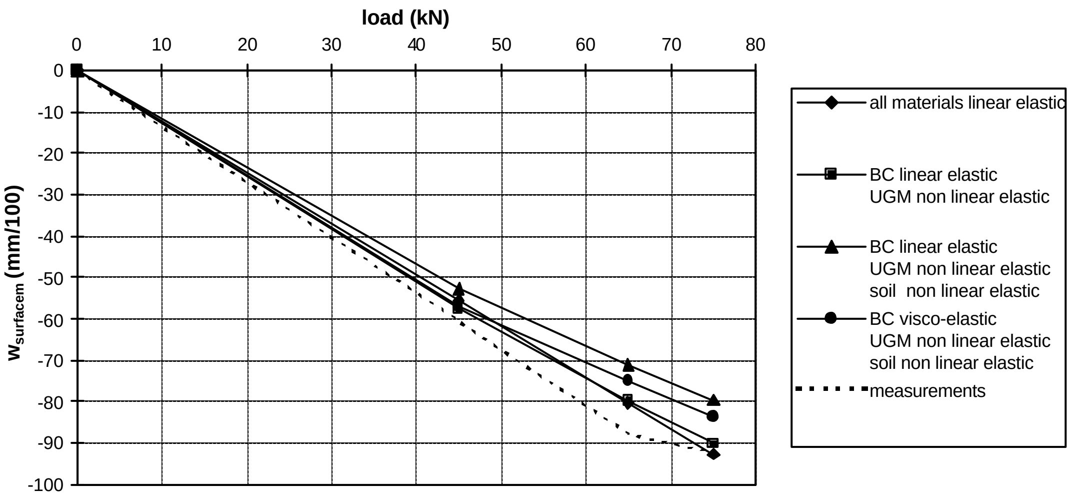


FIGURE 8.3: COMPARISON OF THE 4 MODELS - SURFACE DEFLECTION WITH LOAD

Longitudinal strains at the bottom of the bituminous layer:

The predictions obtained with the 4 models for the longitudinal strain ε_{xx} in the bituminous layer are presented in Figure 8.4:

- results obtained with linear elasticity were not satisfactory as values of ε_{xx} were over-predicted by about 30% and their non-linear increase with load was not correctly described (this could be improved by sub-layering the granular layer);

- the results improved when the non-linear behaviour of the granular material was taken into account. The variation of ε_{xx} with load level became non-linear, but the values obtained remain about 20% too high;
- the non-linear behaviour of the soil had practically no effect on ε_{xx};
- introduction of visco-elasticity for the bituminous material further reduced ε_{xx} by about 10% and gave the best prediction

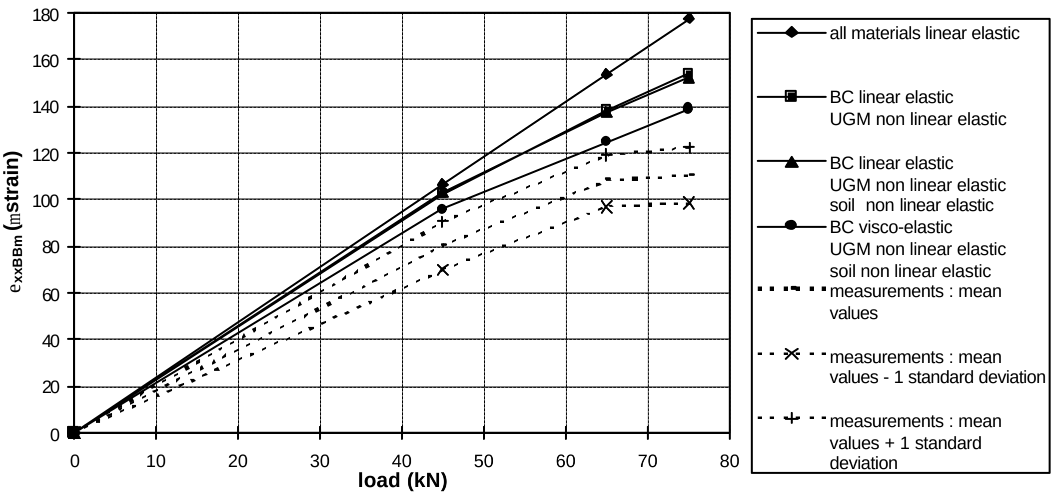


FIGURE 8.4: COMPARISON OF THE 4 MODELS - LONGITUDINAL STRAIN ε_{XX} WITH LOAD

Transversal strains at the bottom of the bituminous layer:

The results obtained for the transversal strain ε_{yy} in the bituminous layer are presented in Figure 8.5. They are similar to those obtained for ε_{xx}:

- the experimental variation of ε_{yy} with load level was clearly non-linear;
- the linear elastic model could not simulate the non-linear variation and over-predicted ε_{yy};
- the results improved significantly by accounting for the non-linear behaviour of the granular material;
- the behaviour of the soil had no effect;
- the introduction of visco-elasticity slightly reduced ε_{xx}, and lead to the best prediction

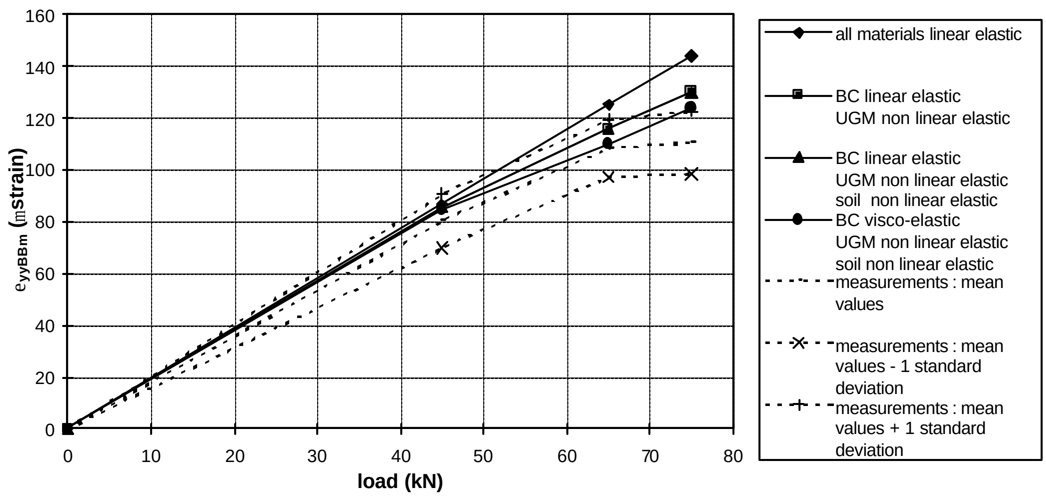


FIGURE 8.5: COMPARISON OF THE 4 MODELS - TRANSVERSAL STRAIN ε_{YY} WITH LOAD

Vertical strains e_{zzUGM} at the top of the granular layer:

The results obtained with the 4 models for ε_{zzUGM} are presented in Figure 8.6:

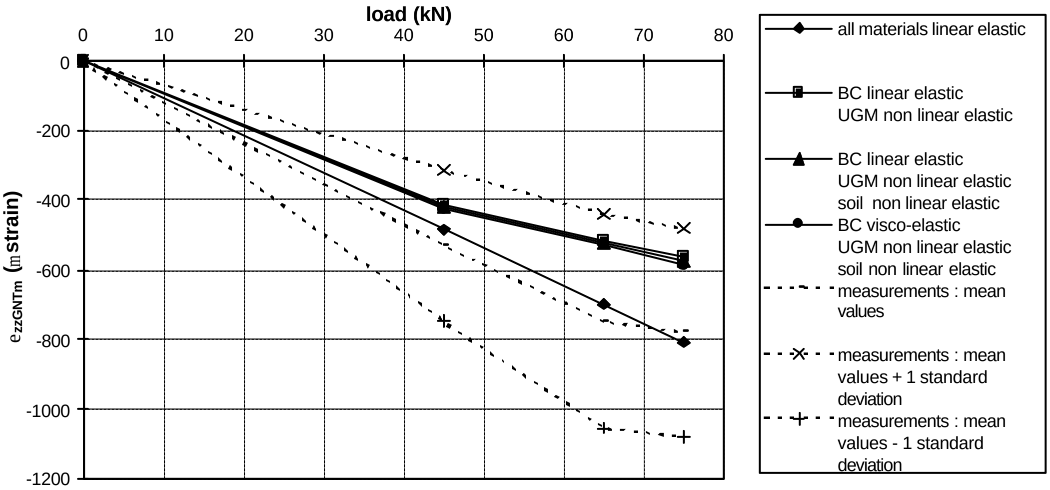


FIGURE 8.6: COMPARISON OF THE 4 MODELS - VERTICAL STRAIN AT THE TOP OF THE GRANULAR LAYER ε_{zzUGM} WITH LOAD

- the experimental variation of ε_{zzUGM} with load level was non-linear, but the experimental results were fairly scattered;
- the linear elastic model gave values of ε_{zzUGM} which were quite close to the mean of the experimental results, but which varied linearly with load level;
- the introduction of the Boyce model for the granular material lead to a non-linear increase of ε_{zzUGM} with load level, which better reflected the trend of the measurements; but the values obtained were at the low end of the measurements;
- changing the models of the soil and of the bituminous material had practically no effect on ε_{zzUGM}

Vertical strains e_{zzsoil} at the top of the soil:

The predictions obtained with the 4 models for ε_{zzsoil} are presented in Figure 8.7:

- the linear elastic model and the Boyce model for the granular material gave values of ε_{zzsoil} which were at the high end of the experimental measurements and varied linearly with load level;
- accounting for the non-linear behaviour of the soil significantly improved the data fit and produced a non-linear increase of ε_{zzsoil} with load level;
- the behaviour of the bituminous concrete had practically no effect

In conclusion, comparison of the 4 models confirmed the importance of taking into account:

- ➢ the non-linear behaviour of the granular material which mainly influences the strains in the bituminous layer, ε_{xx} and ε_{yy}, and in the granular layer ε_{zzUGM}. It lead to a non-linear variation of these strains with load, which agreed well with experimental observations;
- ➢ the non-linear modelling for the soil which significantly improved the prediction of the strains at the top of the soil ε_{zzsoil} (non-linear variation with load level) and also influenced the surface deflection;

the visco-elastic model for the bituminous concrete which lead mainly to a decrease in the bituminous layer strains, ε_{xx} and ε_{yy}, which was favourable. It had little effect on the strains in the unbound layers.

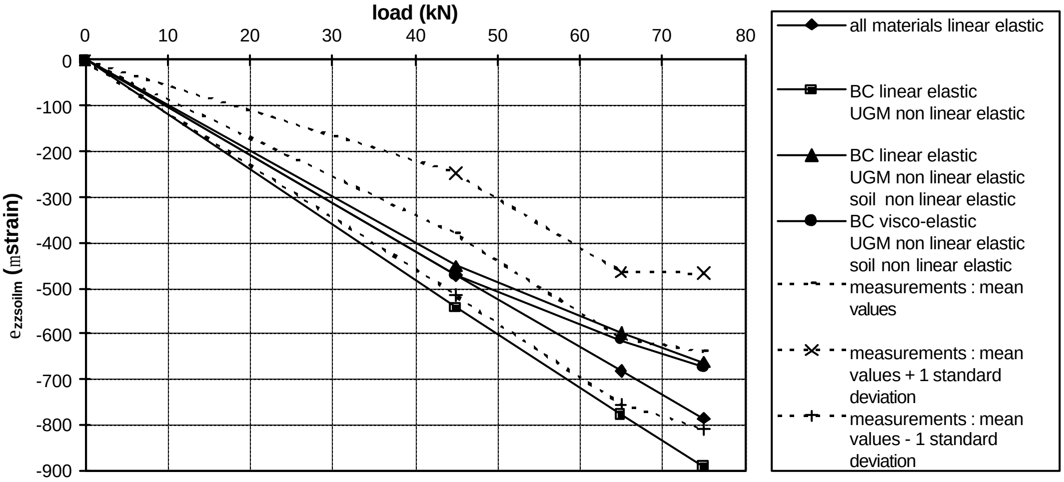


FIGURE 8.7: COMPARISON OF THE 4 MODELS - VERTICAL STRAIN AT THE TOP OF THE SOIL ε_{zzSOIL} WITH LOAD

8.2.2.2 Influence of the moisture content of the granular material

Calculations were performed with the "complete" pavement model (bituminous concrete visco-elastic, granular material and soil non-linear elastic), with Boyce model parameters corresponding to different moisture contents of the granular material (w = 2.3%, 3.8% and 4.8%) to appreciate the effect of the moisture content on the response of the pavement (see Figures 8.8 to 8.11).

