COURAGE – Demonstrating Efficient Use of Unbound Granular Materials in Pavement Construction

Michael Mundy, Transport SA, South Australia, Australia
Andrew Dawson, School of Civil Engineering, University of Nottingham, Nottingham, UK

ABSTRACT
The paper gives a broad summary of the COURAGE project, a two-year, 10-centre, collaborative project funded by the European Union, national research and industrial contributions. The project has sought to improve our knowledge of the performance of granular material for road construction, in particular investigating in-situ condition, laboratory-based performance testing and in-situ performance evaluation. These aspects were linked together by data analysis and modelling so that the effects of material, condition and application could be understood and strategies for optimal use defined. The paper reports on these studies and shows that successful road construction using thin asphalt layers and thick granular layers is possible even in temperate and frost-affected climates. It clearly illustrates the need for the user to understand the true behaviour of the compacted material at the relevant moisture content, rather than to study individual stone characteristics. Such an approach allows any material to be assessed – even if a non-conventional aggregate – and applied sensibly. The implications for cost savings and natural resource protection are immense.

1 Introduction
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. These include the prediction of traffic volumes (present and future and their distributions, day and night), the variation of 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). Regarding European pavements, it may be noted that:

- 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 300-1500 mm in thickness, is constructed from Unbound Granular Material (UGM),
- Thus between 6000 and 30 000 tonnes of aggregate is consumed per kilometre of single carriageway pavement,
- The total European aggregate consumption from quarries and gravel pits into unbound pavement layers is of the order of 750 million tonnes per annum.

The “COURAGE” project (which stands for COnstruction with Unbound Road AGgregates in Europe) was established to improve the understanding of UGMs which may be used in the base and sub-base layers through:

- a targeted laboratory and field based testing programme to study test methods and their inter-relationships and to characterise a range of UGMs,
- field trials to study seasonal variation in UGM condition and pavement performance,
- material response modelling,
- full-scale experimental pavements to compare with numerical predictions (not described here),

in order that this material might be used more efficiently in pavement construction.

The COURAGE project was funded by the European Commission under the DG VII Fourth Framework Programme.
RTD framework programme and comprised a research team as follows:

Co-ordinator: UN (University of Nottingham) – United Kingdom
Partners: IST (Instituto Superior Técnico) – Portugal
          VTT (Technical Research Centre of Finland) – Finland
          LCPC (Laboratoire Central des Ponts et Chaussées) – France
          NRA (National Roads Authority) – Ireland
Associate Partners: ZAG – Slovenia
                  Finnra (Finnish Roads Administration) – Finland
                  PRA (Public Roads Administration) – Iceland
                  UH (University of Hannover) – Germany
                  TEI-Athens – Greece

The COURAGE project has collaborated closely with the COST 337 “Unbound Granular Materials” Action and with the parallel project on “Alternative-Materials” (ALT-MAT) so that its inputs and impact were wider than would otherwise have been the case.

To improve the use, efficiently, of UGM in pavements, the project team took the view that their work should seek to describe UGM in ways suitable for use in mechanistic (i.e. analytical or rational) design methods. So particular attention was given to:

- defining tests and procedures suitable to determine the functional properties of UGMs, specifically the elastic resilient modulus\(^1\) and Poisson's ratio,
- their density and moisture content at which these properties apply,
- the climatic conditions, which influence these functional properties,
- the stress-dependency of these properties (i.e. the value changes depending on the stress levels that the UGM experiences),
- appropriate modelling to incorporate these factors into analysis,
- comparing the results obtained from performance tests with those of simple tests which are easier to perform in practice,
- high quality material testing at a range of expected in-situ conditions in order to obtain a reliable characterisation of a number of granular materials, representative of those used in different European Countries.

2 Seasonal Variability – Field Trials of Pavement Performance

2.1 Installation and response of moisture sensors

Test sites were located in five countries (see Table 1), although only three are reported in this paper. 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.

