NR2C - New Road Construction Concepts
Work Package 2 – Interurban infrastructures

**Deliverable 2.2**

Concept and design of selected innovations for interurban infrastructure

Synthesis reports

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A - Introduction

The highway traffic size and loads have been continuously increasing all over Europe. At the same time, the expectations of users have been growing. They need more safe, durable and comfortable highway surfaces which make their trips low-cost and do not accept frequent traffic disturbance due to road maintenance and rehabilitation. The obvious answer of the highway engineering to this challenge is the continuous development and improvement of highway construction, rehabilitation and maintenance techniques further.

The state-of-the art review of NR2C Work Package 2 on European highway innovations (Deliverable 2.1[1]) has clearly demonstrated the typical trends of innovative road construction, rehabilitation and maintenance which are as follows:

- the use of very high quality (premium) basic materials eventually with their special treatment,
- the establishment of sophisticated construction, rehabilitation and maintenance techniques utilising up-to-date scientific achievements,
- the development of special measures for enhancing traffic safety even in extreme conditions,
- decreasing the whole life (life cycle) costs of road pavements by constructing long-life variants with infrequent maintenance and rehabilitation need, and, consequently, minimal traffic disturbance,
- the wider use of industrial by-products in road engineering without reducing pavement performance,
- the wider use of recycling (eventually-re-use) of bound pavement structural layers in order to reduce the need for primary basic materials without jeopardising the performance of pavements,
- giving priority to low-energy pavement structural variants reacting to the ever increasing energy prices and the limited availability of crude oil supplies,
- there are some “blue sky” type innovations which utilise some new scientific results eventually coming from science areas far away from highway engineering.

Vision 2040 elaborated by Work package 0 can be considered as the basic document of NR2C project. It identifies the research areas the successful elaboration of which can have a significant contribution to the attaining of reliable, comfortable and safe roads of next-coming decades. The NR2C vision 2040 designates four main concepts for the road of the future. The innovations considered in WP2 all deal with solutions for one or more of the above concept needs, and mainly with the green infrastructure by reservation of rare resources via recycling and use of industrial by-products.

Innovation 2.1 A Design of high performance layer with raw material (EPFL-Switzerland, BRRC-Belgium, and other FEHRL-laboratories as VTI, KTI, DRI).

The goal is to evaluate if the growing share of recycled aggregate used in asphalt high stiffness base courses influences the asphalt mechanical properties. It is the aim of this study that no significant loss in asphalt fatigue, deformation and durability characteristics counterbalances the environmental and economic benefits coming from the use of recycled material. Comprehensive laboratory and Accelerated Loading Tests (ALT) are performed in the project.

Innovation 2.1B Crack free semi-rigid pavement incorporating industrial waste (LCPC-France)

The goal is to evaluate if the natural cement concrete shrinkage can be compensated by adding industrial by-products (steel slag, fly ash) with swelling ability to the mixture. The consequences of the use of additional CaO are also tested. The main idea is to minimise (even to stop) the cracking of hydraulically bound layers and so, to avoid the reflection cracking in the asphalt layers built on them. The innovation is closely connected to the concept “safe infrastructure”.

Innovation 2.2. Use of the infra-red characteristics of materials to improve drivers’ visibility (LCPC-France).
The goal is to enhance the traffic safety by improving the drivers' visibility among unfavourable conditions (darkness, fog etc.). The use of infrared image technique can be the solution. Simulation and real test site measurements are applied for the validation of the innovative technique. It can contribute to the concept “reliable infrastructure”.

Innovation 2.3. Optimisation of the maintenance process (Eurovia-France)

The goal is to evaluate whether asphalt laying activities can be performed without detrimental consequences under extreme weather conditions (too low or too high temperature, rain etc.). The proposed innovative techniques are supposed to ensure the required asphalt quality and not to increase the construction costs considerably. The success of the project can contribute to the lengthening of the construction season without quality compromise. It was to contribute to the concept “reliable infrastructure”.

Innovation 2.4. Improving the mechanical properties of a low noise section (VTI-Sweden, ZAG-Slovenia)

The goal of this innovation initiated in a later phase of NR2C WP2 activities is to evaluate (and eventually to improve) the functional and the mechanical properties (durability) of low-noise poroelastic layers built on cement concrete blocks. Laboratory and site tests are to be performed for the evaluation of these properties. So, it contributes to the concept “green infrastructure”, by the reduction of noise nuisance coming from traffic.
B - Executive summary

As explained in the introduction, the different innovations selected for this work package 2 are focused on various topics. The executive summary regarding each innovation can be found hereafter.

**Innovation 2.1A: Development of high performance underlayers with low cost materials and high percentage of re-use**

This project aimed to optimize the design of mixes with high percentages of recycling material so as to guarantee their long-term performance. In this innovation, three different mixes were designed, optimized and compared, namely with 0 %, 25 % and 40 % reclaimed asphalt. After an extensive laboratory study performed by BRRC, the selected solutions have been further studied in a full-scale ALT facility in LAVOC. The structure tested has been instrumented with strain gauges, deflection and temperature sensors in order to analyse its performances. Fatigue behaviour as well as low temperature behaviour was investigated. In addition to the ALT, tests on large slabs with high temperature conditions, as well as other laboratory tests on mixes and binders have been performed by BRRC, LAVOC and FEHRL laboratories (DRI, VTI, KTI).

This study leaded to the conclusion that no negative effect has been found by using a high percentage of reclaimed asphalt. However key parameters as for instance the mix design and RA properties require special attention.

**Innovation 2.2: Roadway perception technology using the infrared know-how**

This project aims to address elements of interurban roads that can be modified to turn them more cooperative for on board automotive infrared vision systems. It mixes an experimental approach on real sites and in fog tunnel, infrared emissivity measurements with a dedicated apparatus developed for pavement surface characterization and simplified numerical simulation of road scene and attenuation by fog according to the size of water droplets distribution. Comparisons between experiments and simulations are done. Experiments on road site and in fog tunnel allowed us to validate numerical simulations. Numerical simulation tools developed permit to evaluate the size of cooperative elements of infrastructure required to be perceptible on infrared images by taking account the characteristics of on-board infrared vision system used. Nevertheless, experiments have also permitted to verify that improving contrast on infrared images by generating a thermal excitation on infrastructure typical elements had to be favour in front of reducing their emissivity in foggy night conditions. Finally, recovering energy from road could be a sustainable solution to generate active thermal elements for on board infrared vision system.