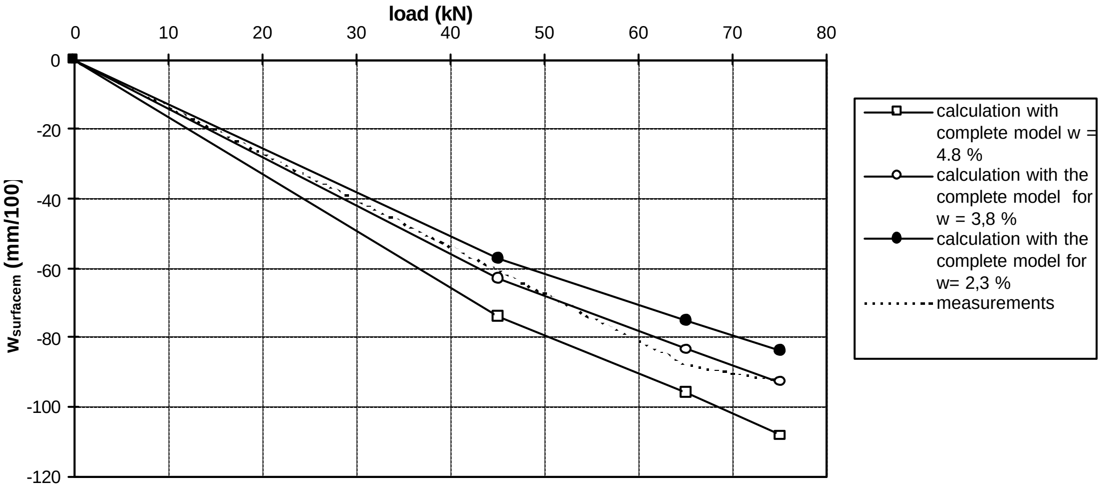


FIGURE 8.8: CALCULATIONS WITH THE COMPLETE MODEL - SURFACE DEFLECTION WITH LOAD

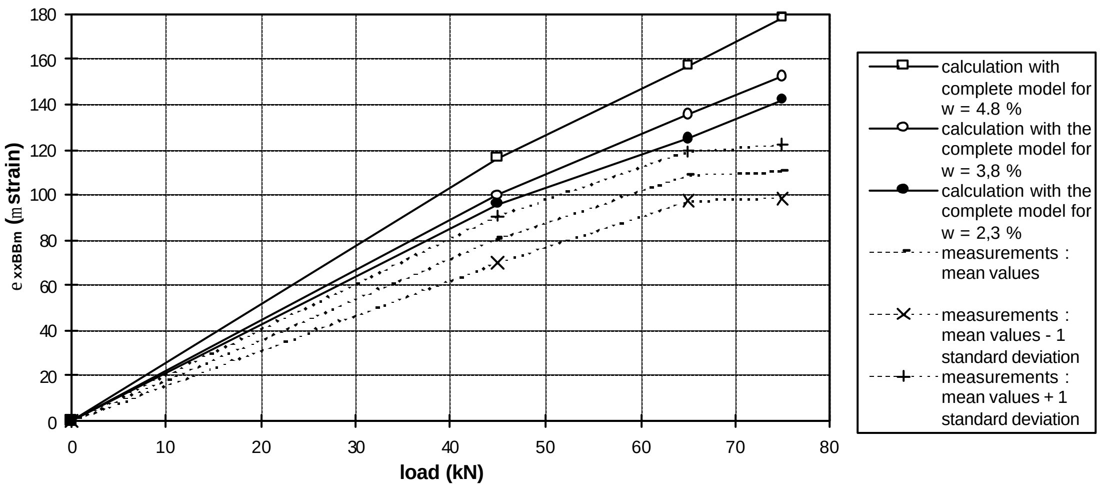


FIGURE 8.9: CALCULATIONS WITH THE COMPLETE MODEL - LONGITUDINAL STRAIN ε_{XX} WITH LOAD

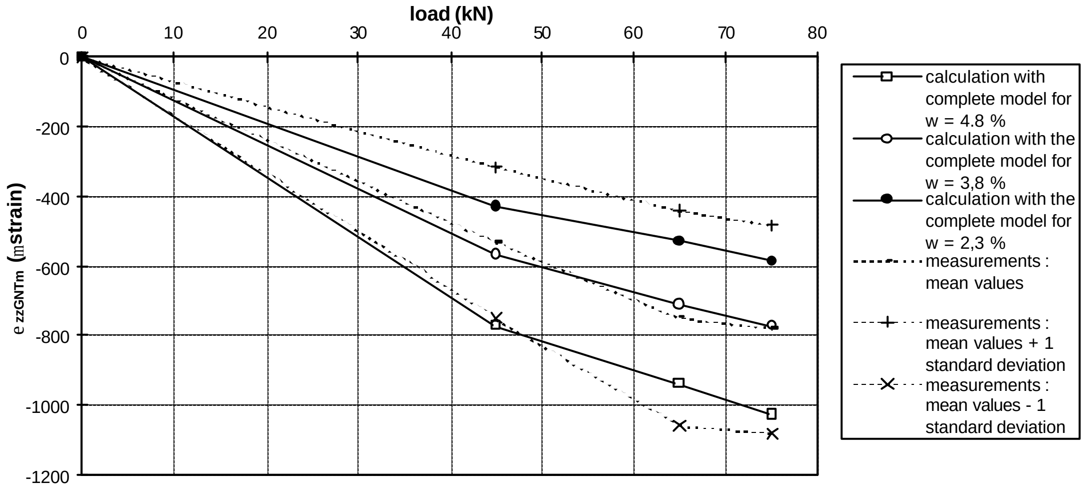


FIGURE 8.10: CALCULATIONS WITH THE COMPLETE MODEL - VERTICAL STRAIN AT THE TOP OF THE GRANULAR LAYER ε_{zzUGM} WITH LOAD

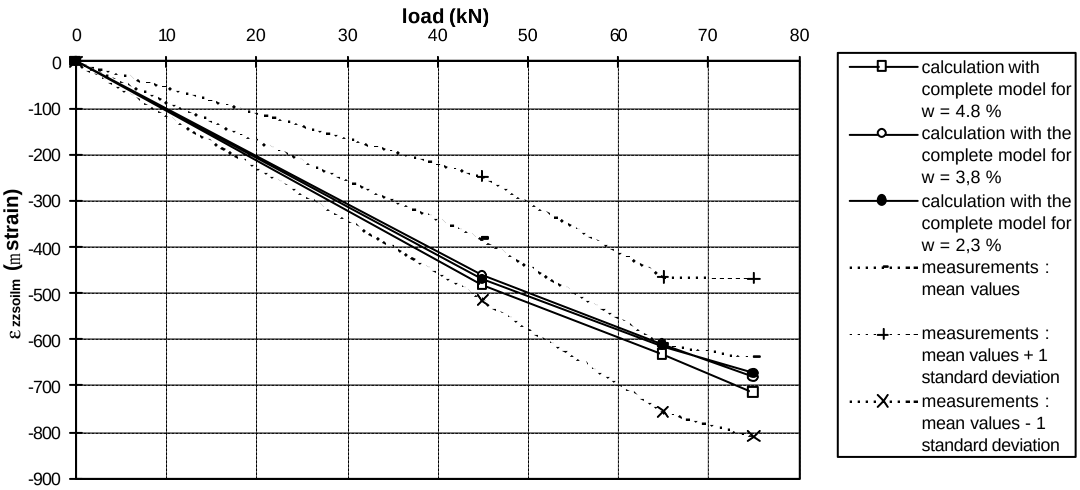


FIGURE 8.11: CALCULATIONS WITH THE COMPLETE MODEL - VERTICAL STRAIN AT THE TOP OF THE SOIL ε_{zzSOIL} WITH LOAD

The results of Figures 8.8 to 8.11 confirm that the complete model describes fairly well the response of the pavement for the 3 load levels. The best adjustment is obtained for the moisture content of 3.8%, which provided the best prediction for the strains in the granular layer, but slightly overestimated the strains in the bituminous layer. Figures 8.12 to 8.14 present examples of detailed comparisons between the experimental and calculated variations of the strains with the position x of the load (for 65kN). The results show that the complete model, (with w = 3.8%) predicted the strain signals quite well. The experimental curves of variation of ε_{xx} and ε_{yy} at the bottom of the bituminous concrete are non-symmetrical, due to the viscosity of the material, and this is well described by the visco-elastic model.

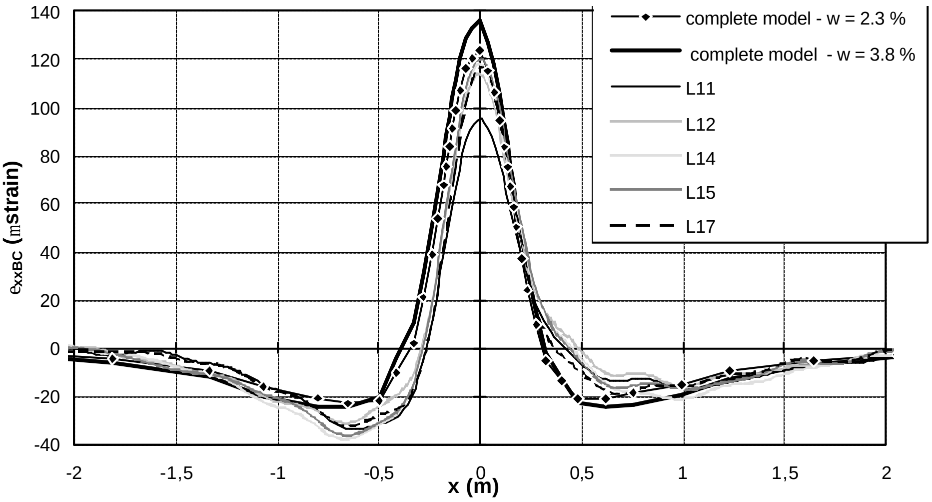


FIGURE 8.12: COMPLETE MODEL - LOAD 65KN - LONGITUDINAL STRAIN ε_{xx} AT THE BOTTOM OF THE BITUMINOUS LAYER (VARIATION ALONG X FOR Y = 0)

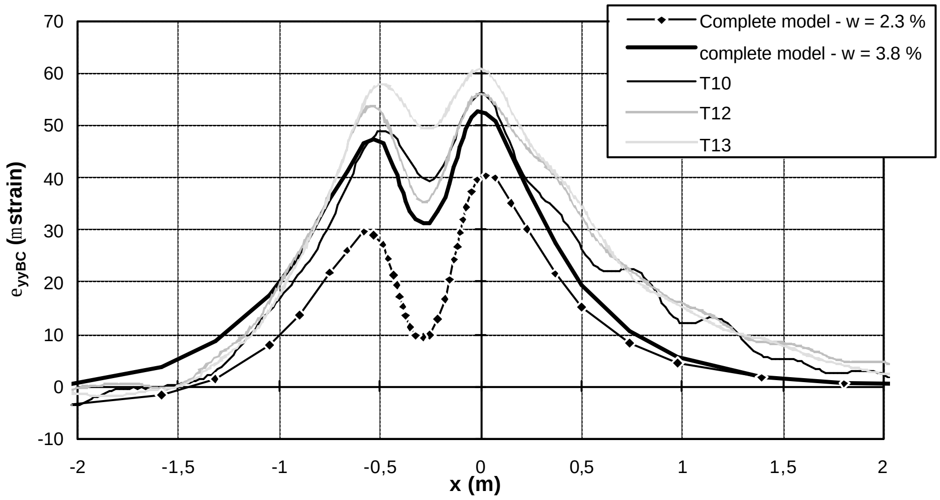


FIGURE 8.13: COMPLETE MODEL - LOAD 65KN - TRANSVERSAL STRAIN ε_{YY} AT THE BOTTOM OF THE BITUMINOUS LAYER (VARIATION ALONG X FOR Y = 0)

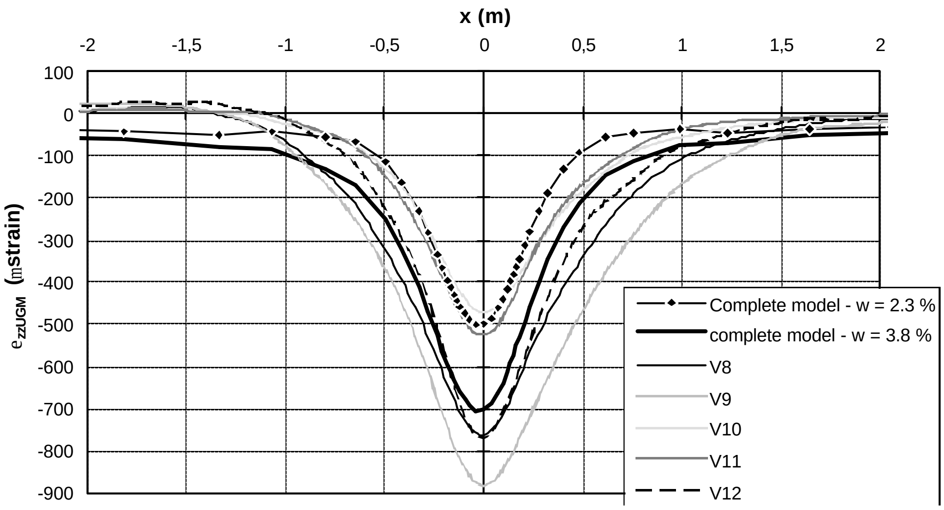


FIGURE 8.14: COMPLETE MODEL - LOAD 65KN - VERTICAL STRAIN ε_{zzUGM} AT THE TOP OF THE GRANULAR LAYER (VARIATION ALONG X FOR Y = 0)

8.2.3 Modelling of the tests with different loading speeds (Load Q = 65kN, v = 3.4, 6.8, 13.6, 27.1, 47.5 and 67.8km/h)

For the modelling of this last series of tests, only the most complete model was used. The values of parameters adopted were the same as in §8.2.2.2, but only two sets of Boyce model parameters, corresponding to w = 2.3 and 3.8% have been used for the granular material. As previously, the best predictions are obtained for the moisture content w = 3.8%.

The following comments concerned the modelling of results for the different loading speeds are made:

- the model underestimates the experimental values of the deflection slightly and, what is more important, underestimates the variation of the deflection with speed by approximately a factor of 2. Between 3.4km/h and 68km/h the experimental variation of the deflection is about 20%;
- the model tends to overestimate slightly the asphalt strains ε_{xx} and ε_{yy}, but predicts fairly well the variation of the asphalt strains with speed. Between 3.4km/h and 68km/h, the values of ε_{xx} and ε_{yy} decrease by about 30%. This confirms the good validity of the visco-elastic model for the bituminous material;
- the values of ε_{zzUGM} and their variation with speed are well predicted;
- it can be seen that the measured vertical strain at the top of the soil ε_{zzsoil} varies largely with the loading speed (between 3.4km/h and 68km/h, the decrease is about 25%), and this is not well predicted by the model. This strong influence of the speed (or frequency) on ε_{zzsoil} (and also on the deflection, which depends largely on the stiffness of the soil) indicates clearly that the behaviour of the soil is sensitive to loading frequency.

So, in this last series of tests, difficulties are encountered with the prediction of the variations of the surface deflection and of the strains at the top of the soil with loading speed. These difficulties do not seem to be due to the visco-elastic behaviour of the bituminous concrete (which seems to be modelled correctly), but to the effect of loading frequency on the response of the soil, which is not taken into account in the modelling.

8.2.4 Additional calculations with ANSYS (Dresden model)

In addition to the modelling work performed with CESAR LCPC, a brief discussion is presented concerning the results obtained using the program ANSYS, with the Dresden model describing the unbound granular material. The modelling hypotheses are those presented in 8.1.2. The series of calculations were for the reference loading (65kN load, 67.8km/h speed). Modelling considered the bituminous material and the soil as linear elastic (with the same parameters as used with CESAR). The granular material, as described by the Dresden model, used parameters obtained for 3 moisture contents w = 2.3%, 3.8% and 4.8%.

The calculations performed with ANSYS using the Dresden model were compared with the experimental measurements and results of a linear elastic calculation. The results show that the ANSYS calculations gave reasonable values for the surface deflection and for the vertical strains at the surface of the soil ε_{zzsoil}. The values of the horizontal strains at the bottom of the asphalt layer and of the vertical strains at the top of the granular layer, on the contrary, were unrealistically low (except for ε_{xx} for y = 0.19 m, which is in the correct range).

From these results, it was concluded that the Dresden model was not implemented correctly in ANSYS. However, despite many tests, the University of Hanover could not find the cause of problem (occurring only in 3D), and so the work with ANSYS was not pursued any further.

8.3 Conclusions of the Modelling Concerning the LCPC Experiment

8.3.1 General Conclusions

The modelling was carried out mainly with the program CESAR-LCPC which allowed several constitutive models for the pavement materials (linear elasticity and the Boyce model for the unbound layers, linear elasticity and visco-elasticity for the bituminous concrete) to be tested.

It was found that the response of the pavement with load level was non-linear (stress-dependent), but that this non-linearity was relatively "limited" due to the thickness of the bituminous layer. A greater non-linear response would have been obtained with a thinner bituminous wearing course (say 40mm). For this reason, the differences between the various modelling hypotheses were also relatively limited.

Concerning the various models tested for each material, the study showed that the:

- Boyce model, with values of parameters determined from laboratory repeated load triaxial tests, gave good predictions of the strains in the unbound granular layers applicable to the lower stress conditions existing in the surfaced pavement. This confirmed the validity of the repeated load triaxial test for predicting the resilient behaviour of unbound granular materials at lower levels of stress. However, the use of the Boyce model for the granular base alone (the other materials remaining linear elastic) was not sufficient to predict correctly the vertical strains at the top of the soil and the horizontal strains in the bituminous layer.
- response of the subgrade soil was non-linear, and the use of the Boyce model for the soil improved the prediction of the strains in the soil. It was also noted that the behaviour of the soil was sensitive to the loading frequency, but this could not be taken into account in the modelling.
- Huet-Sayegh visco-elastic model, with parameters determined from laboratory complex modulus tests, was found to predict the strains in the bituminous concrete quite well. It was found particularly useful in determining the response of the pavement for various conditions of temperature and loading frequency.

Concerning the granular material, the study also showed the important influence of the moisture content of the granular material on the response of the pavement, and particularly on the strains in the granular layer. It was found that an increase of the moisture content from 2.3% (38% of OMC) to 4.8 % (80% of OMC) lead to an increase of:

- about 25% of the pavement deflection and of the horizontal strains at the bottom of the bituminous layer,
- an increase of 80% of the vertical strains at the top of the granular layer.

8.3.2 Influence of the modelling approach on pavement design

To evaluate the significance of the various modelling approaches, the results of the modelling calculations are used to make predictions of pavement life. In this case, the French mechanistic design method will be used which is based on the following main principles:

- The stresses and strains in the pavement layers are calculated using a multi-layer linear elastic pavement analysis program (program ALIZE, based on the Burmister model). The standard axle load is a dual wheel axle load of 130kN.
- The design criteria for bituminous pavements are: the maximum horizontal strain ε_t at the bottom of the bituminous layer; the maximum vertical strain at the top of the granular layer ε_{zUGM} and the maximum vertical strain at the top of the soil ε_{zsoil}. However, for low traffic pavements, (design life less than 250,000 equivalent axle loads, which leads to an asphalt thickness less than approximately 100mm), only the soil vertical strain criterion applies.
- The principle for design is to compare the calculated strains in the pavement layers with

limit strain values which depend mainly on: the characteristics of the material, the traffic, the risk of failure and a coefficient of adjustment depending on the material. Inversely, knowing the strains, a design life of the pavement, in number of standard axle loads, can also be calculated.