The equipment used to measure in-situ moisture content of unbound granular materials was, in all but the Irish sites, using Time Domain Reflectometer (TDR) probes. In addition, thermocouples were used for temperature measurements. The TDR method measures the free (unfrozen) water in the volume of surrounding material, so it cannot accurately record the moisture content of frozen material. In Ireland a simpler, manually operated, system capable of measuring

\(^1\) In this paper the phrase “resilient modulus” is used to describe the ratio of transient stress change to recovered strain upon unloading.
changes in moisture and temperature using a resistive technique was employed. The equipment
and its use is described in more detail in the COURAGE Final Report [5].

<table>
<thead>
<tr>
<th>Country</th>
<th>Site Ref.</th>
<th>No. of Sections</th>
<th>Sensor Type*</th>
<th>Pavement Material</th>
<th>Surfacing Type</th>
<th>Depth (mm)</th>
<th>Traffic (AADT)</th>
<th>Site Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Iceland</td>
<td>IS.1 &amp; 2</td>
<td>2</td>
<td>TDR/A</td>
<td>Gravel S.D.</td>
<td>S.D.</td>
<td>46, 37</td>
<td>445</td>
<td>Raised pavement</td>
</tr>
<tr>
<td></td>
<td>IS.3</td>
<td>1</td>
<td>TDR/A</td>
<td>Gravel S.D.</td>
<td>S.D.</td>
<td>36</td>
<td>370</td>
<td>Raised pavement</td>
</tr>
<tr>
<td>Finland</td>
<td>Fl.1</td>
<td>1</td>
<td>TDR/M</td>
<td>Gravel A.C.</td>
<td>A.C.</td>
<td>50</td>
<td>1,400</td>
<td>Raised pavement</td>
</tr>
<tr>
<td></td>
<td>Fl.2</td>
<td>2</td>
<td>TDR/A</td>
<td>Crushed Rock A.C.</td>
<td>A.C.</td>
<td>80</td>
<td>6,500</td>
<td>Raised pavement</td>
</tr>
<tr>
<td>Ireland</td>
<td>EI.1&amp;2</td>
<td>2</td>
<td>RES/M</td>
<td>Crushed Rock H.R.A.</td>
<td>100</td>
<td>3,600</td>
<td>Pavement in cutting</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EI.3</td>
<td>1</td>
<td>RES/M</td>
<td>Rock Gravel A.C.</td>
<td>100</td>
<td>9,000+</td>
<td>Pavement in cutting</td>
<td></td>
</tr>
<tr>
<td>Slovenia</td>
<td>SI.1</td>
<td>1</td>
<td>TDR/A</td>
<td>Stab. Base Silty clayey gravel A.C.</td>
<td>130</td>
<td>13,000</td>
<td>Pavement with sensors in shallow cutting</td>
<td></td>
</tr>
<tr>
<td>Portugal</td>
<td>PT.1</td>
<td>1</td>
<td>TDR/A</td>
<td>Crushed Rock Mainly A.C.</td>
<td>295</td>
<td>?</td>
<td>Pavement in cutting</td>
<td></td>
</tr>
</tbody>
</table>

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  
AADT = Annual Average Daily Traffic

TABLE 1:  DETAILS OF TEST SITES USED TO MONITOR SEASONAL VARIABILITY EFFECTS

![Graph](image.png)

**Figure 1:** Gravimetric moisture content at Site IS.2, Iceland, and daily precipitation, June 1998 - August 1999

**2.1.1 Test Sites in Iceland**
The most severe conditions encountered were those recorded in Iceland (Sites IS1.2 and 3),
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, reported in Figure 1, shows how precipitation, freezing and thawing of the pavement and subgrade layers, vary throughout the year [1]. 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, as ice turns into free and measurable moisture content. 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.

2.1.2 Test Sites in Finland
For the site in Finland, Site FI.2 (reported by Suni and Kuja-la, [11]) much less variability in moisture conditions was seen (see Figure 2), than as recorded in Iceland, 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 winter 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 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 (refer to Figure 2). The moisture contents in the pavement layers in cutting were slightly higher than those recorded in the embankment section.