**Innovation 2.3: New pavement maintenance technique aiming at enlarging the overall conditions of application**

This innovation consists in the development of new maintenance techniques and procedures aiming at expanding the overall conditions of application of mixtures for pavements. The benefit of such an innovation is to reduce the impact of the weather conditions on the quality of placing of pavement mixtures, and consequently on the mechanical properties and behaviour of the road structure. Another benefit is the reduction of the impact of road closure due to maintenance on the road users, as it would be more likely to carry out pavement maintenance at more opportune times or periods throughout the year.

**Innovation 2.4**

This project aims at improving an innovative low noise pavement. Poroelastic pavement was invented more than thirty years ago. It is a road surface type with excellent noise reducing properties. However, the implementation of this road surface has been impeded for a long time by difficulties to achieve a good adhesive bond between the poroelastic material and the supporting underlying road structure, and more recently, by the discovery that wet frictional properties of the poroelastic material could be seriously damaged by traffic polishing. A solution to the adhesion problem is to glue the poroelastic material on top of paving stones in a clean controlled environment. This solution requires a strong supporting layer. One aim of this project is to make this solution feasible by experimentally verifying the requirements for sub layers supporting a poroelastic road surface on top of paving stones. The other aim is to improve the wet frictional properties of the poroelastic layer to achieve a polish resistant road surface.
C - Synthesis reports of the different innovations

C.1 Innovation 2.1A: Development of high performance base layers with low cost materials and high percentage of re-use

As mentioned in the introduction, NR2C developed long-term visions for road infrastructure and carried out some specific innovations, in which long-term visions and ideas are linked to short-term actions. One of the ideas in the framework of sustainable road construction worked out in NR2C concerned the development of high stiffness base layers with high percentages of re-use materials. Although high stiffness base layers are already extensively used in some European countries, the experience with re-use in such mixtures is still very limited. There is indeed a fear for a limited durability of these mixtures because of the combination of a hard binder (which is typical for these mixtures) and re-use material.

C.1.1 Material characterisation and mix design

High stiffness modulus mixes were prepared with Belgian as well as with Swiss materials. One and the same hard binder 10/20 was used through the whole study. Its characteristics are given in table C.1.1 [2]. The mixes with the Belgian materials were used for extensive laboratory testing. These tests provided information for the designs to be made with the Swiss materials, which were applied on the LAVOC ALT (accelerated loading testing) facility.

<table>
<thead>
<tr>
<th>Type</th>
<th>Pen [1/10mm]</th>
<th>R &amp; B [°C]</th>
<th>Binder content [%]</th>
</tr>
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<tbody>
<tr>
<td>Belgian RA</td>
<td>17</td>
<td>67.3</td>
<td>5.5</td>
</tr>
<tr>
<td>Swiss RA</td>
<td>32</td>
<td>59.4</td>
<td>4.8</td>
</tr>
</tbody>
</table>

Table C.1.1: Characteristics of the hard binder 10/20.

The BRRC software PradoWin was used for the mix designs and optimization. PradoWin is a user-friendly program, adapted for the volumetric mix design of bituminous mixtures, and with a special feature to facilitate the mix design of mixtures with re-use materials. The required input data (the characteristics of the constituent materials) were determined by BRRC [3]:

High stiffness mixtures for base layers can be achieved by using a high percentage of stones and a hard binder. Together with an increased binder content compared to a conventional dense asphalt composition suitable for base layers, this allows to design, despite of the high percentage of stones, relatively dense mixtures with a good coating of the aggregates and hence, a good performance in durability.

Two basic mix designs were made:
- one mix design with Belgian materials,
- one mix design with the Swiss materials to be used in the ALT study.

Different variants (with different percentages of RA) were designed, based on approximately the same grading curve:
- Variant 1: Design without RA (reference).
- Variant 2: Design with 25% RA.
- Variant 3: Design with 40% RA.

The analytical mix design was combined with subsequent gyratory compaction tests according to EN12697-31 to verify the compactability and the air void content. Depending on the results of the gyratory tests, the analytical mix design was adapted.

Table C.1.2 shows the final mix gradings for the various variants. The percentage of RA given in table C.1.2 stands for the percentage of old binder (from RA) on the total binder content.
An extensive laboratory study was then performed on all mixtures to check the laboratory performances:

- **Stiffness modulus** was determined according to EN12697-26 annex A (two-point bending test on trapezoidal samples) for temperatures between -20 °C and 30 °C and for frequencies between 1 and 30 Hz.
- **Resistance to fatigue** of the different mixes was determined according to the BRRC-method [4] (two point bending test on trapezoidal samples) at 15 °C and 10 Hz. This test method is close to EN12697-24 annex A, but is stress controlled and performed on large samples.
- **Resistance to permanent deformation** is determined according to EN12697-22 (large device in air) at a temperature of 50 °C.
- **Water sensitivity** is determined as the indirect tensile strength according to EN12697-23 before and after conditioning in water according to EN12697-12.

The results are given in table C.1.3. We note that for the Swiss mixtures with RA, some of the tests were performed with a lower binder content (5.7 and 5.6 % for 25 % and 40 % of RA respectively, instead of 5.8 %). The reason for this is that in an asphalt plant, the variations on binder content of RA are usually larger than in the laboratory. With a high percentage of re-use, the impact of this parameter on the total binder content is important. A way to deal with this uncertainty in the phase of mechanical performance testing is to make the tests with the most unfavourable estimation of the binder content. For the mix with 40 % of RA, a variation of 0.5 % on the binder content of the RA would lead to a variation of 0.2 % on the total binder content. By doing some tests with a total binder content of 5.6 % instead of 5.8 %, the laboratory tests will be on the safe side.