- For unbound granular layers, their modulus is determined on the basis of an empirical classification, depending on several aggregate characteristics.

The design method is used to calculate design lives of the pavement using strains calculated with different approaches:

- The standard French design method (unbound layer moduli from empirical classification),
- Modelling with all materials linear elastic (as done in this chapter),
- Modelling with the "complete model" as done in this chapter, considering different moisture contents of the granular material.

The results of these design calculations are presented in Table 8.5. Two important remarks must be made about these results:

1. The **design values** of strains considered in Table 8.5 are values determined at the interface of the pavement layers (ε_{tBC} right at the bottom of the bituminous layer, ε_{zz} right at the top of the granular layer or of the soil), whereas the **experimental values** of strains, considered in the modelling work, are measured at slightly different levels (10mm above the interface for ε_{tBC}, 50mm below the interface for ε_{zz}, due to the transducers used). For this reason, the "experimental" strains presented in this table are extrapolated values at the interface (a correction, based on linear elastic calculations, to take into account the difference in level).
2. If the French pavement design method was applied strictly here, only the strain criterion for the soil should be used. However, it was considered that the thickness of the bituminous layer (85mm) was still sufficient to consider the asphalt strain criterion as well. This criterion leads to a design life which agree quite well with the experimental observations (40% of the pavement surface cracked after 500,000 loads).

		Design criterion		
Modelling approach		e_{tBC}	e_{zzUGM}	e_{ZZsoil}
Experimental strains (corrected) [(2)]	Strain value (µstrains) NE[(1)]	*155* **476,000**	*828* **621,000**	*678* **1,530,000**
Standard design method	Strain value (µstrains) NE[(1)]	174 267,000	455 9,210,000	711 1,230,000
Linear elasticity E_{UGM} = 130MPa	Strain value (µstrains) NE[(1)]	212 99,600	804 709,000	783 799,000
Complete model w = 2.3%	Strain value (µstrains) NE[(1)]	170 300,000	605 2,550,000	744 1,010,000
Complete model w = 3.8%	Strain value (µstrains) NE[(1)]	186 191,600	852 546,000	741 1,020,000
Complete model w = 4.8%	Strain value (µstrains) NE[(1)]	219 84,700	1147 143,100	767 877,000

(1) NE = number of equivalent standard axle loads (130kN).
(2) The experimental strains are corrected to account for the difference in level between the position of the transducer and the pavement layer interface.

TABLE 8.5: ESTIMATION OF DESIGN LIFE FOR LCPC PAVEMENT WITH DIFFERENT MODELLING HYPOTHESES

The design calculations (Table 8.5) show that:

- With all the modelling hypotheses, the failure is governed by the asphalt strain criterion.
- The experimental strains give a design life of 476,000 standard loads, which is in fairly good agreement with the experimental behaviour of the pavement
- The standard pavement design method, which leads to a modulus of 400MPa for the granular base and 100MPa for the sub-base, gives a design life of 267,000 standard loads, which is much lower than the real life. However, with this approach, the vertical strain at the top of the granular layer is strongly underestimated.
- The linear elastic model (where the modulus of the granular material is chosen to match the measured strains in the granular layer) leads to a low, unrealistic, design life. It is important to note that the response model used for calculations was a very simple, one with no sub-layering of the granular or soil layers implemented to account for the non-linearity of these materials. It is expected that much improved results, particularly for the asphalt and soil could have been expected with a more refined model.
- The results obtained with the "complete model" show clearly the effect of the moisture content of the granular material on the design life of the pavement. The lowest moisture content, w = 2.3%, which is close to the moisture content measured experimentally at the end of the experiment, yields a reasonable design life of 300,000 standard loads. For the intermediate moisture content, w = 3.8%, which was found to predict well the experimental strains **at the beginning of the experiment,** the design life is reduced by a ratio of 1.6. For the highest moisture content, w = 4.8%, the reduction of the design life is very important (ratio of 3.5 compared with w = 2.3%).
- Concerning the other design criteria, (which do not govern the failure) It can be noted that: the soil criterion yields high design lives (between 800,000 and 1,500,000 loads) and does not vary much with the pavement model used. The granular layer criterion, on the contrary, gives design lives, which depend very strongly on the pavement model used and which, in the case of linear elasticity and the complete model, are quite realistic.

The design calculations presented here are only an example, obtained with one particular design method (the French method). However, they serve to illustrate the importance for the design of the choice of the modelling hypotheses for the granular layer (linear or non-linear) and also of the moisture content of the granular material (which depends on drainage and climatic conditions).

9. RELATIONSHIP WITH OTHER EC PROJECTS

9.1 Projects Related to COURAGE

A number of EC RTD Projects currently active are closely linked with COURAGE as they have an interest in pavement material performance.

COST 337, Unbound Granular Materials for Road Pavements

COST 337 is concerned with all unbound granular materials, including alternative materials. This project has very strong links with the COURAGE project in many respects as it aims to:

- develop the measurement of the structural properties of unbound materials
- establish the most important factors affecting their performance
- derive values to be used in pavement design
- develop appropriate measurement technologies to assist in the standardisation of mechanical testing of unbound materials
- stimulate the use of secondary materials where appropriate.

The COURAGE project uses the 'appropriate' test measurement technologies and methodologies, as determined by COST337, to determine 'key' unbound granular material performance indicators, such as strain rate and resilient modulus. In addition, COURAGE has practically used some of the constitutive models (recommended by COST337) for describing the resilient and permanent deformation behaviour of granular materials subjected to laboratory repeated loading, to simulate the action in road pavements. These models have assisted in developing a better understanding of material behaviour and they have been implemented into a computer modelling program code to allow the analytical modelling of pavement structures for performance assessment. Both the COST337 and COURAGE projects should produce valuable feedback to the CEN test method inquiry process to improve the standardisation of mechanical testing of unbound pavement materials.

COST 336, Falling Weight Deflectometer

This project deals with issues surrounding the use of Falling Weight Deflectometers for measuring the bearing capacity of a road pavement, and also with the post processing procedures required for actual assessment of the strengthening overlay.

Task 1 has most relevance for the work of COURAGE as it addresses the problem of back-calculating in-situ layer resilient modulus in road pavements for various materials. Data collected in countries, particularly those participating in both COST 336 and 337, has been great assistance to COURAGE particularly in determining in-situ moduli values for the trials described in Chapters 4 and 8. It has allowed some comparisons to be made with calculations of moduli from in-situ measurements by FWD and laboratory measurements using repeated load triaxial testing, which is central to the work of COURAGE. FWD data from countries such as Iceland, Ireland and Finland where unbound bases with thin surfacings are widely used, is of particular interest.

ALT-MAT, Alternative Materials in road construction

ALT-MAT is intended to encourage the wider use of alternative unbound granular materials in road construction. The project aims to provide information to bridge the gap between laboratory tests and field behaviour. The objective is to define methods by which the suitability of alternative materials for use in roads can be evaluated under appropriate climatic conditions. The methods will cover the basic mechanical properties, functional requirements, leaching potential and long-term stability of materials, which will be used in an unbound state.

Close links have been maintained between COURAGE and ALT-MAT to compare the results of simple tests undertaken on primary and secondary process materials and to predict which of the alternative materials would be suited to having more rigorous functional tests applied to them. Some of the alternative materials have been shown to clearly be unsuitable for high to moderate stress level applications in the loaded pavement structure. Conversely, some of the other materials, deemed somewhat marginal in nature, will only be able to be tested under a new modified CEN RLT stress regime.

The selection of alternative materials in ALT-MAT is of importance to COURAGE. Interest lies in whether the many selected alternative materials such as:

* recycled crushed concrete
* blast furnace slag
* steel slag
* municipal solid waste incinerator ash

perform under mechanical *and* environmental tests. One material common to both projects is recycled crushed concrete. Tests characterising the strength and stiffness of a supply product of this material have been carried out in the COURAGE project, while other tests such as leaching tests have been carried out in ALT-MAT using a different source material. Alt-Mat will monitor the performance of this material by studying a 5 year-old constructed pavement in the UK, where concrete was used as sub-base, and a 6 year old site in Denmark, where crushed concrete was used as unbound road base. In both of these countries reference sites with natural aggregates have also been identified.

The work carried out in Work Package 3 of ALT-MAT obtains information on the performance of alternative materials, which have been used in road construction. The properties of materials presently found in trafficked roads is being compared with the initial information which was at hand at the time of construction. The choice of test methods for use in ALT-MAT should be influenced by the work of COST 337 and COURAGE, and the results from ALT-MAT should be fed back into these two projects.

COST 333 and AMADEUS, Advanced Models for Analytical Design of European Pavement Structures

COST333 undertook an exercise to analytically evaluate existing, advanced pavement design model approaches by comparing their predictions using a number of standard inputs. The ability of these models to deal with different materials, pavement construction, climate and traffic characteristics was considered. The study showed that of the 29 countries that participated in the exercise, 30% of these used material performance parameters (resilient modulus and Poisson's ratio) as input into the design response models. It should be remembered that such material parameters are directly measured in the COURAGE project. The evaluation lead to recommendations and guidelines to promote appropriate use of these design models and associated material performance parameters.

After the work of COST333, AMADEUS then established a framework for an improved pavement design methodology using a number of design elements, of which pavement material performance properties and climate data were two. This framework means that a wide range of construction materials can be taken into consideration. Current methods, generally, only deal with higher-grade 'standard production' aggregate materials and they are not versatile enough to deal with newer recycled materials, which can reduce environmental problems.

The evaluation exercises undertaken to date within the AMADEUS project have been carried out with models, which clearly show how important the characteristics of the granular materials are for the evaluation of stress/strain conditions in the pavement. It was reported that to improve the design models further, more realistic models and data for granular materials should be used. In addition, the importance of climate data to enable prediction of incremental changes in material properties should not be forgotten. This illustrates the value of the work contribution to AMADEUS by the COURAGE project.

9.2 Beneficiaries of COURAGE Information

To illustrate how the many linked projects and key end-users would view the value of the information stemming from COURAGE, a number of potential beneficiaries of the technical content of COURAGE Final Report have been identified in Table 9.1.

Potential Users	This Report's Technical Chapters					
	4	5	6	7	8	A6
Researchers in Pavement Materials	*	*	*	*	*	*
Pavement Designers and Maintenance Engineers	*			*		*
Researchers in Pavement Performance Modelling	*			*	*	
Researchers in Moisture Variations in Pavements	*					*
Quarry Industry Groups	*	*		*	*	
COST - Actions						
COST 324 (Long term performance of road pavements)	*			*		*
COST 333 (Development of new bituminous pavement design method)	*		*	*	*	*
COST 336 (Falling Weight Deflectometer)	*			*		*
COST 337 (Unbound granular materials for road pavements)	*	*	*	*	*	*
Transport RTD Projects						
ALT-MAT (Alternative Materials for road construction)	*			*		*
AMADEUS (Advanced Models for Analytical Design of European pavement Structures)			*	*	*	*
FEHRL Projects						
PAVE-MAINT (Pavement maintenance)	*			*		*

TABLE 9.1: USERS WHO COULD BENEFIT FROM THE TECHNICAL CONTENT OF THE COURAGE REPORT

10. COST BENEFIT ANALYSIS

10.1 Introduction

Maintenance aims to provide longevity to a pavement, which results in benefits such as reduced vehicle operating costs. Quantifying such benefits, and associated costs, in a systematic way provides a method of identifying the advantages and disadvantages of undertaking an investment in highways and, at a detailed level, the type of pavement to be adopted.

A cost-benefit evaluation of highway pavements in four European nations (France, Finland, Ireland and Slovenia) was considered within the COURAGE Project. The analyses, undertaken by Scott Wilson Ltd, used a simple economic model containing information on highway construction and maintenance from these countries to consider the economic benefit to road users. The possibilities of lower initial costs and whole-life costs of pavement construction were evaluated by comparing two different flexible pavement types, containing unbound granular materials, to a "base-case". Where information was not available on a country specific basis, appropriate values from other European countries and experience was used. This study considered changes in journey time and vehicle operating costs

10.2 Pavement Types Studied

The framework for evaluation utilised current construction and maintenance practice for three types of highway pavement:

A Base Case: 100mm surface treatment, 150mm deep crushed stone or gravel unbound base and 150mm deep crushed stone or gravel unbound sub-base.

Type 1 pavement: Same as construction as the Base Case, except lower quality materials are used for the base and sub-base thus reducing the pavement life from 20 to 10 years.

Type 2 pavement: 80mm surface treatment, 150mm deep crushed stone or gravel unbound base and 300mm deep crushed stone or gravel unbound sub-base.

Figure 10.1 illustrates the different thicknesses of pavement construction used in the study nations. These thicknesses are assumed throughout this study and reflected in the costs presented below.

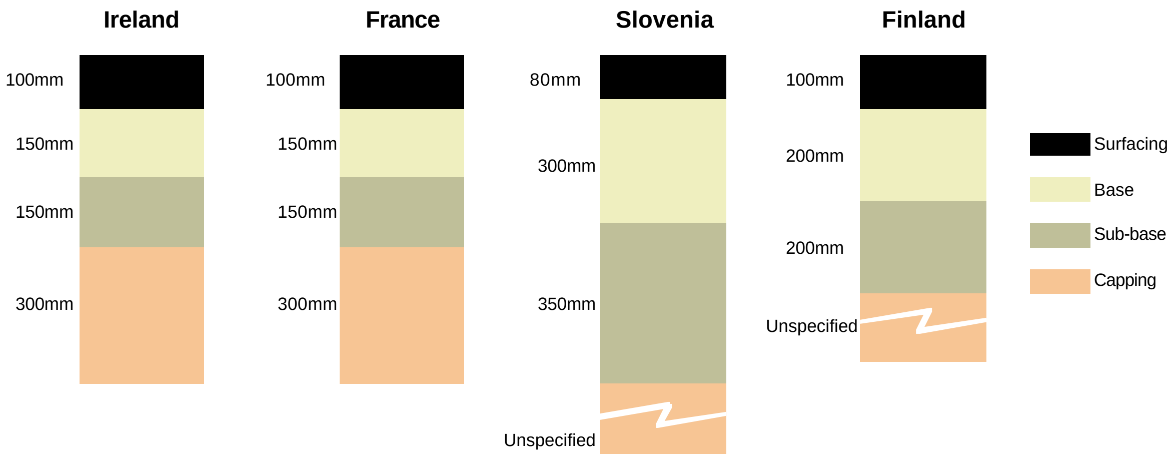


FIGURE 10.1: TYPICAL LAYER THICKNESSES OF PAVEMENTS ANALYSES

10.3 Assumptions Used

The following 'key' assumptions were used in the analysis:

- Single carriageway, length of 40 km, width 13.50m comprising two 3.75m wide lanes and two 3m wide paved hard shoulders.
- Construction, maintenance and associated costs were representative of a sample of similar roads in the study nations.
- Maintenance regimes were modelled based on information provided by team members which reflected been defined by current practise in each nation.
- A 30 life of asset based on NRA timings of expenditure for Base Case and Type 1. A 35 year asset life was assumed for pavement Type 2.
- No assumption was made on the deterioration of the pavement between investments.
- Information from NRA provided the year of maintenance for each pavement type and these were assumed applicable to each country. Maintenance investment timings in Finland and Slovenia were assumed to be the same as those of Ireland, whilst adjustments to the timings of maintenance were assumed for France. As maintenance practise is significantly different between the nations, it makes a like-for-like comparison rather difficult.
- Two vehicle classifications adopted for this analysis were:
  - Light vehicles,
  - Heavy vehicles with a HV component of 15% was assumed throughout the evaluation.
- Where possible, different annual growth rates were used for light and heavy vehicles.
- The main disbenefit of maintenance expenditure arose during the maintenance period due to disruption to traffic using the road, i.e. increase travel time and therefore a disbenefit to road users. Value of time (VOT) represents the trade-off people place between time and money; it is the amount of money one pays to save a certain amount of time.
- Vehicle operating costs (VOCs) were estimated using a function of road roughness to VOC taken from the Highway Design and Maintenance standards model version 3 (HDM III).