2.1.3 Test Sites in Ireland
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 contents of the materials have remained very stable at both sites since April 1999. For the granular materials at Site EI.1-2, the moisture content recorded was about 2% while 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 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 thickness of surfacing as the carriageway so that infiltration of water through the shoulder is severely restricted (see Figure 3).

![Figure 3: Typical Irish pavement with a 3m wide hard shoulder other sites](image)

### 2.2 Influence of moisture on pavement performance

FWD surveys are a valuable means of monitoring the response of the pavement structure as it varies during the year, and it also allows comparisons to be drawn between similar pavement types in different locations. As a summary of the data collected, the central deflection ($D_0$) and deflections at certain radii from the centre of the plate are shown in Table 2. The central deflection gives an indication of the stiffness response (or ‘bearing capacity’) of the pavement and supporting layers while the outer deflection measurements give an indication of the lower pavement layers, especially the subgrade.

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 are obvious changes in the bearing capacity of granular road pavements in Iceland. In both 1998 and 1999, 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. This is briefly summarised in Table 2.

The results in Table 2 show that there is considerable variation in mean maximum deflections between the experimental pavements in the different countries. These differences can be attributed, in large part, to the variations in weather conditions, pavement construction thicknesses, type of granular materials used, the underlying subgrade, and the type, depth and temperature of bituminous surfacing. The following specific factors were identified as
influencing the FWD readings:

- the high deflections measured at Site IS.2 were greatly influenced by the peat subgrade,
- at Site FI.2 higher readings were recorded in the near-side wheel track than towards the centre of the pavement indicating that, even though the shoulder was surfaced for a width of 1.25m, the moisture ingress from the edge of the pavement is having a deleterious effect,
- a stiff, serviceable pavement can be constructed where a sufficient depth of good quality granular material is used, and the soil support conditions are favourable (FI2, EI1 & EI3),
- the use of high quality crushed limestone in the base and sub-base layers over a rock subgrade, produced a very strong, stiff pavement (EI2),
- there is considerable pavement weakening on thawing and the associated moisture increase (e.g. compare Table 2 and Figure 1 for site IS2).

<table>
<thead>
<tr>
<th>Site</th>
<th>Date</th>
<th>Pavement Layer Thickness (mm)</th>
<th>Mean Deflections (Microns)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Surfacing</td>
<td>Base + Sub-base</td>
<td>Do</td>
</tr>
<tr>
<td>IS.2</td>
<td>31/3/99</td>
<td>37</td>
<td>770</td>
<td>122</td>
</tr>
<tr>
<td></td>
<td>9/4/99</td>
<td></td>
<td></td>
<td>1449</td>
</tr>
<tr>
<td></td>
<td>27/4/99</td>
<td></td>
<td></td>
<td>934</td>
</tr>
<tr>
<td></td>
<td>30/8/99</td>
<td></td>
<td></td>
<td>681</td>
</tr>
<tr>
<td>IS.3</td>
<td>9/4/99</td>
<td>36</td>
<td>814</td>
<td>517</td>
</tr>
<tr>
<td></td>
<td>8/5/99</td>
<td></td>
<td></td>
<td>1076</td>
</tr>
<tr>
<td></td>
<td>30/8/99</td>
<td></td>
<td></td>
<td>877</td>
</tr>
<tr>
<td>FI.1</td>
<td>9/10/98</td>
<td>50</td>
<td>500 Base + 430 sub-base</td>
<td>1011</td>
</tr>
<tr>
<td></td>
<td>11/5/99</td>
<td></td>
<td></td>
<td>983</td>
</tr>
<tr>
<td></td>
<td>28/9/99</td>
<td></td>
<td></td>
<td>858</td>
</tr>
<tr>
<td>FI.2/1</td>
<td>13/10/98</td>
<td>80</td>
<td>500+300 Filter sand Layer</td>
<td>340</td>
</tr>
<tr>
<td></td>
<td>19/11/98</td>
<td></td>
<td></td>
<td>71</td>
</tr>
<tr>
<td></td>
<td>28/5/99</td>
<td></td>
<td></td>
<td>374</td>
</tr>
<tr>
<td></td>
<td>8/10/99</td>
<td></td>
<td></td>
<td>334</td>
</tr>
<tr>
<td>FI.2/2</td>
<td>19/11/98</td>
<td>80</td>
<td>500+300 Filter sand Layer</td>
<td>38</td>
</tr>
<tr>
<td></td>
<td>28/5/99</td>
<td></td>
<td></td>
<td>295</td>
</tr>
<tr>
<td></td>
<td>8/10/99</td>
<td></td>
<td></td>
<td>278</td>
</tr>
<tr>
<td>EI.1</td>
<td>11/10/99</td>
<td>100</td>
<td>300+ 600 Capping</td>
<td>145</td>
</tr>
<tr>
<td>EI.2</td>
<td>11/10/99</td>
<td>100</td>
<td>600 Capping</td>
<td>98</td>
</tr>
<tr>
<td>EI.3</td>
<td>26/3/99</td>
<td>100</td>
<td>600 Capping</td>
<td>348</td>
</tr>
<tr>
<td></td>
<td>8/10/99</td>
<td></td>
<td></td>
<td>284</td>
</tr>
</tbody>
</table>