### Table C.1.2: Grading of the different mixtures

<table>
<thead>
<tr>
<th>Mixes with Belgian materials</th>
<th>Mixes with Swiss materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>% RA</td>
<td>0</td>
</tr>
<tr>
<td>% total binder</td>
<td>5.5</td>
</tr>
<tr>
<td>% RA aggregates</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>% passing on sieve</th>
</tr>
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<tbody>
<tr>
<td>20 mm</td>
</tr>
<tr>
<td>14 mm</td>
</tr>
<tr>
<td>10 mm</td>
</tr>
<tr>
<td>6.3 mm</td>
</tr>
<tr>
<td>4 mm</td>
</tr>
<tr>
<td>2 mm</td>
</tr>
<tr>
<td>1 mm</td>
</tr>
<tr>
<td>0.5 mm</td>
</tr>
<tr>
<td>0.25 mm</td>
</tr>
<tr>
<td>0.063 mm</td>
</tr>
</tbody>
</table>
### C.1.3 Accelerated loading tests

The accelerated loading tests have been performed in LAVOC’s facility. Using this facility, the different mixtures have been tested in an accelerated way by controlling different parameters. In addition to the ALT, some other tests on in situ mixes have been performed with the aim of providing additional information for a better performance analysis.

#### C.1.3.1 Accelerated loading tests setup

The selected solutions were tested in full-scale accelerated loading testing (ALT) facility of LAVOC. They have been applied in a test section which dimensions are 13.1 m x 5.4 m (circulation direction). The pavement design has been carried out using two specific pavement design softwares based on the multilayer theory of Burmister: the Belgian software DimMET (developed by BRRC and Febelcem for the Walloon Ministry of Equipment and Transport) and a French design method with the help of the

### Table C.1.3: Laboratory performance of the different variants

<table>
<thead>
<tr>
<th>% RA</th>
<th>Mixes with Belgian materials</th>
<th>Mixes with Swiss materials</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0</td>
<td>25</td>
</tr>
<tr>
<td>% total binder</td>
<td>5.5</td>
<td>5.5</td>
</tr>
<tr>
<td>% voids (gyratory)</td>
<td>3.3</td>
<td>3.8</td>
</tr>
<tr>
<td>Rut depth [%] at 30'000 cycles, 50 °C</td>
<td>2.7</td>
<td>-</td>
</tr>
<tr>
<td>ITS-testing (hydrostatic)</td>
<td>3.3</td>
<td>2.8</td>
</tr>
<tr>
<td>Strength before [MPa]</td>
<td>2.5</td>
<td>2.3</td>
</tr>
<tr>
<td>ITS-ratio</td>
<td>98 %</td>
<td>95 %</td>
</tr>
<tr>
<td>Stiffness modulus [MPa] at 15 °C - 10 Hz</td>
<td>12740</td>
<td>-</td>
</tr>
<tr>
<td>Fatigue [15 °C, 10 Hz]</td>
<td>0.156</td>
<td>-</td>
</tr>
<tr>
<td>Slope a ε=6 (μstrain)</td>
<td>123.4</td>
<td>-</td>
</tr>
<tr>
<td>N for ε = 120 μstrain</td>
<td>1.2 x 10^6</td>
<td>-</td>
</tr>
</tbody>
</table>

(*) determined for 5.7 % binder content instead of 5.8 % to investigate the risk of durability loss

(**) determined for 5.6 % binder content instead of 5.8 % to investigate the risk of durability loss

It was concluded that a high laboratory performance was reached on all aspects:

- All mixes have a very high stiffness around 12'000 – 13'000 MPa at 15 °C and 10 Hz.
- The resistance to permanent deformation is very high: always below 5 %. Note that this is the lowest value (best performance) according to the European specifications in EN1308-1.
- The resistance to the action of water is very high: for all mixes above 90 %, which shows that durability problems are not to be expected.
- The resistance to fatigue is very high: above 1.0 x 10^6 cycles at 120 microstrain. This is at least a factor seven better than a conventional Belgian mix for underlayers.
- The void content (measured hydrostatically) of the Swiss mixes with 25 % (binder content 5.7 %) and 40 % RA (binder content 5.6 %) is rather high: 5 % respectively 6 %.
- Mixes with reclaimed asphalt have equivalent performance as mixes without RA.
NOAH software. For this pavement design, the aim was to have a structure which is expected to reach the end of its design life after 2/3 of the planned passages. More details can be found in [5].

The tested structure, represented in figure C.1.1, is as follows:
- Layer 1: AC MR8 (3 cm),
- Layer 2: High Modulus Asphalt (8 cm),
- Layer 3: Soil foundation composed by gravel 0/60 (40 cm), fine sand (145 cm) and concrete.

Four different sections have been studied, with various HMA contents: a reference without RA (field 0), a section with 25 % of RA (field 1) and two sections with 40 % of RA of which one doesn’t include a wearing course (field 3).

The sections have then been loaded with a heavy traffic simulator (axle load of 12 tons, tyre pressure of 0.8 MPa and super-single tyre), which simulates traffic close to in situ conditions. In order to tests the different pavement types, three positions of circulation have been defined (figure C.1.2):

- Position A: Two wheels on field 3 with 40 % RA, no top layer (axles A1 and A2)
- Position B: One wheel on field 2 with 40 % RA (axle B1) and one wheel on reference field 0 (axle C1)
- Position C: One wheel on the reference field (axle C2) and one wheel on the field 1 with 25 % RA (axle D1)

Figure C.1.1: Cross section view of the tested pavement

Figure C.1.2: Positions of circulation for ALT testing
In order to measure horizontal stress and strain as well as temperature, different sensors have been installed at the bottom of the HMA and also at the interface between HMA and top layer. For temperature measurements, classical Pt100 have been used. Deformation measurements have been achieved using KYOWA strain gauges. A total number of 57 sensors have been installed in the pavement.

In addition to these sensors, surface deflection has been measured using a specific deflection beam instrumented with LVDT sensors (Figure C.1.3). Using this device, measurements close to the wheel passage have been performed.

![Figure C.1.3: Specific deflection beam used in the ALT facility](image)

The behaviour of the different sections has been assessed through two following test phases:

- In a first part, fatigue tests have been performed at a constant air temperature of 15 °C. During these fatigue tests, about 100'000 wheel passages have been performed on each position except position A that has got a total of 190'000 fatigue passages.
- The fatigue tests have then been followed by low temperature tests (LT). The aim of the second test phase was to simulate temperature cycles with circulation as well. Hence, air temperature variations between 2 °C and -7 °C were applied during 12 day for each position of circulation. In order to have a good temperature control, an isolated cabin with a cooling system using ventilators has been applied.