10.3 Analytical Findings

The analysis sought to ascertain, on a country by country basis, whether a lower initial capital expenditure with frequent maintenance expenditures offered greater or less benefit than a large initial capital expenditure, which defers maintenance expenditure. Each country has its own highway maintenance practise and the analysis attempted to find common ground between them. In doing so, some broad assumptions have been made and qualified as described above.

Comparison with the Base Case

A comparison of the net present value (NPV) over time of each pavement type compared to the base case for each country was undertaken. Common to all was that a positive NPV compared to the base case for both pavement types. Further, the analysis suggested that Type 2 pavements offer significantly better economic benefits than Type 1 compared to the Base Case.

Cost Savings

Pavement Type 2 offered greater benefits compared to the Base Case than pavement Type 1 for all of the study nations. These benefits were mainly derived from construction and maintenance cost savings that were approximately 3 to 5.5 times greater (in present values) than pavement Type 1 cost savings. For all the study nations, over 90% of the total benefit (NPV) of Type 2 pavements was derived from cost savings.

Reduction in Disbenefits

One pavement type did not appear to consistently offer a greater reduction in disbenefits than the other pavement type. The reduction in disbenefit was significantly smaller than for cost savings. France was the only nation to show an increase in disbenefit from investing in pavement Types 1 and 2 compared to the Base Case. However, the cost savings were 4 (Type 1) and 19 (Type 2) times greater than the absolute value of the increased disbenefits thus resulting in a positive NPV.

Overall

Most of the benefits of pavement Type 1 and Type 2 were elicited from initial cost savings as shown in Figures 10.2 and Figure 10.3. In general, pavement Type 2 offers a greater benefit than pavement Type 1 compared to the Base Case. The basis of the assumptions made in this analysis result in a wide range of NPVs from 5.7 billion Euro to about 0.3 million Euro.

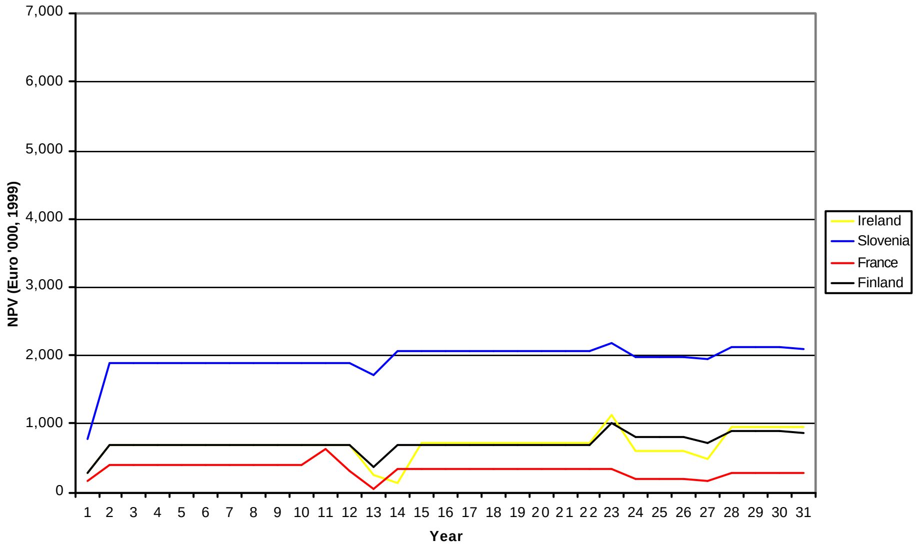


FIGURE 10.2: COMPARISON OF PAVEMENT TYPE 1 BENEFIT TO THE BASE CASE FOR THE STUDY NATIONS

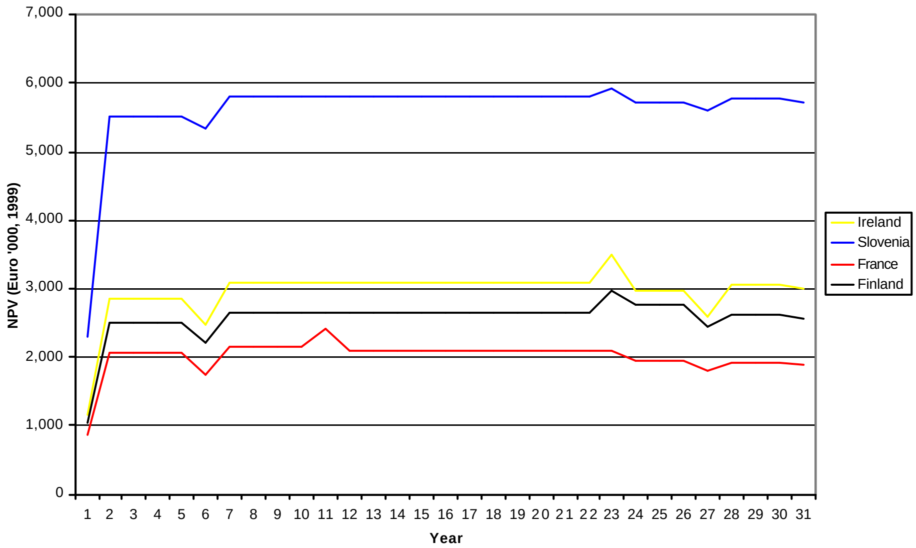


FIGURE 10.3: COMPARISON OF PAVEMENT TYPE 2 BENEFIT TO THE BASE CASE FOR THE STUDY NATIONS

Ranking the nations by NPV showed Slovenia to benefit the most from the alternative pavement types. Over 80% of the benefit was elicited from cost savings compared to reduction in disbenefits. The greatest absolute benefit of investing in Type 1 (NPV = 2.1 billion Euro) and Type 2 (NPV = 5.7 billion Euro) pavements was experienced by Slovenia. Both lower quality pavements require frequent expenditure but the benefit of a lower initial capital expenditure and deferred, albeit more frequent, rehabilitation expenditure produces a positive net present value.

11. CONCLUSIONS AND RECOMMENDATIONS

This chapter summarises the main conclusions of the work of COURAGE based on the research results obtained.

CLIMATIC EFFECTS ON UGM WITHIN PAVEMENTS:

- In all European countries in which pavements were monitored for seasonal moisture changes, it was observed that the variation in all structural layers followed clearly defined seasonal variations, with the moisture content being the highest in autumn and spring.

- Considerable seasonal moisture variations can occur in the pavement structure from country to country, primarily due to material type, temperature and rain/snow fall. Test pavements showed the following variations of moisture content relative to optimum:

➢ Finland, Site FI.1	base 1 layer, RMC = 42% to 81% of modified Proctor OMC
	base 2 layer, RMC = 35% to 98% of modified Proctor OMC
	sub-base layer, RMC = 122% to 151% of modified Proctor OMC
➢ France, (test track)	base layer, RMC = 42% to 98% of modified Proctor OMC
➢ Iceland, Site IS.1-3 #	base 2 layer, RMC = 67% to 133% of standard Proctor OMC
	sub-base 1 layer, RMC = 44% to 131% of standard Proctor OMC
	sub-base 2 layer, RMC = 54% to 119% of standard Proctor OMC
➢ Ireland Site EI.1-2	base layer, RMC = 43% to 68% of modified Proctor OMC
	sub-base layer, RMC = 43% to 68% of modified Proctor OMC
Site EI.3	base layer, RMC = 54% of modified Proctor OMC
	sub-base layer, RMC = 54% of modified Proctor OMC
➢ Portugal, PT.1*	sub-base layer, RMC = 27% to 97% of modified Proctor OMC

\# It should be noted that high levels of moisture in the Icelandic pavements are strongly influenced by short thaw periods during winter and the spring-thaw period in April.

* measurement ranges applicable to the period of time shortly after construction.

- Pavements founded on embankments are more likely to perform better than pavements through cuttings. In pavement structures through cuttings the moisture content was found to be slightly higher, but the variation was lower, compared to the pavement structure founded on an embankment. For structures through cuttings, water has the opportunity to flow into the pavement layers particularly if deep side drains are not installed.

- The sealing of the shoulders of the pavement was found to be an important issue for consideration in trying to keep water out of the pavement for improved performance. It was noted that a moisture gradient exists between the centreline of a well-sealed pavement and near the edge of the surfacing where the shoulders were unsealed. The ingress of water into the upper granular layers, through unsealed shoulders, can occur in the form of rainfall or rapidly melting snow during the spring period. The lower pavement layers and subgrade may still be frozen when the surface snow is melting, and so the moisture becomes entrapped in the upper layers. In addition, FWD testing in northern Europe showed that higher deflections occurred near the outer edge of the pavement as opposed to measurements in the middle of the sealed pavement. This affect can result from moisture variations and also from better quality compaction toward the centre of the pavement, due to improved material support, than closer to the shoulders.

- The moisture in the pavement structure is very dependent on the:
  - precipitation levels
  - the integrity of the sealed surface
  - the final preparation applied to the shoulders of the pavement (sealed or unsealed and seal width, partial or full)
  - level of the pavement (raised pavement or pavement in cutting)
  - ability of the pavement to self drain (permeability)
  - adequacy of the pavement's drainage system

- Increased moisture in the pavement structure has a detrimental influence on the bearing capacity of the pavement. This is strongly linked to the reduced levels of resilient modulus and higher susceptibility to permanent deformations of UGM when their moisture content increases (as highlighted in §7.3 and §7.4). FWD surveys showed that in Northern countries, increases in moisture content of the granular layers were accompanied by increased pavement deflections and a reduction in strength which was most noticeable after the Spring thaw.

- The strength of the experimental pavements monitored during the project varied considerably. The Portuguese site in particular exhibited quite high deflections considering the large total thickness of the bituminous materials (295mm). However, FWD results from two sites in Ireland and one site in Finland, showed that provided sufficient depths of high quality granular material for the prevailing subgrade conditions are used, strong pavements with a long life can be constructed mainly with granular materials.

- Difficulties were encountered in the installation and operation of TDR probes on some sites. The evidence from the experimental sites suggests that more work is needed to develop equipment which can confidently monitor low moisture contents (<4%) in granular materials.

PERFORMANCE PROPERTIES OF LABORATORY TESTED UGMs:

- The simple, empirically-based classification tests were found to *aid* in material quality assessment, but fail to clearly indicate overall material quality. This can be due to:
  - the specific nature of the test type (eg, fragmentation, weathering or abrasion)
  - the limited portion of the material's parent grading curve used in the simple test
  - the severity of the conditions used in the simple test
  - the very broad range of specification limits from country-to-country

- The gyratory compaction test was found to provide a good means of assessment for the materials examined, however, the procedure was quite labour intensive.

- Static triaxial testing provides a good indication of a material's shear strength prior to RLT testing, used to determine a material's performance properties.

- Marginal-type materials could not support the high stress loading applied in the present draft of the CEN RLT Method A, which is more in-keeping with initial heavy compaction traffic loadings. At present, only high quality basecourse-type materials are capable of being assessed at the preconditioning and resilient modulus stress stage paths currently specified in the CEN procedure. A separate set of preconditioning and stress stage paths must be specified in the CEN RLT test procedure for more marginal materials, in keeping with the capability of the material to support such stresses, which are aligned with those experienced in the pavement.

- The effect of moisture content applied to a given material for a particular grading significantly effects the material performance characteristics (resilient modulus and permanent strain).

- The density of a material significantly affects the permanent strain susceptibility of UGMs, however, its effect on resilient modulus tends to be dependent on the nature of the material.

- Permanent deformation characteristics of different UGMs determining in the wheel tracking tests were comparable with those determined by RLT testing. For a very thinly surfaced unbound granular pavement, the wheel tracking tests showed between 73 to 95% of the total layer permanent deformation resulted in the upper half of the basecourse layer.

- The LCPC permanent strain model (using parameter A_1) and the permanent strain rate model (using parameter $d\varepsilon_p/dN$) provide a good indication of the strain susceptibility of UGM, *provided that the stress levels used for testing reflect those experienced in the loaded pavement*.

UGMs IN EXPERIMENTAL PAVEMENTS:

- An intermediate thickness of the bituminous layer (85mm) indicates:
  - a "limited" non-linear, load-dependent response of the flexible pavement (which can be further limited by cold test conditions causing stiffening of the bituminous layer).
  - a strong strain measurement dependence on the loading speed for speeds up to 40km/h

- A thick bituminous layer (295mm) indicates:
  - low levels of strain in the unbound layers resulting from vehicle and FWD tests, despite heavy traffic.
  - a very linear response of the pavement to different levels of load.
  - a strain measurement dependence on the loading speed for very low speeds up to 15km/h
  - good agreement between strains measured under wheel loading at high speed (40km/h) and FWD loading (65kN load).

MODELLING OF EXPERIMENTAL PAVEMENTS:

- Varying the moisture content of the granular material was found to greatly effect the response of the pavement. Under a 85mm bituminous surfacing, it was found that an increase of the UGM moisture content from 38% to 80% of OMC lead to substantial strain increases, particularly horizontally at the bottom of the bituminous layer and vertically at the top of the granular layer. When equated to pavement design life, the moisture content increase from 38% of OMC to 80% of OMC resulted in a life reduction factor of 3.5.

- Four modelling hypotheses were used to model the LCPC pavement structure experiment:
  1. all materials linear elastic;
  2. bituminous concrete and soil linear elastic, granular material described by the Boyce model;
  3. bituminous concrete linear elastic, granular material and soil described by the Boyce model;
  4. bituminous concrete visco-elastic, granular material and soil described by the Boyce model

  The results of this work showed that:
  - all modelling hypotheses showed the failure was governed by the asphalt strain criterion.
  - the linear elastic model (where the modulus of the granular material is chosen to match the measured strains in the granular layer) did not predict the expected asphalt fatigue design life very well. However, it is important to note that the response model used for calculations was a very simple, one with no sub-layering of the granular or soil layers implemented to account for the non-linearity of these materials. It is expected that much improved results, particularly for the asphalt and soil could have been expected with a

more refined model.

- linear elasticity and the "complete" model predicted the granular layer criterion quite well.
- the Boyce model, with values of parameters determined from laboratory repeated load triaxial tests, gave good predictions of the strains in the unbound granular layers applicable to the lower stress conditions existing in the surfaced pavement.
- as the response of the subgrade soil was non-linear and sensitive to the loading frequency, and the use of the Boyce model for the soil improved somewhat the prediction of the strains in the soil but could account for loading frequency induced strain variation.
- the Huet-Sayegh visco-elastic model, with parameters determined from laboratory complex modulus tests, was found to predict the strains in the bituminous concrete quite well. It was found particularly useful in determining the response of the pavement for various conditions of temperature and loading frequency.

COST-BENEFIT ANALYSIS STUDY:

A cost benefit analysis was performed to assess the utility of the findings of the COURAGE project. In the study, four thinly surfaced pavement constructions, such as might be used in different European countries, were selected. In one of these, a more marginal UGM was employed as might be recommended following the use of the testing procedures outlined in the COURAGE report, together with the analytical design techniques described in it. This analysis demonstrated that the thinly surfaced, aggregate rich pavement offers significant cost advantages for moderate traffic levels, given a thirty year assumed maintenance period before complete reconstruction is required.

12. FUTURE RESEARCH

Following the research conducted in COURAGE, the following aspects are recommended for consideration in future research proposals.

CLIMATIC EFFECTS ON UGM WITHIN PAVEMENTS:

- Seasonal monitoring of the pavement test sites where moisture measurements were conducted should be continued for at least another two years to confirm the climatic cycles in all countries.

- The evidence from the experimental sites suggests that more work is needed to develop equipment which can confidently monitor low moisture contents (<4%) in granular materials.

- Given that large moisture changes were found to occur in a number of test pavements monitored under the work of COURAGE, it is considered that research into the migratory movements of water through unbound materials should be addressed. This is particularly important as a result of the strong relationship of strength, stiffness and rutting susceptibility to the moisture content of these materials.