**Table 2: Deflections Measured Under FWD 50KN Load From Test Sites**

### 3 Laboratory Study

#### 3.1 Selection of materials

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

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

In addition, two secondary 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 generous parallel funding provided by Roadstone Ireland. The Irish materials assessed were:
- greywacke
- white reef limestone
- dark argillaceous limestone
- crushed limestone
- crushed granodiorite
- basalt/dolerite
- rhyolite

Additional aggregates were used locally for particular investigations.

### 3.1.1 Simple characterisation testing

For each material, split representative samples were sent to the different laboratories involved (UN, IST, LCPC, NRA, ZAG, PRA) and subjected to an extensive programme which is summarised in Table 3. Generally CEN (e.g. [4]) or national test procedures were used to assess the materials.

<table>
<thead>
<tr>
<th>Tests</th>
<th>General Material Assessment:</th>
<th>Fragmentation / Abrasion Assessment:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Petrographic Description</td>
<td>Los Angeles (LA)</td>
</tr>
<tr>
<td></td>
<td>Flakiness Index (Shape)</td>
<td>Modified Bg Index (MBg)</td>
</tr>
<tr>
<td></td>
<td>Plasticity (LL, PL, PI)</td>
<td>Dutch Static Compression (DSC)</td>
</tr>
<tr>
<td></td>
<td>Compaction (Modified Proctor)</td>
<td>Wet/Dry Strength (10% Fines)</td>
</tr>
<tr>
<td></td>
<td>Specific Gravity</td>
<td>Micro Deval (MDE)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gyratory Compaction (PCG)</td>
</tr>
<tr>
<td></td>
<td>General Strength/Stiffness Assessment:</td>
<td>Performance Assessment:</td>
</tr>
<tr>
<td></td>
<td>Static Triaxial (STXL)</td>
<td>Repeated Load Triaxial (RLT)</td>
</tr>
<tr>
<td></td>
<td>Ultrasonic Wave Velocity (UWW)</td>
<td>Wheel Tracking (WT)</td>
</tr>
</tbody>
</table>

**Table 3:** Simple & Functional Tests Applied to Selected Materials

### 3.1.2 Repeated Load Triaxial (RLT) testing

For the RLT testing (by UN, IST, LCPC & ZAG), materials were assessed at a defined range of density and moisture content conditions to investigate the effect of these variations on permanent strain and resilient modulus. In addition, three distinct material gradings were selected (low fines, reference-control and high fines gradings). The gradings were categorised according to the percentages of fines (passing a 75 µm sieve) for each selected material (refer to Table 4). The reference 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 local (UK) gradings for this material. For each percentage of fines, different moisture and density conditions were selected centred around a ‘normal’ dry density 97% of optimum and a moisture content 2% dry of optimum (optimum being defined using the modified compactive effort). For the RCC&A material, the central moisture content was 4% less than optimum. The testing methodology and stress path sequences for the RLT test are set in accordance with the draft CEN procedure [4].