C.1.3.2 Results and analysis of the ALT

A total of more than 370 measurements have been performed during the whole test duration. For an assessment of the fatigue resistance of the different mixtures, special emphasis has been put on the deformations at the bottom of the asphalt layer. Traction is effectively most important at the bottom of the HMA and fatigue cracking will most likely occur at this interface [6].

Different comparisons between the mixtures have been carried out with the aim to assess if there is any negative effect using mixes with a high percentage of reclaimed asphalt instead of a mixture without RA. Parts of the results and the conclusions are presented hereafter, more details can be found in [7].

Comparison between 25 % RA and 0 % RA

In following figure C.1.4, each gauge is represented by a line while the code indicates the measurement axle C2 (0 % RA) or D1 (25 % RA). The first 100,000 passages correspond to the fatigue tests at 15 °C while the passages performed between 100,000 and 210,000 correspond to the low temperature tests (LT).

It is obvious that the general trend is the same for both axles i.e. both mixes. The small differences observed are not significant enough for concluding to a much better behaviour of the section with 25 % RA, but show that its resistance is at least the same as that of the reference mix. Moreover, the general order of magnitude for the deformation decreases during the low temperature tests. This was expected because by reducing the temperature, the pavement becomes stiffer and consequently the deformation decreases.
Figure C.1.4: Comparison between the performance of axle C2 (0 % RA) and D1 (25 % RA)

Comparison between 40 % RA and 0 % RA
The same comparison has been carried out for the mixtures with 40 % RA (axle B1) and the reference mix without recycling material (axle C1).

In figure C.1.5 below, the general trend shows a decreasing of the deformation during low temperature tests (between 100,000 and 190,000 passages) that was expected, but the order of magnitude is slightly bigger than for previous comparison with deformation up to 300 με. Comparing both mixes, the measurements on the section with the reference mix are a bit lower than with the mix containing 40 % recycling material. However, the differences measured are very small and they cannot be considered as a conclusion about a major behaviour difference between the mixes.

Figure C.1.5: Comparison between the performance of axle C1 (0 % RA) and B1 (40 % RA)

Considering both comparisons above between the different mixes, we can conclude that the same order of magnitude has been measured for the deformation in the mixes without and with RA. The
general trend of slightly lower deformation in the section with 25 % RA is not sufficient enough in order to deduce a difference in fatigue resistance.

In addition to the different measurements, some calculations have been performed using the NOAH software. These calculations have been carried out at the end of the tests with the aim of providing some additional information about the different material behaviour and an assessment of the difference between calculations and measurements. Moreover, this has been used in order to assess the quality of the measurements in comparison with theoretical values.

For this part of the study, updated values obtained through laboratory tests have been considered in order to have accurate material characteristics. In fact, a few points with stabilized temperature have been chosen and the calculation compared with the measurements for these selected points, considering the elastic modulus in function of the layer's temperature registered. The outputs of these calculations were very interesting and quite good correlations with measurements have been found. In some cases, less than 15 % difference has been calculated. Considering all the input parameters that influence the calculation (in situ material not necessary the same as laboratory samples, consideration of fatigue effect, Burmister theory, ..) the results obtained are in good agreement with the measurements. Moreover, the additional calculation permitted to make a sensitivity analysis on the bonding conditions and the effect of the top layer, as well.

Figure C.1.6 gives an example of the calculation results. In this figure, the calculated points are represented with red dots.

![Deformation vs Passages](image)

**Figure C.1.6:** Comparison between calculated and measured deformations for axle B1 (40 % RA)

We can conclude from the ALT study and additional calculations that no negative effect of a high percentage of RA has been observed. Indeed, the mixes with RA showed at least equal performance as the mix without recycling material. Moreover, the tests conducted reached a performance rate and load repetition that is high enough for concluding that the behaviour of the mixes is very good.

**C.1.4 Other tests related to ALT**

In addition to the ALT, additional tests and analysis have been performed on in situ mixes. They are discussed hereafter. More details can be found in [7].

**Wheel tracking tests**

Wheel tracking tests have been conducted on laboratory mixes during the mix design. In order to assess the resistance to rutting of the in situ mixes that have been tested through ALT, different slabs have been constructed during the laying phase. These slabs have then been tested at 50 °C according to the EN 12697-22. Moreover, the rutting resistance under severe conditions have also
been assessed through a test at 60 °C. Indeed a test with extreme temperature has been found relevant as it would be possible to have HMA base layers that could reach very high temperature in some southern European countries, for instance.

The tests results showed very good performance regarding rutting for all mixes. Indeed, the proportional rut depth at 30’000 cycles was always under the limit of 5 % which corresponds to the best category for the future European method at 50 °C (EN 1208-1). The tests at 60 °C permitted to confirm the very good resistance of the different materials regarding rutting, even with severe conditions. Indeed proportional rut depths close to 5 % have been measured.

**Tests on cores**

Different coring campaigns were carried out during the tests at the ALT facility: before the beginning of the tests, after the fatigue tests and at the end of the low temperature tests, as well. At each stage, cores have been taken in undisturbed areas and/or in wheel path, this in order to further analyse the effect of the load and temperature on the pavement characteristics.

The maximum densities of the test sections did not change during the ALT study. The section with 40 % RA had 20 kg/m³ less maximum density compared to the laboratory mix with the same content of RA. The difference is probably due to the larger variability of RA compared to clean aggregate. The maximum densities for the other test sections matched the densities of the laboratory mixes.

The bulk densities were used to calculate the air void content. In the sections with 0% and 40% RA, the air void content were approximately 2.5%, although the measurements indicated that the air void content was approximately 1% higher initially. This recorded decrease could be the result of compaction by the loading tests done, but this conclusion is contradicted by the tests done on drilled cores outside the wheel path after the ALT and low temperature tests had been finalized. These cores had the same air void content as the ones in the wheel path after the ALT. For the section with 25% RA, the air void content was approximately 5%, which is higher than what was recorded in the laboratory studies. Also for this section there is an indication that the air void content was initially higher.