PERFORMANCE PROPERTIES OF LABORATORY TESTED UGMs:

- At present, only high quality basecourse-type materials are capable of being assessed at the preconditioning and resilient modulus stress stage paths currently specified in the draft CEN procedure. A separate set of preconditioning and stress stage paths need to be specified in the CEN RLT test procedure for more marginal materials, in keeping with the capability of the material to support such stresses, which are aligned with those experienced in the pavement. This will aid in the utilisation of these materials in practice.

- In keeping with the above point, the magnitude of the preconditioning stress stage need to be investigated to more closely reflect the level of stress experienced by unbound granular materials in the sealed pavement condition. This will allow the relationship between permanent deformations determined in the repeated load triaxial test to be more directly compared with those measured on experimental sites under real loading conditions.

- Once modifications to the existing draft CEN procedure occur, it is proposed that performance-based testing of 'new' alternative materials is investigated to support the initial research undertaken in the Alt-Mat Project. This will allow an assessment of the mechanical suitability of these materials for use in constructed pavements.

- The influence of fines quality (below 425 microns) on the performance of different granular materials assessed in repeated load testing and practical use in constructed pavements should be investigated. Different quality test methods should be used to describe the quality of the fines such as LL/PL, methylene blue, sand equivalent and particle size distribution.

UGMs IN EXPERIMENTAL PAVEMENTS:

- In-situ trials of thin surfaced pavements (<50mm) need to be undertaken in a planned and controlled manner to provide internal measurements of stress and strain to determine:
  - how a pavement becomes distressed

- the similitude offered by different modelling procedures (detailed linear-elasticity, FE, etc)

It is recommended that sensitivity to load be investigated and moisture levels in the unbound pavement layers be carefully monitored, using the TDR technology described in COURAGE, to facilitate correct modelling practices.

MODELLING OF EXPERIMENTAL PAVEMENTS:

- Further research needs to take place looking at the ability of different response models to accurately predict pavement performance, particularly for very thinly surfaced pavements. In addition, work needs to be undertaken to provide adequate performance models which can delimit acceptable and unacceptable pavement behaviour. This will lead on from the activities already undertaken in the COURAGE and AMADEUS projects.

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ANNEX 1 - PROJECT STRUCTURE AND DELIVERABLES

1. WORKPLAN & DELIVERABLES

1.1 Overview of the Technical Content

The COURAGE project consisted and functioned under the division of the work into five separate Work Packages (WPs) which enable milestones and progress to be monitored. Of these, the first is non-technical, being the project managerial role.

WP1: Project Administrative Management

The University of Nottingham was responsible for the project management. This work overlapped closely with the liaison role of WP5. In addition to the co-ordinating aspects described there, the managerial aspects involved:

- planning and programming of activity,
- checking progress against the project milestones and objectives,
- assessing the extent of, and quality of, deliverables which were supplied,
- corrective action including the re-allocation of work,
- financial control, distribution and invoicing,
- communication with the EU on non-technical issues, including editing and submission of bi-monthly, interim and full reports,
- setting-up and administering contractual relationships between the partners, co-ordinating their efforts,
- determining and establishing a framework for exploitation of any technically valuable findings,
- planning, conducting and record keeping of meetings,
- establishing and maintaining records.

WP2: Mechanical Characterisation of Existing Highway Materials

In this WP, conventional aggregates (such as gravel and crushed rock) were tested in the laboratory. Conventional practice is to apply simple tests to individual or a small collection of individual stones. As these yield, at best, only an index of performance, this WP concentrated on fundamental, performance-related testing of compacted granular materials in, as near as possible, their condition (density, grading and moisture content) as found in-situ. As these may be rather complex to perform routinely, simplified versions were attempted for possible commercial adoption. Some traditional testing were performed for reference purposes.

With the development of rational pavement design methods, which can incorporate realistic models of the behaviour of pavement materials, there is a need to understand better the properties of these materials, and to select or develop performance tests, which can be used in practice to determine these mechanical properties. The main objectives of this WP were:

- To define tests and test procedures suitable for determining the functional properties of unbound granular materials. (Repeated load triaxial test, and other performance tests).
- To compare the results obtained with these new performance tests with those of simpler tests, easier to perform in practice (simplified mechanical performance tests or identification tests).
- To assess, with these two test types, some granular materials, representative of those used in different European countries.

CEN, or draft CEN, laboratory tests were used wherever available.

WP3: Variability of In-situ Condition

The purpose of this WP was to increase the knowledge about the existing conditions in a pavement section through measuring and monitoring existing national test road sections, and to produce basic information for the new analytical design method (WP4).

WP3 was planned to clarify and measure the variation of moisture content in the unbound material of the pavement structure. The recent introduction of new measuring device (TDR, Time Domain Reflectometry) has made it possible to accurately monitor the moisture on granular materials in pavement sections. Up to this moment the designer has only known the moisture content during construction.

Moisture content and its variation during the time is a basic state variable of which knowledge is required to adjust the simulation and design process, thus leading to a more accurate prediction and more economical pavements. Some of the results were also used to verify the material and design models in other WPs. WP3 was mainly carried out by in-situ monitoring and measurements of the moisture content in some test road sections under national test road projects.

The main objectives of this WP were to:

- produce information about the variability of moisture content and moisture dependent material parameters needed for highway pavement design and simulation purposes.
- develop an understanding about the relevancy of moisture and material parameter variation and the scheme how they should be determined either in laboratory or in-situ.
- help to focus the attention of national road authorities to the main faults and help to modify the design and construction work if necessary.

WP4: Modelling of Pavement Structures

This WP contained the theoretical part of the investigation. The results of the laboratory tests on materials needed to be interpreted using appropriate models, describing the elastic and plastic behaviour of the unbound materials. Then, by implementing these models in a suitable computational method, it was possible to predict the contribution of the unbound granular layers to the performance of the pavement.

At present, most pavement design methods are still based on multi-layer linear elastic models (Burmister). These models are justified for rigid pavements, with bound base and sub-base materials. For flexible pavements, with unbound granular bases, and thin bituminous surface layers (<10 cm), they are not satisfactory, because they do not take into account the real mechanical behaviour of unbound granular materials. This behaviour is characterised by a marked non-linearity (dependency on stress level), and accumulation of permanent strains under repeated loads (plasticity).

In the past few years, more advanced pavement response models, based on the finite element method, have been developed; these models can incorporate more realistic constitutive models for unbound materials. In the previous "Science" project, work was carried out to improve several such finite element codes.

However, now that such numerical tools are becoming available, there is a strong need to validate them, by comparison with results from laboratory experiments, and measurements on instrumented pavements, and to define an approach for applying them to practical pavement design.

The main objectives of this WP were to:

- propose suitable models for describing the stress-strain behaviour of unbound granular materials. Several existing models were tested for this purpose, by comparisons with results of laboratory

tests from WP2.
- synthesise data from several experiments on instrumented pavements (real pavements or full-scale accelerated tests), which was used for comparisons with models.
- compare the predictions obtained using the material models, implemented in finite element codes, with the results of the in-situ experiments.
- discuss the validity of the different models, having selected those giving the best results, and make proposals for their application to pavement design.

WP5: Assessment and Dissemination

The purpose of this WP was to provide a focus for the technical activity. This WP had a central technical role in maximising the value and applicability of the research outcomes of the other packages. It assisted in defining the detailed work plan of WPs 2 and 3 to eliminate duplication of effort and/or omission of critical study elements. WP5 also established links with organisations and experts outside of the COURAGE project, drawing on their experience, publications and practice to accelerate the research. It developed the applicatory framework for the project into which the outcomes of the other WPs were fed. By this means, the results of the different strands of the research were synthesised in an agreed, appropriate manner. The WP was also responsible for preparing the reports and drafting the Practitioners' guide.

The objectives, therefore, were to ensure that:
- all the relevant areas of study were addressed in one WP or another.
- required linkages between the work of WPs 2-4 were established and maintained.
- the outputs of WPs 2-4 were appropriately synthesised.
- relevant input from sources outside the project was obtained and fed to the appropriate WP.
- Relevant project information was disseminated through the WWW.
- the outcomes of the different WPs were analysed for cost-benefit ratio.
- the outcome of the project was reported.
- a draft practitioners guide was prepared

The work of the WPs was divided between the participants as summarised in Table 1.1.

			Work Package					
Code	**Co-ordinator**		**1**	**2**	**3**	**4**	**5**	**Totals**
1	UNott	United Kingdom	8	6			4	**18**
	Partners							
2	ISTI	Portugal		9	2.5	2.5		**14**
3	VTT	Finland			3			**3**
4	LCPC	France		4.6		3.5		**8.1**
5	NRA	Ireland			2.5		3.5	**6**
	Associated Partners							
6	ZAG	Slovenia		8,4	2			**10,4**
7	Finnra	Finland			2,5		2	**4,5**
8	PRA	Iceland		1,5	1,5			**3**
9	UHannover	Germany				2,5		**2,5**
10	TEI-A	Greece					0,1	**0.1**
	Sub-Contractors							
11	UOulu	Finland		1,8*	0,6*			**2,4***
12	Scott Wilson	United Kingdom					0.75	**0,75**
		TOTAL (Man-Months)	**8**	**31,3**	**14,6**	**8,5**	**10.35**	**72,75**

TABLE A1.1: WORK PACKAGE DIVISION OF COURAGE

WP Leaders are marked by shaded boxes. *Approx figures, work done on a per test basis.

The purpose of this breakdown was to provide separate arms for the management/co-ordination, dissemination of project findings with the technical aspects of the project. In addition, particular leaders were allocated work based on their relevant expertise. Also, topics were separated to allow as much parallel working as possible. The schedule of activities of the Work Packages and their interrelationships are illustrated in Figure 1.1. As far as possible, each WP's programme was designed not to be dependent on progress in another WP. Nevertheless, a good deal of inter-connection between the different tasks was required in order that the project arrived at useful outcomes derived from all WPs.

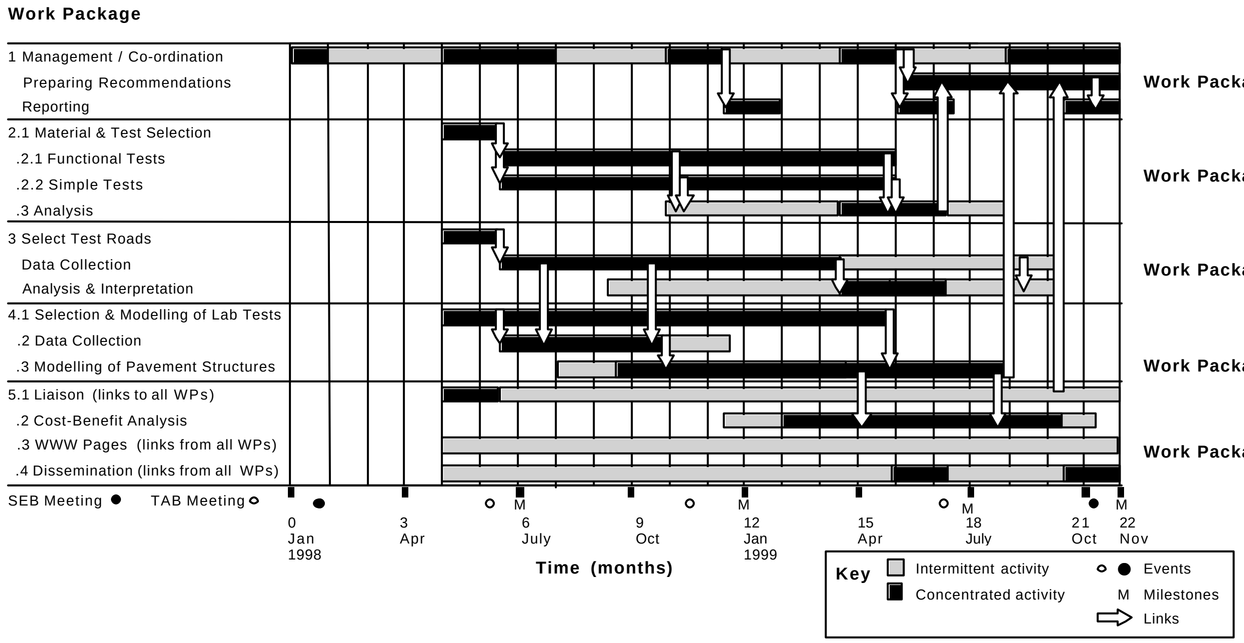


FIGURE A1.1: SCHEDULE OF ACTIVITIES OF THE WORK PACKAGES AND THEIR INTER RELATIONSHIPS

1.2 Technical Deliverables

- A database of granular material performance obtained from both laboratory and in-situ testing.
- Demonstration cases of improved granular materials - partly incorporating novel material.
- An analytical procedure for the design and assessment of pavements comprised largely of granular material layers.
- A draft of a guide for practising engineers to the best means of testing, modelling and incorporating unbound granular materials (conventional and secondary) into pavements.
- A WWW site giving details of the research project, its findings, contact information and pointers to further information.
- A seminar, jointly with COST337, at the end of the contract to publicise the work and stimulate implementation.
- Technical papers submitted to professional journals.

1.3 Contract Deliverables

- An interim report available after 17 months summarising
- a review of the state-of the art.
- the adopted direction and research strategies of the project as determined at the beginning of the project.
- a summary of progress to date.
- a final report containing:
  - ⇒ Recommendations for both simplified and performance-related procedures for laboratory

test procedures/equipment capable of determining the mechanical behaviour of all granular materials, with an indication of their relationship.

⇒ A classification system for unbound granular materials for structural purposes.

⇒ An assessment of the effects of site and climatological variability on in-situ condition and, hence, on performance.

⇒ Recommendations for constitutive models of mechanical behaviour of unbound granular materials based on both laboratory and field assessments.

⇒ Recommendations for the better methods of use of both conventional and secondary unbound granular materials in pavements (especially in thinly surfaced pavements) by an improved incorporation of behaviour into pavement design.

⇒ An assessment of the benefits of the implementation of the findings.

⇒ A plan for a post-contract conference session UNBAR5 (5th International symposium on UNBound Aggregates in Roads) to be held in June 2000 at the University of Nottingham.

2. PROJECT MANAGEMENT STRUCTURE

2.1 Overall Management

The overall co-ordination of the project was by the University of Nottingham. One representative of the co-ordinator and each of the full partners (plus a secretary) formed the Scientific & Executive Board (SEB). They were also members, together with one representative of each of the associate partners, of the Technical Advisory Board (TAB). Supplementary membership of the TAB by invited observers who were able to contribute expertise was also envisaged.

Scientific & Executive Board (SEB)

The SEB was chaired by the project co-ordinator. It had two roles:

a) Management: In this role it was the chief decision making body. It was responsible for the general administration of the project, provided support to the project co-ordinator and was responsible for maintaining the programme and adjusting activities to ensure this. In addition it was responsible for making appropriate legal arrangements concerning the development of Intellectual Property developed as a consequence of the project and of securing optimum dissemination of the findings. It formed the link with DGVII.

b) Scientific: In this role the Board was responsible for the overall direction of the project, determining the main technical agenda, maintaining the scientific rationale and ensuring technical targets were met in each WP.

The members of the SEB each chaired sub-committees, one for each technical WP (Nos. 2 to 5).

Technical Advisory Board (TAB)

The purpose of the TAB was to ensure a wide discussion of the research programme, purpose, method and findings. To achieve this, all partners (full and associate) were represented by one member together with specific research staff who may have been invited as the need arose. In addition, it was planned to hold most of the TAB meetings contiguously with COST 337 management committee meetings from which all members were invited to attend the TAB.

The meetings were held so as to provide relevant background and technical direction at various stages during the project. Meetings were held at the commencement of the project to review progress; at the mid-point to review the findings, suggest conclusions and dissemination plans as the project reached its last month or so and, finally to provide comments on the Final Report and

Guide just prior to project completion. Thus, the TAB served the function of a semi-independent review body to maximise the project outcomes.

Technical Sub-Committees for WPs

Only WP2 was large enough in its work effort to warrant a sub-committee which consisted of the WP leader in the chair with one member from each organisation participating in that WP. It met as determined by the members of the sub-committee. The other WPs used telephone conferencing, e-mail, and meetings held contiguously with main Board meetings.