### 3.1.3 Wheel Tracking (WT)
Wheel tracking (WT) testing was performed for the COURAGE project using the Pavement Test Facility at the University of Oulu. This Facility is a laboratory-scale test track device with a moving vehicle load. It can be used easily and reliably to simulate the stresses caused by traffic in the road structure [8]. 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 of filter sand, was used as the test structure. 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 measured at three different levels. The principal aim was to provide a comparison of the rutting susceptibility of the three high-grade aggregates already assessed in the RLT. The three primary materials were tested at the reference grading at a dry density 97% of optimum and a moisture content at 55-60% of the optimum value. The wheel tracking and repeated load triaxial tests enabled the moisture sensitivity of resilient modulus and the resistance to permanent deformation of the different materials to be assessed.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Coarse</th>
<th>Reference</th>
<th>Fine</th>
<th>Supply</th>
</tr>
</thead>
<tbody>
<tr>
<td>(mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>98.9</td>
</tr>
<tr>
<td>14</td>
<td>68</td>
<td>79.7</td>
<td>91</td>
<td>84.8</td>
</tr>
<tr>
<td>10</td>
<td>55</td>
<td>68.6</td>
<td>82</td>
<td>72.4</td>
</tr>
<tr>
<td>6.3</td>
<td>42</td>
<td>58.3</td>
<td>70</td>
<td>56.5</td>
</tr>
<tr>
<td>4.0</td>
<td>32</td>
<td>45</td>
<td>60</td>
<td>44.4</td>
</tr>
<tr>
<td>2.0</td>
<td>22</td>
<td>33.3</td>
<td>49</td>
<td>32.6</td>
</tr>
<tr>
<td>0.6 / 0.5</td>
<td>11</td>
<td>16.4</td>
<td>30</td>
<td>17.6</td>
</tr>
<tr>
<td>0.212 / 0.2</td>
<td>7</td>
<td>10.8</td>
<td>20</td>
<td>6.6</td>
</tr>
<tr>
<td>0.075</td>
<td>3</td>
<td>7.4</td>
<td>10</td>
<td>2.7</td>
</tr>
</tbody>
</table>

Table 4: Grading Sizes for the RLT and WT Materials

3.2 Test Results
3.2.1 Characterisation
A wide range of simple tests were completed together with CBR, static triaxial and gyratory shear abrasion testing [5]. Of particular interest may be the work performed by PRA (Iceland) who assessed aggregate resistance to fragmentation, weathering and abrasion in an extensive testing programme. Their results [2] demonstrate that degradation tests can be 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. In summary:

- the Modified Bg 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.
- The new gyratory shear abrasion 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 (e.g., fragmentation, durability or abrasion),
limited portion of the material's parent grading curve used in the simple test,
severity of the conditions used in the simple test, with little relationship to real loading conditions,
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.

3.2.2 Results of RLT testing

a) Permanent Strain
A typical result for permanent axial and radial strains during the preconditioning test is shown in Figure 4. 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. As a result of the COURAGE testing, it is clear that some materials can support the ‘heavy’ preconditioning load applied ($\sigma_1 = 700\text{kPa}$ and $\sigma_3 = 100\text{kPa}$) 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.

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 is probably a consequence of the different compaction methods used by LRPC St. Brieuc (vibro-compression) and IST and ZAG (vibrating hammer).
Two forms of modelling analysis were carried out (see Section 3.3.2). The results of the high-grade aggregates (at their control gradings) and RCC&A (at its supply grading) are presented in Figures 5 and 6 for both the LCPC and TSA strain models.