The water sensitivity as measured by the indirect tensile strength ratio, did not change during the course of the ALT. It remained close to 90% for all test sections during the accelerated tests, which is similar to the values recorded for the laboratory mixes.

The stiffness modulus measured at four temperatures did not appear to change during the accelerated loading tests done on the test sections.

Master curves of stiffness moduli were recorded and described by the equation:

$$E = \frac{E_{\text{max}}}{(1 - \text{Exp}(-0.5f_r))^h}$$

which has only two parameters, the limiting stiffness modulus, $E_{\text{max}}$, and the factor $h$. The repeatability limits for these factors for a mean of three samples were determined to be 10% and 0.16, respectively. There were no recorded change above the repeatability limits for $E_{\text{max}}$, or $h$ during the course of the ALT study for the three test sections. The master curves for the tests sections with 0% and 40% RA were also almost identical to the master curves recorded for the laboratory mixes with the same compositions.
Binder analysis
Making a study with reclaimed asphalt, the behaviour of the binder and especially the mix of old and new binder are also important. In order to assess these parameters, a specific study of the binders was carried out and different conditions selected:
- Raw binder with and without ageing
- Binder recovered from RA
- Binder laboratory mixes and ageing using RTFOT
- Binder recovered from in situ mixes
- Binder recovered from laboratory mixes.

Classical tests (pen, R&B) have been performed for each binder condition but also rheological tests like DSR. This analysis showed that the different laboratory and in situ mixes are consistent as rather comparable values have been found. This study also demonstrated that RTFOT ageing corresponds rather well with laboratory production ageing whereas field ageing seemed to be more severe. This can be linked with the high production temperatures.

Tests on big slabs
A specific test in controlled conditions has been performed on big slabs taken in the ALT facility. While the tests in the ALT facility focused on fatigue testing at 15 °C and low temperature tests, it has been decided to investigate the mix behaviour at elevated temperatures. Hence, three large slabs were extracted from undisturbed areas and sent to DRI for testing in the Danish Asphalt Rut Tester (DART). These devices permitted to simulate a rolling load with side wander randomly distributed [7].

For the tests in DART on big slabs, a standard testing has been conducted with 50 kN wheel load and 110.000 load applications. The results have then been compared with the results obtained on Danish motorway pavements. After first tests according to the standard procedure, a further 44.000 loads were applied at an elevated temperature of 50 °C surface temperature / 40 °C bottom temperature, this in order to be sure to reach rutting and analyze the limits of the material, as well. The permanent deformation evolutions for the three slabs are illustrated in figure C.1.8:
As the deformation obtained during the first 20'000 loads will to some extent mainly depend on initial compaction under the wheel load, it cannot be concluded from these data that rutting of the slab with 40 % RA is much less pronounced than for the other two slabs. For a comparison of rutting behaviour of different slabs without this initial disturbance, the increase of rutting from 20'000 to 40'000 loads is calculated. For the different slabs, we obtained a rutting increasing between 0.25 mm (25 % RA) and 0.32 mm (0 % RA). Comparing these results with the Danish standards on highways (between 0.5 and 5 mm), we conclude that rutting susceptibility is much better than for standard motorway pavements. Moreover, no significant difference between the different slabs can be determined. Concerning elevated temperature tests, the slab with 40 % RA had a slightly but not significantly faster rutting development than the slab with 25 % RA.

C.1.5 Conclusions and recommendations

In this innovation, a sophisticated methodology has been used, based on a mix design software, different laboratory performance tests and also ALT testing. Hence, a lot of performance characteristics under several circumstances have been investigated through these tests, to obtain as much as possible information about the performances of the different mixes.

High performance mixtures were obtained with equivalent performance for mixes with RA than mixes without RA, provided that an optimization of the mix design was performed based on the analytical mix design study and on the results of the performance tests. Three designed mixtures were further tested in the ALT-testing facility of LAVOC. No failure was observed in these experiments, and mixes with RA showed equivalent performance as mixes without RA. It was shown that the use of high percentage of reclaimed asphalt in base layers has no negative effect on the laboratory mix performance. ALT, wheel tracking tests, tests in DART and also laboratory tests on cores and binders samples came to the same conclusion that no negative effect of a high percentage of RA could be identified so far.

However, it is important to keep in mind that such a conclusion cannot be extended to all the HMA mixes. Some parameters like the grading curve, recycling material and binder type play a key role in the final properties. Considering these elements, it is recommended to put special attention on the characterisation of the reclaimed asphalt, on the mix design and on the laboratory tests, in order to fully guarantee performance.

According to the results obtained and the good results with the different mixes, it is advisable to make some in situ tests that would permit to validate the results of this research. Another important question is where to put the efficient limit concerning the RA content. Obviously, the encouraging results obtained allow us to think about 50 % or 60 % recycling material. However, further increase of RA content will necessitate a very good control of some parameters such as viscosity may require the use of new techniques as well.
C.2 Innovation 2.1B: Crack-free semi-rigid pavement incorporating two industrials by-products

C.2.1 General scope of the innovation

In semi-rigid pavement structures, the well graded treated aggregate used as a base course always present transversal cracks because:
- it is submitted to different shrinkages: desiccation in the first months and thermal shrinkage during its all life;
- its shrinkage is restrained by the sub-base;
- it’s a rigid material, with an elastic modulus generally higher than 20 GPa.

Due to the traffic and the weathering, the longitudinal cracks inevitably reflect themselves in the wearing course. When the cracks are wide-opened, the interlocking of the pavement blocks on the both sides of the cracks is limited and then the structural efficiency of the pavement is lessened. Moreover, these cracks damage the aesthetic, the evenness and the comfort of the wearing course. They finally facilitate the penetration of water, which accelerates the ageing of the structure.

That is why regular maintenance (cracks silting up) is generally needed, approximately each 3 years. Yet, although it is efficient on the structural point of view, it does not solve the aesthetic and evenness matters. Moreover, maintenance works disturb the road users and represent an important cost.

So cracking appears less and less acceptable by the construction financing authorities and different strategies were developed to avoid its appearance:
- pre-cracking system in the construction phase. Thanks these techniques, it is possible to control the position of the cracks, then to limit the space between two cracks (around 2 meters) and consequently the thickness of the cracks;
- interface anti-cracking systems which try to block the cracks under the wearing course;

Unfortunately, the efficiency at long term of these techniques is not completely demonstrated and generate extra-cost at construction.