3. DURATION

The estimated duration of the COURAGE project was 18 months (January 1998 to October 1999).

4. ESIMATED COSTS

The estimated cost of the COURAGE project was 532,388 ECU.

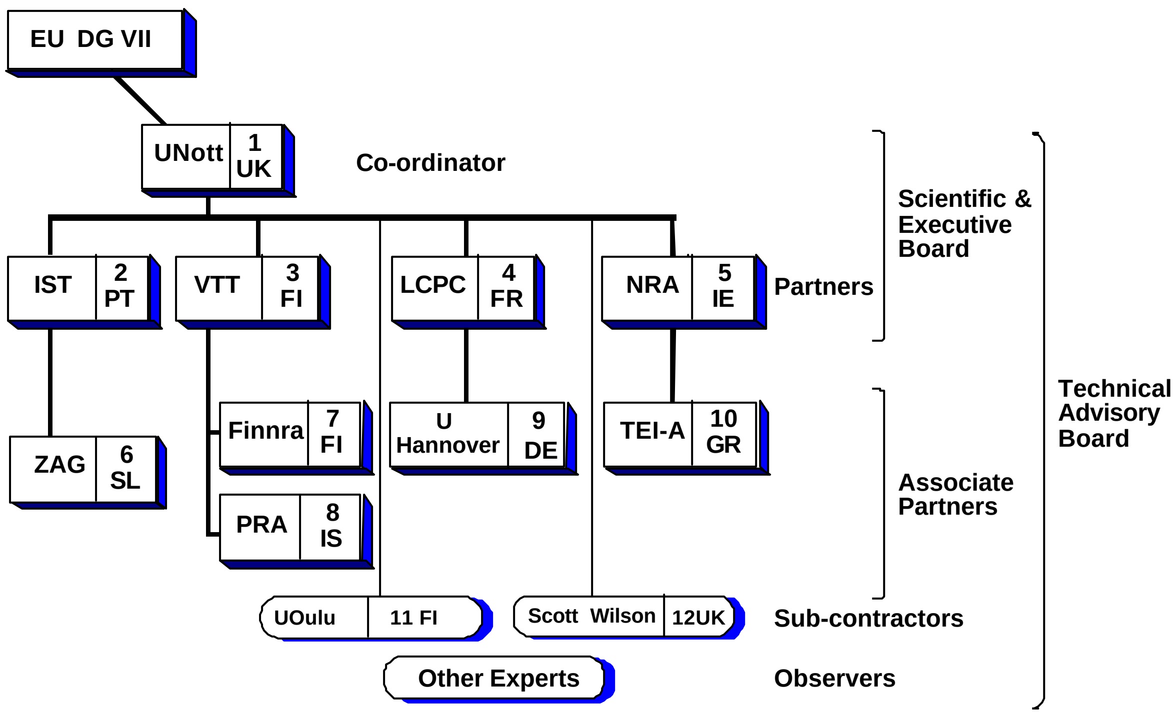
ANNEX 2 - COURAGE MANAGEMENT COMMITTEE MEMBERS

THE PARTNERSHIP

A Consortium of four Partners, five Associate Partners and two Contractors from nine European countries carried out the COURAGE project under a central point of co-ordination. The twelve Organisations involved and their participation status in the project were as follows:

Co-ordinator	University of Nottingham – United Kingdom
Partners:	IST – Portugal
	VTT - Finland
	LCPC – France
	NRA – Ireland
Associate Partners:	ZAG – Slovenia
	Finnra - Finland
	PRA - Iceland
	University of Hannover – Germany
	TEI-Athens – Greece
Contractors:	UOulu – Finland
	Scott Wilson - Basingstoke, United Kingdom

The Relationships between Partners, Associate Partners and Contractors is given:



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ANNEX 3 - GLOSSARY

ALT-MAT	ALTernative MATerials in road construction
AMADEUS	Advanced Models for Analytical Design of European pavement Structures
ASTM	American Society of Testing Materials
CBR	Californian Bearing Ratio
CCP	Constant Confining Pressure
CEN	European Committee for Standardisation
COURAGE	COnstruction with Unbound Road AGgregates in Europe
DSC	Dutch Static Compression
EC	European Commission (Brussels)
FEHRL	Forum of European National Highway Research Laboratories
FINNRA	Finnish Road Administration (Finland)
FWD	Falling Weight Deflectometer
IST	Instituto Superior Técnico (Lisbon, Portugal)
LCPC	Laboratoire Central des Ponts et Chausées (Nantes, France)
LA	Los Angeles abrasion
LL	Liquid Limit
MDD	Maximum Dry Density
MDE	Micro Deval
NPV	Net Present Value
NRA	National Roads Authority of Ireland
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plastic Limit
PRA	Public Roads Administration (Iceland)
RCC&A	Recycled Crushed Concrete and Asphalt
RLT	Repeated Load Triaxial
RTD	Road Transport Development
RMC	Relative Moisture Content to OMC
SEB	Scientific & Executive Board
TAB	Technical Advisory Board
TDR	Time Domain Reflectometer
TEI-A	Technical Institute – Athens
TSA	Transport SA
UGM	Unbound Granular Materials
UNott	University of Nottingham (United Kingdom)
UWV	Ultrasonic Wave Velocity
VCP	Variable Confining Pressure
VOC	Vehicle Operating Costs
VOT	Value of Time
VTT	Technical Research Centre of Finland
WP	Work Package
ZAG	Slovenian National Building & Civil Engineering Institute

ANNEX 4 - TEST METHODS

This annex contains the following test methods:

4.1 Repeated Load Triaxial (RLT)
4.2 Gyratory Compaction (PCG)

4.1 Repeated Load Triaxial

A diagramatic representation of the test equipment used for performing the VCP RLT test is shown in Figure A4.1.

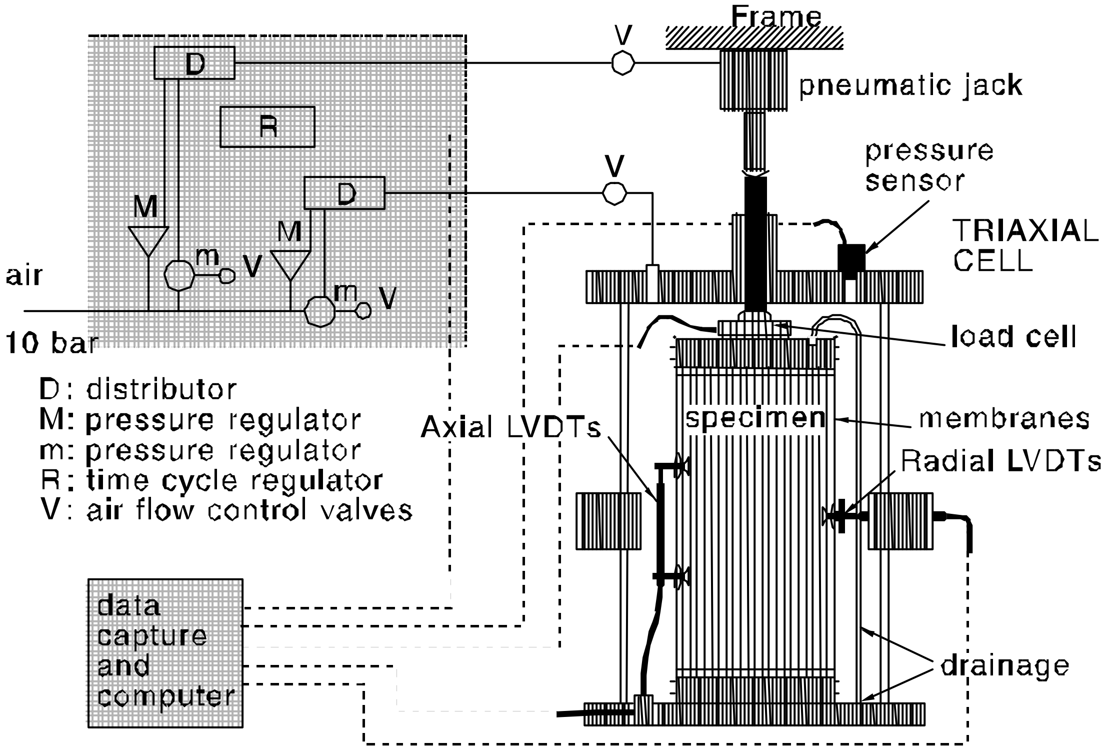


FIGURE A4.1: VARIABLE CONFINING PRESSURE TRIAXIAL APPARATUS (VCP)

In general, test specimens are manufactured to the desired density and moisture content conditions required.

PRECONDITIONING TEST:

Test specimens are subjected to a preconditioning test of 20,000 cycles involving an applied loading of p = 300kPa and q = 600kPa (or σ_1 = 700kPa and σ_3 = 100kPa).

RESILIENT MODULUS STRESS STAGE TEST:

Following the preconditioning test, the material specimens are subjected to stress stage resilient modulus test of 100 cycles per stage, with a range of applied loading sequenced according to the CEN procedure with stress ratio loading paths of q/p = 0, 0.5, 1.0, 1.5, 2.0 and 2.5 performed in sequence.

In addition, CEN test method Method B (constant confining pressure) is used only for the condition of ρ_d = 97%$\rho_{d\ OPM}$ and w = w_{OPM} – 2%. The stress paths of Method B follow a separate set of stress levels to Method A and are applied on the same specimen. These stress levels are much reduced in magnitude compared to those of Method A, in keeping with material placed in the lower levels of a pavement structure.

A plot of the stress paths for each of the CEN Method A and Method B tests is illustrated below in Figure A4.2.

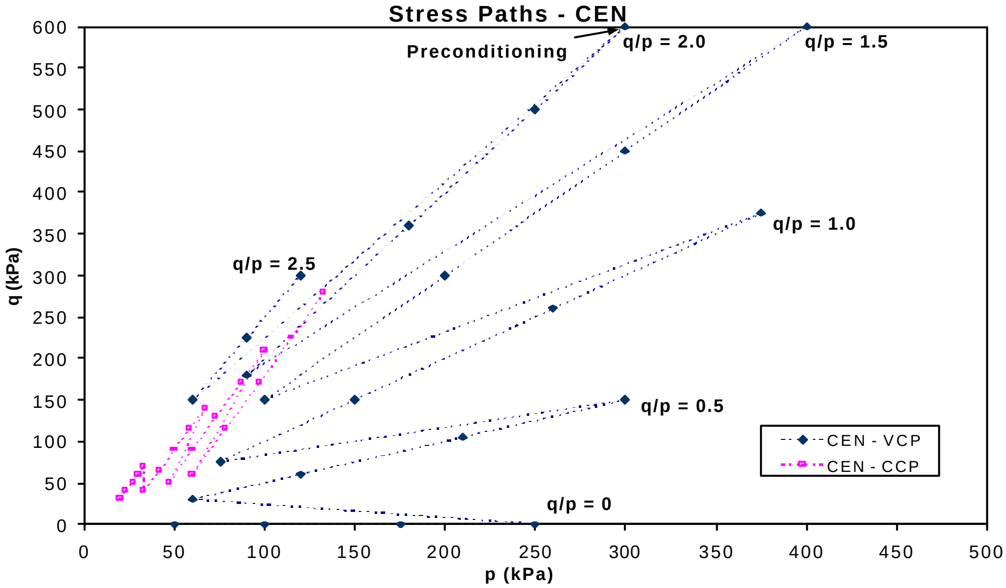


FIGURE A4.2: CEN RLT STRESS PATHS (METHOD A AND METHOD B)

In order to directly compare the resilient modulus of different materials, a 'characteristic' stress level has been chosen at which to report the elastic 3D Hooke Law modulus. This stress level of applied loading is p = 250kPa and q = 500kPa (or σ_1 = 583kPa and σ_3 = 83kPa).

The permanent strain component of the CEN RLT test procedure was not performed for any of the materials examined in this project.

4.2 Gyratory Compaction Test

1. SCOPE

This procedure covers the attrition that results from the preparation of a material using the gyratory compaction apparatus. Whilst the material is statically compacted in the mould at an angle to the vertical, a gyratory force is applied such that the axis of the applied load generates the surface of a cone.

2. SAMPLE

The sample shall be obtained and prepared as follows:

(a) Obtain by riffling or splitting the sample, a representative sub-sample of about 27kg of dry material for testing. All material shall be oven dried initially

(b) Fractionise the material into portions.

(c) Prepare six 4.5kg dry samples of the material, each made up *separately* by combining the material fractions according to the following grading. The grading curve used for all materials in this test will be the control grading.

(d) Prepare each sub-sample of the material to optimum moisture content. Thoroughly mix each sub-sample and place it in a separate sealable container. Allow each sub-sample to cure for an appropriate time for 24 hours.

3. PREPARATION OF TEST SPECIMENS

(a) Place 4.000kg of cured material, in total, into a 150mm diameter by 175mm height gyratory compaction mould. This could be achieved by placing approximately three separate additions of material using a scoop, being careful to ensure segregation does not occur.

(b) Roughly level the surface of the material with a suitable tool after each addition takes place.

Weigh the mould assembly plus the wet material.

4. PROCEDURE

4.1 Specimen assembly

(a) Place the top platen on the surface of the material.
(b) Place the mould in the mould cage and clamp in position.
(c) Set up gyrocompactor to 1.25 degrees of axial offset
(d) Use the gyratory compactor software to set the rotation speed to 30 revolutions per minute.
(e) Set the input information as:
- 150mm diameter mould and compaction stress to 0.6MPa

(f) Set the compactor to compact the material to the target number of gyrations.

4.2. Testing

For each sub-sample, perform:
(a) 10, 50, 100, 150, 200 and 300 revolutions of the sample in the gyratory compactor.
(b) Remove the sub-sample from the compaction mould and measure the specimen height (mm) at three points around the compacted sample and diameter. Report if the compacted specimen varies by more than 2mm. Record the measurements made.
(c) Gently pry the compacted specimen apart into very small clusters in a large oven tray. This will minimise further breakdown of the material when trying to pry apart the cemented block of material (once oven dried), prior to wet sieving.
(d) Place the specimen in an oven set to 105^{o}C and allow the material to dry.
(e) Once dry, allow the specimen to cool and then determine its dry mass.
(f) Calculate the moisture content of the material and the compaction dry density.
(g) Perform wet sieving of the material on a nested set of 9.5mm, 2.0mm, 0.600mm and 0.075mm sieves, in order to thoroughly break-up the cemented material and fully separate the individual grain-sizes.
(h) Re-dry the separated material grain-size components in an oven set to 105^{o}C
(i) Once the material has cooled, perform a dry sieve using the sieve sizes employed originally to prepare the reference grading.
(j) Determine the dry masses of each material fraction retained. Take into account the fines passing the 0.075mm sieve which were washed away during the wet sieve.

5. REPORTING

(a) Plot the grading curve for each sub-sample tested at a different number of gyrations as well as the reference grading curve.
(b) Determine the % increase in the amount material passing each sieve size.
(d) Plot the % increase in the amount material passing each sieve size against number of revolutions.

ANNEX 5 - MODELLING

A5.1 Theoretical Modelling

This annex contains supplementary information regarding the theoretical models used in COURAGE.

A5.1.1 Isotropic Boyce Model

This non-linear elastic model describes the resilient behaviour of unbound granular materials. The Boyce model derives from an elastic potential W, expressed by:

$$W = \frac{p^{n+1}}{p_a^{\,n-1}}\left[\frac{1}{(n+1)K_a} + \frac{1}{6G_a}\left(\frac{q}{p}\right)^2\right]$$

where K_a, G_a and n are the 3 parameters of the model ($K_a > 0$, $G_a > 0$ and $0 < n < 1$). $p_a = 100$kPa.

This leads to the following stress-strain relationships for the volumetric and shear strains:

$$e_v = \frac{1}{K_a}\frac{p^n}{p_a^{\,n-1}}\left[1 + \frac{(n-1)K_a}{6G_a}\left(\frac{q}{p}\right)^2\right] \quad \text{and} \quad e_q = \frac{1}{3G_a}\frac{p^n}{p_a^{\,n-1}}\frac{q}{p}$$

The Boyce model can be expressed in terms of bulk moduli and shear moduli (refer to §5.2.2.1).