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 same materials, at finer or coarser gradings were generally found to be more resistant to permanent deformation than at their control gradings for the same value of moisture content relative to OMC. The recycled crushed concrete and asphalt material, possibly along with the granitic
material, were completely failing at 67% of OMC. By using static triaxial testing, it was found that the shear strength of the RCC&A material at this condition was equivalent to the preconditioning stress levels applied to the specimen, thus the material was 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 both the VCP and CCP tests, comparable with the granite. These illustrate that for all the materials it is possible to deduce a relative moisture content at which the permanent strain rate increases rapidly to an asymptote. The gneiss seems to be the material with the lowest moisture sensitivity (highest relative moisture content).

At similar moisture contents, the dark argillaceous and carboniferous limestones from Ireland exhibited relatively high rates of strain, as did the greywacke and crushed granodiorite, although the white reef or crushed limestone, rhyolite and basalt/dolerite materials performed well at around 50% of their optimum moisture content.

The response of the granite and gneiss materials to density was variable. The gneiss showed a similar resistance to permanent deformation development at all densities (DDR = 96, 97, 100% of MDD), while the granite showed a very large reduction in permanent strain susceptibility as the density increased.

b) Resilient Modulus

The test results were fitted to the models described in Section 3.3.1. For comparative purposes, a ‘characteristic’ stress level (p = 250kPa and q = 500kPa, or $\sigma_1 = 583kPa$ and $\sigma_3 = 83kPa$) is used with the Hooke Law to define a single value of resilient modulus for each material. All the high-grade aggregates, at their control grading, displayed a marked reduction in modulus as the moisture content increased relative to OMC. The levels of reduction between the moisture range of RMC = 40 to 70% were:

- gneiss, 10%
- granite, 17%
- limestone, 18%

It was found that variations in grading did not significantly effect modulus for the materials tested, although the modulus of the finer-grained granite was around 10% lower than that of the granite at its control grading across the moisture range. The finely-graded gneiss appeared to generate a higher stiffness at a higher moisture content than the control grading product.

The recycled crushed concrete and asphalt material was failing at 80% of OMC, with still relatively high strain rates at 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 variable confining pressure (VCP) test was quite high around 490MPa. High moduli often result after significant permanent deformation has been induced in the material. This also was the case for the limestone material.

All of the Irish limestone materials were found to have high ‘characteristic’ moduli at the stress level used for comparison. This appears to be a common feature of most materials of this geology. The white limestone samples 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 OMC.

3.2.3 Wheel tracking
The estimated stress at the top of the granular material layer (with a 30mm asphaltic surfacing) directly under the wheel load is approximately $\sigma_1 = 600\text{kPa}$. The confining stress determined at this point is approximately $\sigma_3 = 85\text{kPa}$. This stress combination results in approximately $p = 255\text{kPa}$ and $q = 510\text{kPa}$, compared with the RLT test preconditioning load of $p = 300\text{kPa}$ and $q = 600\text{kPa}$ ($\sigma_1 = 700\text{kPa}$ and $\sigma_3 = 100\text{kPa}$). The permanent and elastic deformations for the pavement structures tested are tabled below (Tables 5 and 6).

<table>
<thead>
<tr>
<th>Loading cycle range</th>
<th>Granite IST Portugal</th>
<th>Gneiss LCPC France</th>
<th>Limestone ZAG Slovenia</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Part of the base course</td>
<td>Whole</td>
<td>upper</td>
</tr>
<tr>
<td>0 – 1 000</td>
<td>1.731</td>
<td>1.543</td>
<td>0.188</td>
</tr>
<tr>
<td>1 000 – 5 000</td>
<td>1.119</td>
<td>1.080</td>
<td>0.039</td>
</tr>
<tr>
<td>5 000 – 10 000</td>
<td>0.506</td>
<td>0.482</td>
<td>0.024</td>
</tr>
<tr>
<td>10 000 – 20 000</td>
<td>0.375</td>
<td>0.344</td>
<td>0.031</td>
</tr>
</tbody>
</table>