In conclusion, well graded aggregate is a durable material but its cracking tendency limits the life duration of semi-rigid pavement structure. So it would be interesting to develop crack free well-graded treated aggregate to obtain long life semi rigid pavement with low maintenance cost. The scope of this innovation is to evaluate the feasibility of such a material thanks the use of two by-products.

The idea is the following:
- some by-products like steel slag display spontaneous swelling behaviour and release lime;
- pozzolans like fly ashes react with lime to give hydrates (CSH) and mechanical properties;
- optimized mixtures of such swelling by-products with inert materials and pozzolans could present structural properties in combination with a controlled swelling, which could overcome the thermal shrinkage.

If such mixtures could be realized, they would allow:
- construction cost savings (as a part of the components are by-products);
- maintenance cost savings (no cracks means thinner structure and no joint maintenance);
- high quality materials savings (by the use of by-products).

These goals are in perfect accordance with the main objectives of the Innovation task which are low cost pavement construction and maintenance techniques.

C.2.2 Bibliographical search

During the production of iron and steel, the iron oxides and some of the metal oxides are reduced forming the metal melt. The remaining oxides will be bound into an oxide melt: the slag. The production of iron and steel is normally carried out as a series of discrete operations with essentially: the reduction in the blast furnace, the steel process using the Basic Oxygen Furnace (BOF) and
Electric Arc Furnace (EAF) process following by the finishing of liquid steel in the secondary metallurgical treatment.

The pig iron production in the blast furnace leads to the blast furnace slag, which properties can be influenced by the cooling conditions: air cooling generates the crystalline air cooled blast furnace slag (ABFS) and rapid cooling with water or even with air generates the glassy granulated blast furnace slag (GBS). Both produce about 250-300 kg per ton of pig iron made (the European pig iron production are more than 60 million tons in 2005), they are fully used respectively as aggregate in road construction or concrete, and as cement compound or as addition to concrete.

Steel slag is produced from the further refining of iron in a Basic Oxygen Furnace (BOF Slag) or from melting recycled scrap in an Electric Arc Furnace (EAF slag). Both produce about 100 kg per ton of steel made and in Europe every year nearly 12 million tons of steel slags are produced. Owing to the intensive research work during the last 30 years, about 65% of the produced steel slags are used on qualified fields of application, today. But the remaining 35% of these slags are still dumped. In France, the dumping rate of steel slags is close to 25%, for an annual production close to 1.7 million tons (in 2005).

Owing to their physical, chemical and mineralogical properties, the steelmaking slags are suitable for various kinds of applications in industrial areas. EAF slag is basically used in road construction (as layer with or without binder) and earthworks (road cover, road base, way's consolidation) and their high resistance to wear successfully promoted their use as mineral aggregates for wearing course in road surfaces. BOF slag has been previously used in the blast furnace to recover the iron in the slag and as lime carrier. Due to the extent requirements on the phosphorous content of steel, BOF slag recycling became more and more restricted and new applications had to be found. BOF slags are used in road construction, as aggregate for concrete or for hydraulic engineering, as fertilizer in agriculture, as pollutant removing filter or soil stabilization.

The BOF- and EAF-slags from different sources within Europe, are generally comparable and independent of their producers. Differences arise from the use of dolomite rather than lime as fluxes with the effect of a higher MgO-content in the slag. BOF- and EAF-slags are calciumsilicatic with a range of CaO between 42 and 55%, and a range of SiO2 between 12 and 18%. EAF-slags comprise CaO between 25 and 40% and 12 to 17% SiO2. Their MgO-content may be higher due to the reactions with the refractory lining. The main mineral phases of BOF- and EAF-slags are dicalciumsilicate, dicalciumferrite and wustite. The content of free lime and free MgO is the most important component for the utilisation of steel slags for civil engineering purposes, with regard to their volume stability. In contact with water, these mineral phases will react to hydroxides. Depending on the rate of free lime and/or free MgO hydration, it causes a volume increase of the slag mostly combined with a disintegration of the slag pieces and a loss of strength. So, the volume stability is a key criterion for using steel slags as a construction material.

Then, for many steelworks, a significant proportion of this slag will be landfilled, and of the materials dumped, it will often form the largest proportion. With increasing pressure in many countries for greater use of secondary aggregates to preserve the natural resources, steels slags offer a promising and relative abundant alternative. During the last twenty years, the problem of volume stability was the main objective of the research work on steel slag in Europe. Today, this problem can be avoided with a suitable weathering of the slag in order to favour the free lime hydration. Such slag can then be safely treated with bituminous binder for road wearing course notably. Moreover, steelmaking slags can be used at all levels (unbound in the lower layers, bituminous-bound in the upper courses, and as a surface dressing) (Piret et al., 1982). Yet the slag maturation still remains problematic because of the associated handling or the pressure imposed by the environmental policies. This is why, it is today necessary to promote a new approach for the valorisation of steel slag in civil engineering, considering slags of lower quality and trying to convert slag disadvantages into positive aspects.

The combination of BOF slag (with or without weathering) with other materials is another way to limit the volume instability. Numerous studies dealing with the composition of mixes of BOF slag aggregates with other materials, such as granulated blastfurnace slag, municipal solid waste incinerator bottom ash, fly ash, used for road construction can be found in the literature. For example, by mixing 70-85 % of weathered BOF slag with 15-30 % of granulated blastfurnace slag, a road base
was produced without significant expansion damage. Moreover, a slowly setting composition is obtained which provides, at low cost, a quality road base which can be considered as semi-rigid in comparison with concrete bases (Piret et al., 1982). Best (1987) developed a well graded composite similar to the French 'graves-laitier', but containing both air-cooled blast furnace slag (5-20 mm, 57 vol.%), LD slag (0-5 mm, 28 vol.%) and quenched blast furnace slag (15 vol.%). The cementitious action of the quenched blast furnace slag, activated by the free lime embedded in the BOF slag binds the mix. Such a pozzolanic reaction consumes the free lime non-expansively, lowering the tendency of the aggregate for expansion. This new material referred to as "self-binding slag composites" presents the advantage to minimise the need of chemical activators owing to the free lime present in the slag. Juckes (1991) confirmed that the dilution of the BOF slag and the associated cementitious reaction lead to a limited expansion of such mixes, arguing in favour of an absorption of the expansion attributed to the BOF slag aggregate by the semi-rigid environment. This was recently shown again in a recent study made by Tikkakoski et al., (2005).