A5.1.2 Anisotropic Boyce Model

As mentioned in §6.2.2.2, when anisotropy was introduced into the model, the expression of the elastic potential proposed by Boyce was modified by multiplying the principal stress σ_1 by a coefficient of anisotropy γ. The elastic potential then becomes:

$$W = \frac{\left[\frac{\gamma\sigma_1 + 2s_3}{3}\right]^{n+1}}{p_a^{\,n-1}}\left[\frac{1}{(n+1)K_a} + \frac{1}{6G_a}\left(\frac{\gamma\sigma_1 - \sigma_3}{\frac{\gamma s_1 + 2s_3}{3}}\right)^2\right]$$

After some simplifications, this leads to the following stress-strain relationships:

$$\varepsilon_v{}^* = \frac{1}{K_a}\frac{p^{*n}}{p_a^{\,n-1}}\left[1 + \frac{(n-1)K_a}{6G_a}\left(\frac{q^*}{p^*}\right)^2\right] \quad \text{and} \quad \varepsilon_q{}^* = \frac{1}{3G_a}\frac{p^{*n}}{p_a^{\,n-1}}\frac{q^*}{p^*}$$

The anisotropic Boyce model can be expressed in terms of bulk moduli and shear moduli (refer to §5.2.2.2).

ANNEX 6 - SUMMARY RESULTS

This annex contains the following data tables:

A6.1 Repeated Load Triaxial - Resilient Modulus Modelling Results

This annex contains the following design charts:

A6.2 Repeated Load Triaxial - Resilient Modulus Design Charts

A6.1 RLT Resilient Modulus Modelling Results

Extensive modelling of the resilient behaviour of the three principle granular materials studied in COURAGE, the gneiss, the limestone and the granite, was conducted using the anisotropic Boyce model and the Dresden model (presented in Chapter 5). The objective is to evaluate the ability of the models to describe the resilient behaviour of the materials, and analyse the parameters obtained.

It should be noted that the measure of fit is defined as:

$$r = 1 - \sqrt{\frac{1}{2}\left\{\left[\frac{\Sigma\left(e_{vmeas} - e_{vcalc}\right)^2}{\Sigma\left(e_{vmeas} - meane_{vmeas}\right)^2}\right] + \left[\frac{\Sigma\left(e_{qmeas} - e_{qcalc}\right)^2}{\Sigma\left(e_{qmeas} - meane_{qmeas}\right)^2}\right]\right\}}$$

A value of ρ greater than 0.7 indicates a satisfactory level of fit.

A6.1.1 Anisotropic Boyce Model

A6.1.1.1 Gneiss

There were 3 series of tests, for three different grading curves, with different percentages of fines (3%, 7% and 10%). For the anisotropic Boyce model the levels of fit were satisfactory; the measure of fit, ρ ranging between 0.718 and 0.889, with a mean value of ρ = 0.821.

The first test, n°9-007-1a, was performed at a moderate moisture content w = 4.1% (RMC = 68% of OMC) and the fit was very satisfactory (ρ = 0.884). The second test, n° 9-012-1a, was performed at a low moisture content w = 2.3% (RMC = 38% of OMC); in this second test, the behaviour was somewhat different: the volumetric strains depended much less on the stress path q/p applied, and the fit was not as good (ρ = 0.730). Another particularity of this second test was the high value of the coefficient of anisotropy γ, which was equal to 1.76 (it is considered that a value higher than 1 corresponds to a higher modulus in the horizontal direction). This value appears questionable, for two reasons:

- the specimens of unbound granular material are compacted using a vertical load, and this generally leads to a higher stiffness in the vertical direction (γ ≤ 1); values of γ higher than 1 are very seldom obtained;
- In addition, when one examines the strains obtained in this test for isotropic loadings (with q/p = 0, see Figure A6.1), it appears that the shear strains $\varepsilon_q = (\varepsilon_1 - \varepsilon_3)$ are practically equal to 0, which corresponds to isotropic behaviour.

So, it seems that this value γ > 1 given by the fitting method is not indicating anisotropic behaviour, and should be taken with caution. More generally, it can be noted that for the gneiss, for all the tests performed at the lowest moisture content (approx. RMC = 38% of OMC), the values of γ are higher than 1, and the measure of fit values are lower than for the other moisture contents (ρ between 0.718 and 0.790).

Figure A6.1-A6.6 presents variations of the anisotropic Boyce model parameters obtained for the gneiss with the moisture content of the specimens, for the three gradings of the material. The following trends can be observed for the gneiss:

- For the two gradings with 7% and 10% of fines, the results obtained are fairly similar. The behaviour of the material is sensitive to moisture content, and the parameters K_a and G_a decrease when w increases (the decrease of G_a is the most important).
- For the grading with 3% fines, the values of the model parameters are significantly different: K_a increases when w increases, and G_a decreases slightly.
- The parameter γ also changes with the moisture content, for all three gradings. For low moisture contents (approx. RMC = 38% of OMC), γ is significantly higher than one (1.4 to 1.7) as already noted; for higher moisture contents (RMC = 68% to 83% of OMC), γ is close to one, and the behaviour is isotropic.

A6.1.1.2 Granite

There were again 3 series of tests, for three different gradings with 3%, 5% and 10% of fines. For the anisotropic Boyce model, the levels of fit were generally satisfactory, with measure of fit values, ρ ranging between 0.676 and 0.903, with a mean value of ρ = 0.800.

For the granite, the results appeared to be more scattered than for the gneiss, and there was no clear difference between the results obtained for the control and fine gradings. Figure A6.1-A6.6 presents variations of the anisotropic Boyce model parameters obtained for the granite with the moisture content of the specimens, for the three gradings of the material. The following observations can be made:

- The parameter K_a varies very little with moisture content;
- G_a decreases by about a factor of 2 when the moisture content increases from RMC = 38 to 70% of OMC (with some scatter) for the control and fine gradings.
- Small increases in K_a and G_a occur with increasing moisture content for the coarse grading.
- The parameter of γ decreases when the moisture content increases (as for the gneiss), but always remains less than 1. It varies from about 0.95 at RMC = 38% of OMC to 0.65 at RMC = 70% of OMC.

A6.1.1.3 Limestone

There were 3 series of tests, for three different grading curves, with different percentages of fines (3%, 7% and 10%) and two additional tests for an initial production grading, with 6.3% of fines. For the anisotropic Boyce model, the levels of fit were satisfactory for most tests (with measure of fit values, ρ between 0.737 and 0.859). However, a small number of tests performed on specimens with a very low moisture content (RMC = 12 to 20% of OMC) produced poor fits and unrealistic values of model parameters.

The "problematic" tests include:

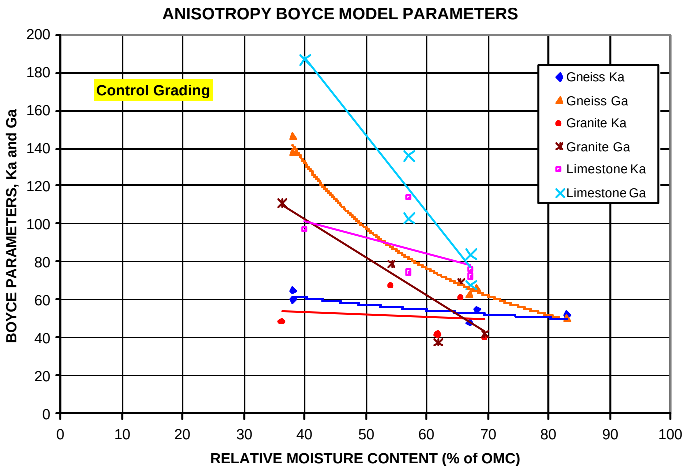
- four tests performed with the control grading (7% fines) and the lowest moisture content, (RMC = 12% of OMC) which, despite being performed on identical specimens, lead to values of the Boyce model parameters which are very different.
- two tests performed with the grading with 10% of fines, and the lowest moisture content, (approx. RMC = 20% of OMC), which give poor levels of fit (ρ = 0.624 and ρ = 0.629), and unrealistic values of the parameter n (n = 0.001 and n = 0.148 when n is usually greater than 0.2).

The behaviour of these dry specimens presents two particularities, which can explain the difficulties encountered with the fit:

- a high stiffness of the material, leading to a loss of accuracy in the strain measurements, particularly for the shear strains ε_q. For some stress paths, the measured values of ε_q do not exceed $2x10^{-4}$ when the accuracy of the measurements is approximately $0.5x10^{-4}$.
- a very high anisotropy, characterised by negative values of ε_q for all stress paths with stress ratios q/p lower than 2.

Figure A6.1-A6.6 presents variations of the anisotropic Boyce model parameters obtained for the limestone with the moisture content of the specimens, for the three gradings of the material. The trends are similar to those obtained for the granite:

- There is no significant difference between the results obtained for the three gradings.
- The parameter K_a does not change much with moisture content.
- G_a decreases largely when the moisture content increases, from about 190MPa (RMC = 40% of OMC) to 75MPa at RMC = 68%OMC for the control grading, with a similar trend but small change occurring for the coarse grading.
- anisotropy is significant. The parameter γ is larger than 1 at less than RMC = 40% of OMC (at RMC = 12% of OMC there is a lot of scatter) and decreases to about 0.6 at RMC = 68%OMC for the control grading.



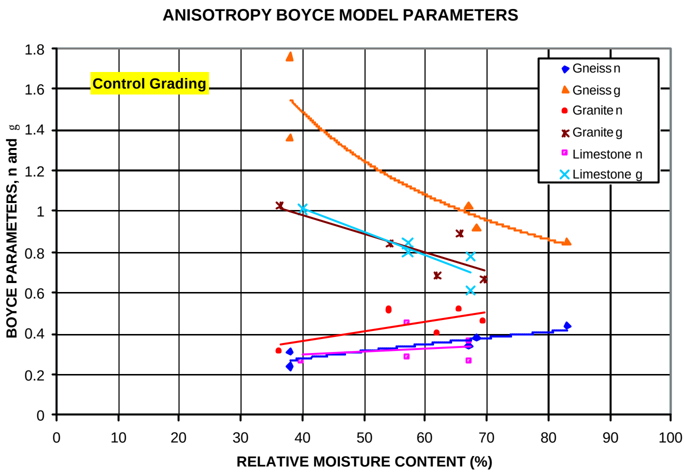
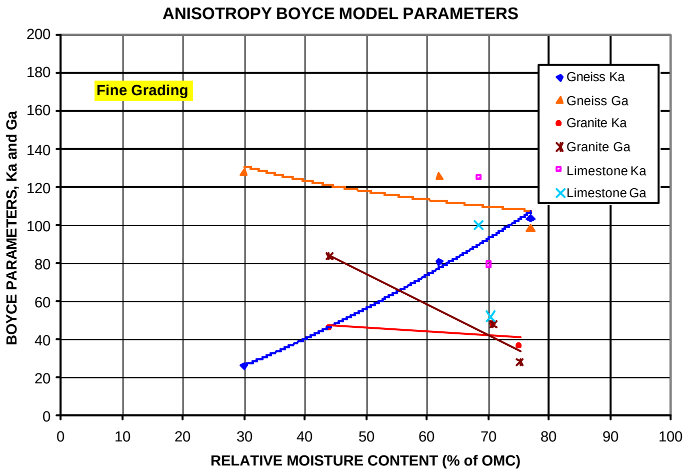


FIGURE A6.1 & A6.2: VARIATIONS OF THE ANISOTROPIC BOYCE MODEL PARAMETERS OF THE GNEISS, GRANITE & LIMESTONE WITH MOISTURE CONTENT - CONTROL GRADING



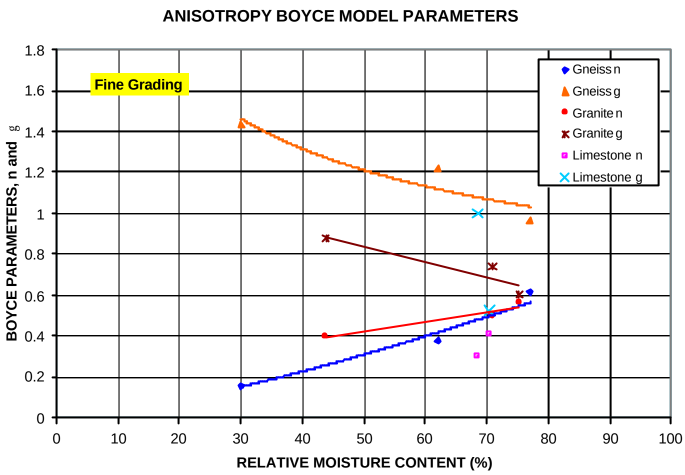
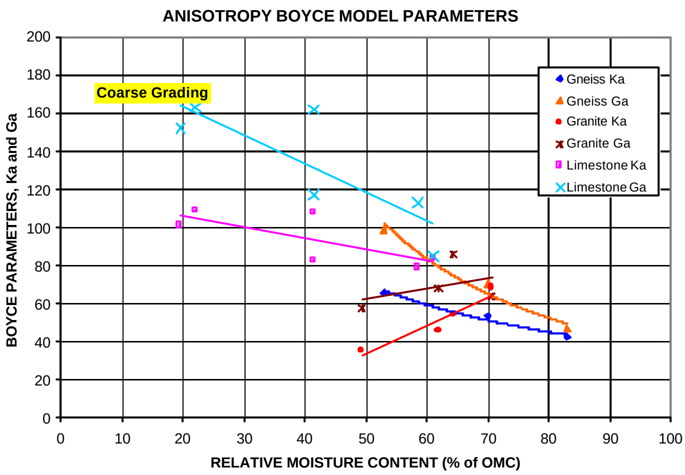


FIGURE A6.3 & A6.4: VARIATIONS OF THE ANISOTROPIC BOYCE MODEL PARAMETERS OF THE GNEISS, GRANITE & LIMESTONE WITH MOISTURE CONTENT - CONTROL GRADING



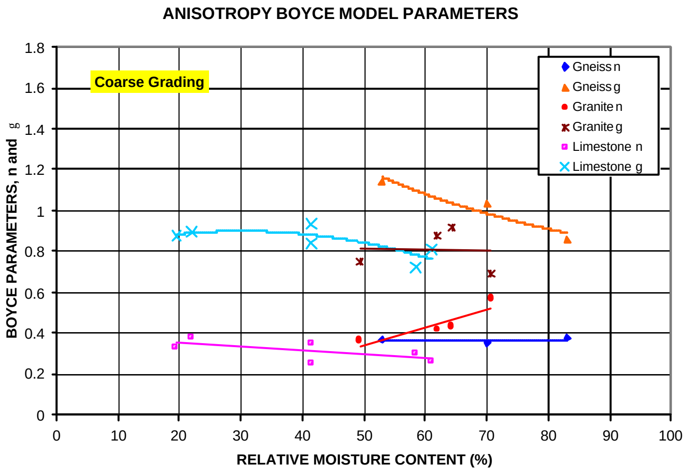


FIGURE A6.5 & A6.6: VARIATIONS OF THE ANISOTROPIC BOYCE MODEL PARAMETERS OF THE GNEISS, GRANITE & LIMESTONE WITH MOISTURE CONTENT - CONTROL GRADING

A6.1.2 Dresden Model

A6.1.2.1 Gneiss

The Dresden model provided levels of fit which were slightly lower than the Anisotropic Boyce model. The measure of fit values ranged between 0.761 and 0.866, with a mean value ρ = 0.795.

As with the Boyce model, the Dresden model predicted the behaviour of the material for different stress ratios q/p quite well. For test 9-007-1a, the measure of fit was ρ = 0.785; for test 9-012-1a, ρ = 0.797. For this second test, the measure of fit was better than with the Boyce model, and the model leads to a practically isotropic response of the material (zero shear strains for isotropic loading), in good agreement with the experimental results.