**Table 5: Permanent Deformations [mm]**

<table>
<thead>
<tr>
<th>Loading cycle range</th>
<th>Granite IST Portugal</th>
<th>Gneiss LCPC France</th>
<th>Limestone ZAG Slovenia</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Part of the base course</td>
<td>Whole</td>
<td>upper</td>
</tr>
<tr>
<td>0 – 1 000</td>
<td>361</td>
<td>209</td>
<td>152</td>
</tr>
<tr>
<td>1 000 – 5 000</td>
<td>376</td>
<td>243</td>
<td>133</td>
</tr>
<tr>
<td>5 000 – 10 000</td>
<td>374</td>
<td>240</td>
<td>134</td>
</tr>
<tr>
<td>10 000 – 20 000</td>
<td>382</td>
<td>229</td>
<td>153</td>
</tr>
</tbody>
</table>

**Table 6: 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 97%, gneiss material 73% and limestone material 85% of the whole basecourse 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). In general the recovered deformation appeared to be least in the limestone material basecourse. In the upper half of the basecourse, the elastic deformations of the granite and gneiss materials were 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 would 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 100 cycles of load. This will allow comparison with the strain rates computed from the RLT
preconditioning test. Figure 7 shows that the strain rate of the granitic material is markedly higher than that of the gneiss and limestone materials. It was observed that the LCPC permanent strain model (using parameter A) and the permanent strain rate model (using parameter $d\varepsilon_p/dN$) illustrated virtually identical strain susceptibility of the three aggregates examined.

If the 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.

### 3.3 Modelling of results

#### 3.3.1 Resilient modelling

It is well known that resilient modulus is stress-dependent (i.e., its value changes depending on the stress levels that the UGM experiences). This can be handled, in design, by an iterative use of relatively simple linear elastic layer techniques or by use of more advanced non-linear response models incorporated into finite element code.

To allow 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 [3] and a modified anisotropic form [7],
- Dresden model [6].

The focus was centred around the non-linear isotropic elastic ‘Boyce’ model [3] which is
defined using invariant stress and strain parameters which are defined as follows for the conditions of the repeated load triaxial test:

\[ p = \left( \sigma_1 + 2\sigma_3 \right)/3 = \text{mean normal stress} \]

\[ q = \left( \sigma_1 - \sigma_3 \right) = \text{shear stress} \]

\[ \varepsilon_v = \varepsilon_1 + 2\varepsilon_3 = \text{volumetric strain} \]

\[ \varepsilon_q = \frac{2}{3} (\varepsilon_1 - \varepsilon_3) = \text{shear strain} \]

where \( \sigma \) and \( \varepsilon \) represent stress and strain and the numbered subscripts indicate principal values. The model can be expressed in terms of bulk moduli (\( K = p/\varepsilon_v \), a measure resistance to compression) and shear moduli (\( G = q/3\varepsilon_q \), a measure of distortional resistance). According to Boyce's derivation the values of \( K \) and \( G \) depend on the applied stresses as follows:

\[
K = \frac{\left( \frac{p}{p_a} \right)^{1-n}}{1 + \frac{(n-1)q}{12G_a} \left( \frac{p}{p_a} \right)^n}, \quad \text{and} \quad G = G_a \left( \frac{p}{p_a} \right)^{1-n}
\]

where \( K_a, G_a \) and \( n \) are model parameters and \( p_a \) is a reference normalising stress taken to be 100kPa.