In these previous studies, the mix optimization was carried out in the laboratory by seeking the best geotechnical performances. It was assessed by classical techniques used in road engineering such as measurement of compressive strength, Proctor optimum, CBR, freeze-and-thaw and rutting resistance or volumetric stability. However, from a practical point of view, the use of BOF slag aggregates in road construction remains unusual because of the uncertainties about volume stability. In fact, this volume stability depends on numerous factors such as proportions of the different components, free lime content of BOF slag, residual potential volume increase after weathering. So, whereas such a technique of combination with other granular materials offers a promising way of valorisation for BOF slags in road, it requires more technical specifications and needs development of tools to be able to predict and ensure the volume stability of the obtained mixes.

In that context, Deneele et al. (2005) have proposed a new method to predict the swelling of any combination of swelling aggregate and inert aggregate fractions. This method is based upon the Compressible Packing Model developed at LCPC which was implemented in the software René-LCPC (Sedran and de Larrard 1995, Sedran and de Larrard 1996, Sedran 1999, de Larrard 1999). This software was first developed for the optimization of concrete, but the framework of the model is much more general and can be used for other types of granular packing or granular suspensions. The software needs three types of data for each granular component: the dimensions of the particles (as given by the grading curve), the specific gravity and the packing density. It then predicts, for any combination of the fractions, either the compaction index from the packing density, or the packing density from the compaction index. This latter parameter is a characteristic of the placing method tabulated thanks calibrations (for example, 9 for packing under vibration and 20 kPa pressure). The software was widely validated in the field of the packing density of dry granular mixtures and gives an error lower than 1% in absolute value, in comparison with the experiments.

The authors have verified that the weathering of compacted BOF slag samples in an accelerated test with a steam apparatus (see below) induces swelling and grading changes of the material. The grading change can be directly evaluated by sieving, and swelling can be interpreted as a difference in packing density of the mix before and after weathering. In fact, swelling S% can be expressed as following:

$$ S = 100 \left( \frac{V_f - V_i}{V_i} \right) = 100 \left( \frac{C_i \rho_i}{C_f \rho_f} - 1 \right) $$

Where:
- $V_i$ and $V_f$ are respectively the volume of the sample before and after weathering
- $C_i$ and $C_f$ are respectively the packing density of the sample before and after weathering
- $\rho_i$ and $\rho_f$ are respectively the specific gravity of the BOF before and after weathering

Deneele et al. (2005) have then introduced the properties of the different classes of grains of BOF slag into Rene-LCPC for two cases: before and after weathering. From these data, they were able to calculate the packing density before and after weathering of several mixes made of BOF slag and Air-cooled Blastfurnace slag on one hand and BOF slag aggregates and limestone on the other hand. They were then able to calculate a theoretical swelling according to the previous equation. These
results confirmed to be in good agreement with the experimental data for mixes containing less than 50% of BOF slag. For higher content of BOF slag, the theoretical swelling was overestimated. According to the authors, this is due to the fact that for high volume of BOF slag, the mixes were less porous so that, during the accelerated swelling test, the steam could not penetrate into the packing and then the experimental swelling was lower than could be expected.

Using the approach proposed in that study, it is then possible to optimize the mix composition of BOF slag with inert aggregate, in order to have an acceptable and limited swelling.

Finally, combining a BOF slag with a fly ash and an inert aggregate could be a good way to obtain a well-graded aggregate with noticeable mechanical properties and a controlled swelling. In fact the fly ash may react as a pozzolan with the lime released from the BOF-slag and the swelling could be controlled by the introduction of an inert aggregate thanks the use of Rene-LCPC.

C.2.3 Preliminary feasability tests

Materials selected for this study were:
- a 0-10 mm fresh Basic Oxygen Furnace slag (the same as presented in Deneele et al. (2005) ) representative of French production
- a 0/6 mm and a 10/20mm "Le Boulonnais" limestone
- coal fly ashes from Surchiste company
- a quicklime (activator)

A first step is to rapidly identify promising well graded aggregate mixtures producing at the same time some swelling to counteract the effect of the different shrinkages, and noticeable mechanical performances. Both are needed to produce a road layer with bearing capacity and no cracks. In case of success, the selected mixtures should be tested in the same way but at 20°C and at longer term. In fact, 20°C is more representative of the temperature during the road life, moreover the temperature has a great influence on the chemical reactions involved: expansion of free lime, pozzolanic reaction between lime and fly ash. So the evolution of swelling compared to that of the elastic modulus and tensile strength may change in great proportions compared to tests at 90°C.

In a first approximation we will aim at a splitting tensile strength between 0,5 to 1 MPa which corresponds to GC1 or GC2 class of cement treated well graded aggregate in EN 14227. A rough calculation of swelling to be aimed at, can be made with the following assumptions:
- the well-graded aggregate have a long term splitting strength of 1 MPa and an elastic modulus of 20 000 MPa;
- the well-graded aggregate has a thermal expansion coefficient around 10^{-6}/°C, and the layer may be submitted to a maximum temperature variation in a range of 30°C (between night and day and summer and winter). This lead to a maximum 300 10^{-6} shrinkage between hot period and cold one;
- autogeneous shrinkage of the well graded aggregate is negligible and its drying shrinkage is around 300 10^{-6}.

The well graded aggregate is then submitted to a maximum 600 10^{-6} strain, yet the acceptable strain at long term before cracking is 1/20000=50 10^{-6}. So the swelling should be at least 250 10^{-6}. In fact this value is probably strongly underestimated because swelling may occur at young age while the well graded aggregate has a low value of elastic modulus. In that case strains generate few compressive strength to counteract the effect of thermal shrinkage which occurs during the all life of the road when the elastic modulus is higher. Moreover part of the benefit of the swelling may be lost with time due to relaxation in the material.