A6.1.2.2 Granite

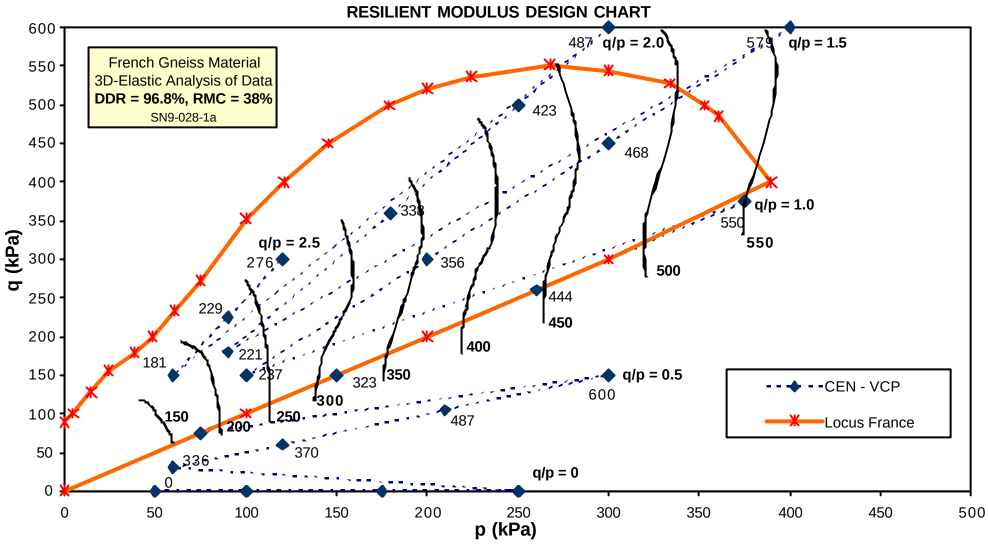
For the granite, the Dresden model provided fitting values which were a little lower than the Anisotropic Boyce model: two tests (n°090299 and n°240599) led to poor fits, with measure of fit values, ρ lower than 0.6. In the other tests, the values of ρ ranged between 0.616 and 0.867, with a mean value ρ = 0.748

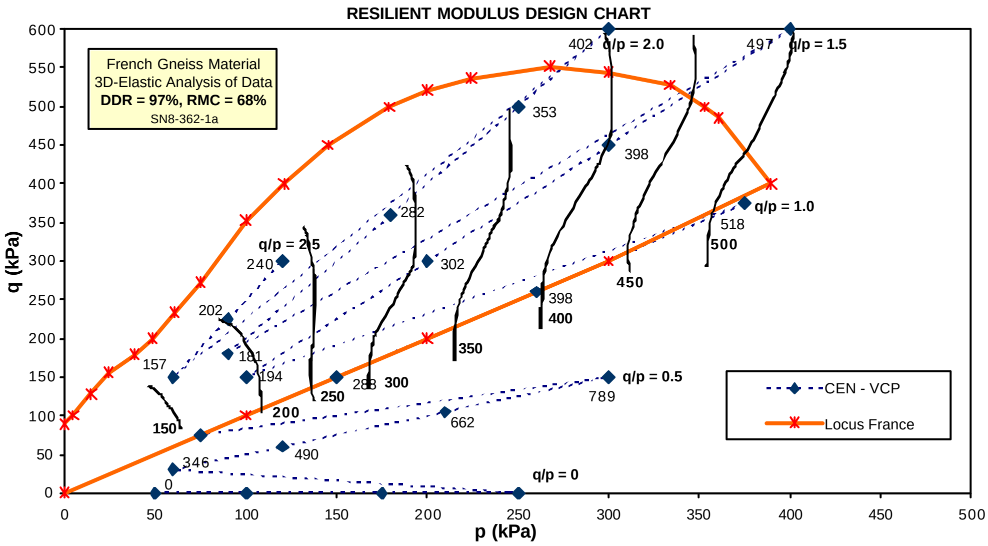
A6.1.2.3 Limestone

For the limestone, the Dresden model gave a generally poor fit, with measure of fit, ρ ranging between 0.269 and 0.816, with a mean value ρ = 0.608, and 11 tests where ρ is lower than 0.6. These less satisfactory results obtained with the Dresden model for the limestone can probably be attributed to the markedly anisotropic behaviour of this material, which the Dresden model does not take into account.

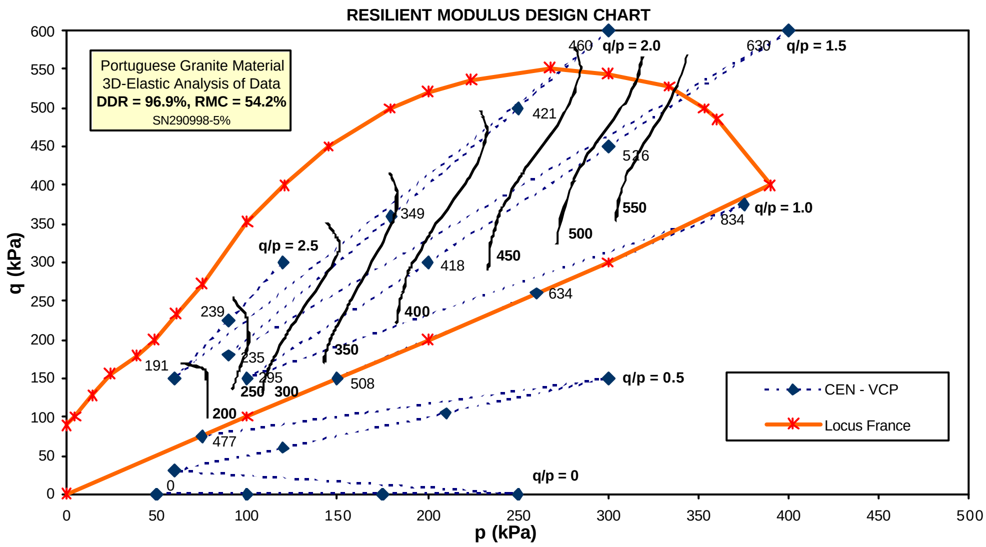
A6.2 Repeated Load Triaxial - Resilient Modulus Design Charts

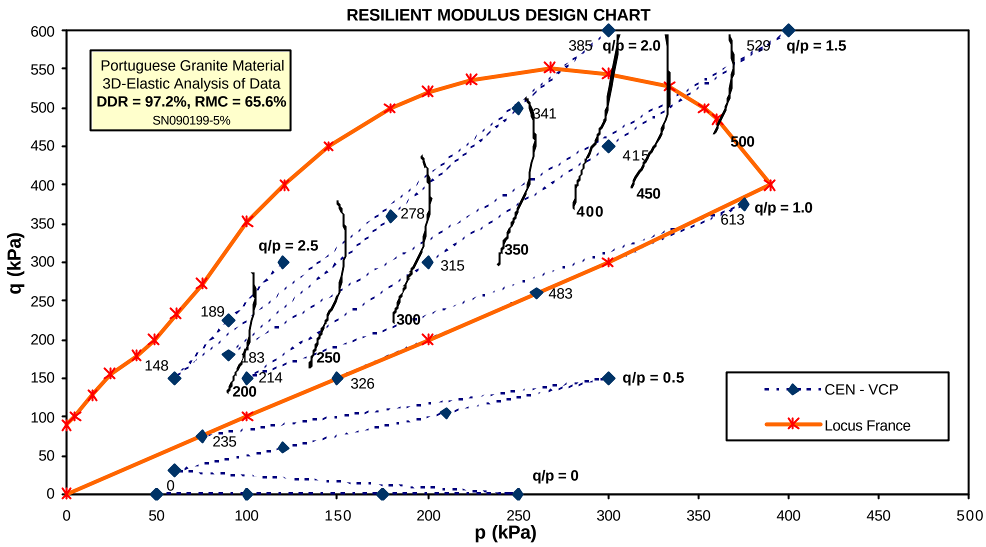
(1) GNEISS MATERIAL



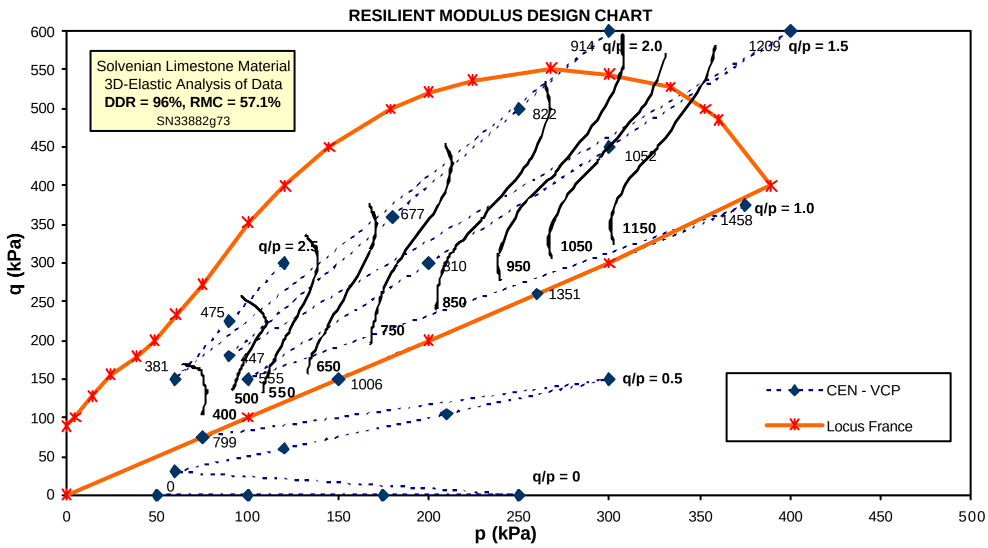


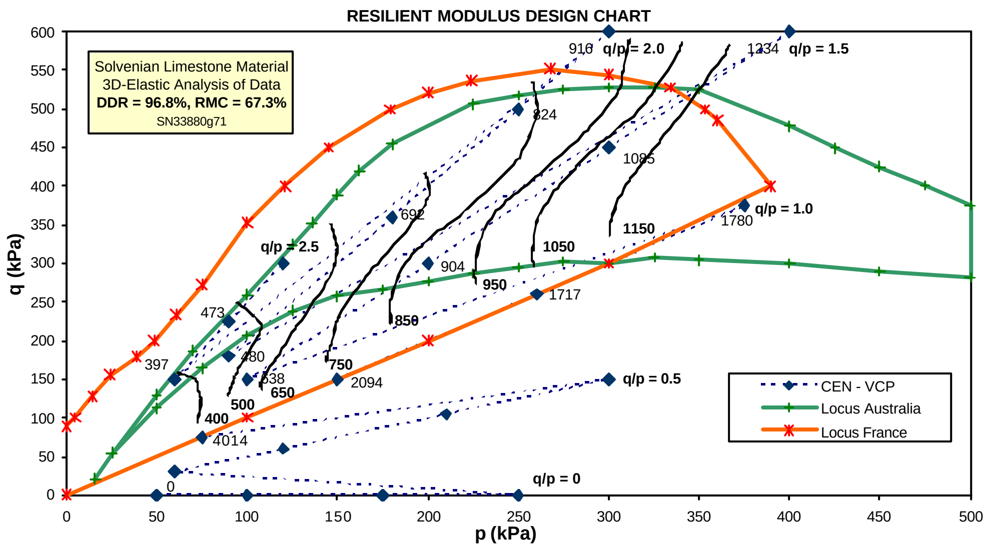
(2) GRANITE MATERIAL





(2) LIMESTONE MATERIAL





ANNEX 7 - DRAFT PRACTITIONERS GUIDE

7.1 Introduction

There is a need to prepare a Practitioner's Guide documenting "best practice" for maximising the use of granular materials in road pavement design and construction, in view of the latest findings from performance testing of granular materials in the laboratory and also from site performance. The improved capability of laboratory tests to predict the performance of unbound granular material along with the greater availability of pavement monitoring equipment, such as the Falling Weight Deflectometer, used to measure the in-situ strength of pavements, now offers the pavements industry greater possibilities. The design of pavements, their performance monitoring and maintenance scheduling can be treated in a manner somewhat different to that which road engineers have traditionally used in the past.

This Draft Practitioners Guide provides the *outline* of the main areas, which need to be addressed in preparing such guidelines. The preparation of such a detailed document requires a great deal of additional work beyond this project, but the main headings for each area which need to be addressed are outlined below.

In providing guidance on the design, construction and maintenance of any road, including those constructed mainly with granular materials, there are certain basic decisions which have to be taken which relate to:

- the importance of the road and the level of serviceability expected by the road user,
- the expected life span of the pavement,
- the level of funding available,
- the quality of locally available unbound granular materials.

All of the above influence many of the detailed engineering decisions which impact on the main areas of pavement design, materials specification, and construction and maintenance of the road pavement.

In the COURAGE Project, the experimental sites used for monitoring in-situ conditions included roads with traffic levels of the order of 400-500 vehicles per day to more heavily trafficked roads with traffic levels of the order of 10,000 vehicles per day or greater. The level of expectation from such roads, at opposite ends of the traffic spectrum, are generally different and this must be reflected in the engineering decisions taken at various stages of the life of the road.

For most roads, the unbound granular materials to be used in the pavement layers can be broadly classified as follows:

CLASS 1:
This material is of the highest quality and must be able to withstand high stresses without exhibiting sufficiently large strains to cause significant permanent deformation in the pavement layer(s). In addition, these materials should possess high levels of resilient modulus in the upper layers of unbound granular pavements where the materials are subjected to very high stresses resulting from traffic loadings.

CLASS 2:
This material is of lesser quality than that outlined above, but must be able to withstand lower stress levels than Class1 without exhibiting large strains. These materials are more likely to be used in the

sub-base layers of heavily trafficked pavements, or in the base of lightly trafficked roads.

CLASS 3:
This material is of lower quality than Class 2 and should be used in the lower pavement layer as a capping, fill or subgrade improvement layer. At this level in the pavement, the material will be subjected to very low stresses and it must withstand these stresses without exhibiting large strains.

All three Classes of material must be able to fulfil their function under the prevailing environmental conditions, which vary from one climatic region to another, and the detailed specifications must reflect these requirements. The guidelines must specify the tests and material state conditions needed to satisfy all of the above requirements and the timing of such tests. It must also specify any extra tests which may be required to cater for the differences between natural aggregates and alternative materials, or materials which need further treatment or processing to bring them up to the standard necessary to be included in any of the above Classes.

The main headings which should be included in a draft Practitioners Guide are outlined below.

1. MATERIALS TESTING

- *ALL MATERIAL CLASSES, CONSIDERING THEIR ROLE AND FUNCTION IN THE PAVEMENT*
  How to measure strain susceptibility, strength, stiffness, cohesion and permeability of natural and alternative materials - equipment, test conditions, etc.

2. PAVEMENT DESIGN

- *SUBGRADE*
  Appropriate strength/stiffness values during life of road - for Northern and South European countries.

- *SUB-BASE LAYER*
  Mechanical Performance Parameters.

- *UNBOUND BASE*
  Mechanical Performance Parameters.

- *SURFACING*
  Mechanical Performance Parameters.
  Type and thickness for different traffic levels.

- *CHOICE OF ROAD CROSS-SECTION*
  Lane width, shoulder width, type of surfacing applied to shoulders.

- *DRAINAGE*
  Type and location of drains. Maintenance of drains.

- *PAVEMENT DESIGN METHODS*
  Design procedures/approaches for BST surfaced granular pavement, thinly surfacing asphaltic granular pavements and thick bituminous granular pavements.

3. SPECIFICATION OF CONSTRUCTION MATERIALS

- *CAPPING OR SUBGRADE IMPROVEMENT LAYER*
  For Northern Countries and South European Countries.

- SUB-BASE
  For Northern Countries and South European Countries.

- *UNBOUND BASE*
  For Northern Countries and South European Countries.

4. CONSTRUCTION GUIDELINES

Spreading and Compaction.
Sealing (surface dressing, sub-base or unbound base).
Surfacing - type, depth, location of joints.

5. QUALITY CONTROL OF PAVEMENT MATERIALS

Type, timing and frequency of testing.

6. MAINTENANCE

Time interval between routine sealing or surface dressing of surfacing.
If stage construction planned, the type and timing of strengthening measures.

7. COSTS and BENEFITS

Initial construction costs, routine maintenance, and planned strengthening.
Benefits compared with other options.