Some granular materials present a somewhat anisotropic behaviour (characterised by different values of axial strains \( \varepsilon_1 \) and radial strains \( \varepsilon_3 \) for isotropic loadings). For these materials, the fit of the experimental results with the Boyce model was not very satisfactory. So as to model 'anisotropy', the expression of the elastic potential proposed by Boyce was modified by multiplying the principal stress \( \sigma_1 \) by a 'coefficient of anisotropy', \( \gamma \), such the mean normal and shear stress expressions were modified [7] to:

\[
p^* = \left( \gamma \sigma_1 + 2\sigma_3 \right)/3 = \text{modified mean normal stress}
\]

\[
q^* = \left( \gamma \sigma_1 - \sigma_3 \right) = \text{modified shear stress}
\]

with:

\[
\varepsilon_v^* = \frac{\varepsilon_1}{\gamma} + 2\varepsilon_3 \quad \text{and} \quad \varepsilon_q^* = \frac{2}{3} (\varepsilon_1/\gamma - \varepsilon_3)
\]

\( p^* \) and \( q^* \) then replace the \( p \) and \( q \) in Equation 1, respectively. 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:

\[
E_h = \frac{9K^*G^*}{3K^* + G^*}, \quad E_v = \frac{E_h}{\gamma^2}, \quad \nu_{hh} = \frac{3K^* - 2G^*}{6K^* + 2G^*}, \quad \nu_{hv} = \frac{\nu_{hh}}{\gamma}, \quad \nu_{vh} = \gamma \cdot \nu_{hh}
\]

In general, experimental results lead to values of \( \gamma \) 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). When these models were applied to the test results it was found that 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.

### 3.3.2 Permanent deformation modelling

Two models were selected for the permanent strain analysis of the materials. The first being a model used by LCPC [10] and the second a strain rate model used by TSA, Australia [9].
The first of these models has the form:

\[ \varepsilon_{1p}^* (N) = A_i \left[ 1 - \left( \frac{N}{100} \right)^{-B} \right] \]

where \( A_i \) and \( B \) are material parameters, \( \varepsilon_{1p}^* \) is the axial permanent strain following the removal of that recorded in the first 100 cycles of loading and \( N \) is number of cycles of loading applied.

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

\[ \varepsilon_{1p} (N) = \varepsilon_r \frac{\mu}{\alpha} N^\alpha \]

where \( \mu \) and \( \alpha \) are material parameters and \( \varepsilon_{1r} \) is the axial resilient strain. 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 [9], viz:

\[ \frac{d\varepsilon_{1p}}{dN} (N) = \varepsilon_r \mu N^{\alpha - 1} \quad [\% / \text{cycle}] \]

Both models are stress-dependent and both neglect the first 100 cycles of measured strains due to effects of bedding, etc.

4 Material Classification Based on Laboratory Testing

An attempt at material classification was made, aiming to place each material tested as part of COURAGE into a material quality “class” (not expressing a ranking relating to traffic level). This “class” was 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. This assessment is described in detail in the Final Report [5]. In principle, materials are divided into three classes – 1=good (high strength and modulus, resistant to rutting, low moisture sensitivity), 2=moderate (a ‘good’ response only in some aspects), 3=poor (few or no ‘good’ aspects). On this basis an overall classification would be:

**Gneiss** - class 2, primarily for reasons of:
- low resilient moduli at expected in-service conditions
- performance in the LCPC experiment with major distress (rutting, surface cracking) after only 400,000 standard loading cycles

**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
- 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
sourced from lower quality concrete areas with material containing little free, previously unhydrated cement paste (which can improve strain and moduli performance with time due to the cementitious effects).

**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

5 Summary and Conclusions

The following key observations were 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 an 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,
  - 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),
  - interval between readings

♦ 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 with thin surfacings (e.g. the EI pavements, Table 2).

♦ 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 RLT was employed to allow the effects of changing moisture content to be assessed. It was found that significant reductions in the resilient modulus of the UGMs were observed as the moisture content increased. This parameter largely dictates the ability of the UGMs to spread traffic loads safely onto the underlying soil without over-stressing it and it strongly influences the fatigue life of the surfacing layer. 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.

* The variation of resilient modulus with moisture content suggests that analytical procedures which adequately incorporate the behaviour of the UGM layers must, for thinly surfaced pavements, incorporate material properties carefully matched to the correct moisture content if they are to successfully predict pavement performance.

* The Boyce constitutive model of resilient stress-strain behaviour [3, 7] provided an appropriate non-linear representation of the UGM.

References