As explained before, the theoretical estimation of the mixes swelling is based on the difference in packing density between a fresh mix and the same mix after weathering, calculated with René–LCPC software. The calculation of these packing densities requires the preliminary following data for the different components:
- the grading curve;
- the specific gravity;
- the packing density.
For the BOF slag, these properties were determined before and after weathering. As the other components were assumed to be inert, their characterization was made only without weathering. The effect of weathering on the slag particles is assumed to be dependant on their size. So, for a better characterization of the slag before and after weathering, its was first separated by sieving it in five fractions. Each fraction was then characterized individually. For particles coarser than 80µm, the weathering procedure was carried out according to the procedure in EN 1744-1, paragraph 19.3, but a modification was brought to the classical procedure since samples were subjected to a very slight compaction with the vibrating table. The steam test equipment available at LCPC is described in figure C.2.1. Another weathering procedure was applied to the finest aggregate class. In fact, when submitting the 0/0.08 mm fraction to the steam test, we observed a sudden raising of the slag sample and covering surcharge, which exceeded the height of the cylinder. Consequently, we have chosen to weather this finest fraction by immersion in water at 110°C (chamber temperature) during five days.

The results are shown below.

Figure C.2.1: Principle of the steam apparatus: the sample is submitted to a steam flow while its height is monitored versus time
In conclusion, it can be observed that the weathering of slag mainly leads to:
- a decrease of the packing density (mean value -7%);
- almost no change in the specific gravity (mean value +2,5%);
- aggregation of small particles and splitting of coarse particles.

The measurements made on the BOF-slag have shown that they contain approximately a total of 10% of free lime and that only 50% of this lime can be hydrated. Moreover the European norm EN 14227-3 and the French one NF 98118 suggest for classical fly ash/lime well graded aggregate the following mean dosages:
- 10% of fly ash (between 8 to 12%)
- 1,7% of CaO (1,4 to 2,1) or 2,5% of Ca(OH)\(_2\) (2 to 3%)

These data lead to select a ratio of CaO/FA=0.17 in our mixes. When possible the CaO will be bring by the BO-slag. Which means a ratio BOF slag/ FA= 3.4.

René-LCPC software was used to generate different preliminary recipes (see Table C.2.2) with different theoretical swelling. Note that these swelling are probably overestimated here as the model...
was developed for packing without any hydraulic or pozzolanic reaction. In the present case, strength and so elastic modulus of the different mixes are expected to increase, thus restraining the swelling.

The assumptions of the calculations were the following:
- the packing index corresponding to the moulding of the sample is fixed 12. This value was fitted from the one obtained for packing at optimum proctor by Pouliot et al (2001).
- the calculations are made with no wall effect to be representative of free swelling of a huge sample of material.

In the first series (F1, F2, F3) we have:
- fixed the BOF-Slag/FA ratio to 3.4. By this way we have made the assumption that all the quick lime necessary for the fly ash was furnished by the slag;
- varied the BOF-Slag/limestone ratio to generate different swelling values. In a first time we have only selected the 10/20 limestone in order lessen the granular interaction with the slag particles. This theoretically allows moderate values of swelling even with high volume of slag ("the slag particles expand between the coarse inert particles with few effect on their packing arrangement).

In the second series (F1, F4, F5) we have:
- the same BOF-Slag/limestone as in the first series to keep almost the same swelling level;
- imposed a content of 10% of fly ash to aim at higher theoretical mechanical properties (before swelling). In that case the quick lime furnished by the slag is not sufficient and we have to add directly a pat of it to maintain the CaO/FA ratio constant.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Fly Ash (%)</th>
<th>Lime stone 10/20 (%)</th>
<th>Lime stone 0/6 (%)</th>
<th>Steel slag 0-10 (%)</th>
<th>CaO (%)</th>
<th>CaO/VC (except CV and CaO)</th>
<th>LD 0-10 (%)</th>
<th>Initial theoretical porosity</th>
<th>Initial dry apparent density (kg/m³)</th>
<th>Theoretical swelling (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1</td>
<td>10</td>
<td>56</td>
<td>0</td>
<td>34</td>
<td>0.17</td>
<td>37.8</td>
<td>0.1959</td>
<td>2162</td>
<td>26.3</td>
<td></td>
</tr>
<tr>
<td>F2</td>
<td>7</td>
<td>69.2</td>
<td>0</td>
<td>23.8</td>
<td>0.17</td>
<td>25.6</td>
<td>0.2455</td>
<td>2025</td>
<td>1.29</td>
<td></td>
</tr>
<tr>
<td>F3</td>
<td>4</td>
<td>82.4</td>
<td>0</td>
<td>13.6</td>
<td>0.17</td>
<td>14.2</td>
<td>0.3105</td>
<td>1846</td>
<td>0.60</td>
<td></td>
</tr>
<tr>
<td>F1</td>
<td>10</td>
<td>56</td>
<td>0</td>
<td>34</td>
<td>0.17</td>
<td>37.8</td>
<td>0.1959</td>
<td>2162</td>
<td>26.3</td>
<td></td>
</tr>
<tr>
<td>F4</td>
<td>10</td>
<td>66.4</td>
<td>0</td>
<td>23.1</td>
<td>0.5</td>
<td>25.8</td>
<td>0.2226</td>
<td>2070</td>
<td>1.30</td>
<td></td>
</tr>
<tr>
<td>F5</td>
<td>10</td>
<td>76.3</td>
<td>0</td>
<td>12.6</td>
<td>1.1</td>
<td>14.2</td>
<td>0.2634</td>
<td>1943</td>
<td>0.60</td>
<td></td>
</tr>
</tbody>
</table>

Table C.2.2: Theoretical swelling of different well graded aggregate calculated with Rene-LCPC software

Table C.2.3 shows the mechanical performance obtained. After casting the cylinders were slowly heated to 80°C during one day, then cured at 80°C for 5 days and finally slowly refresh to 20°C during one day. The cylinders were preserved from desiccation during curing.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Fly Ash (%)</th>
<th>Lime stone 10/20 (%)</th>
<th>Lime stone 0/6 (%)</th>
<th>Steel slag 0-10 (%)</th>
<th>CaO (%)</th>
<th>ρdopm (t/m³)</th>
<th>Wopm (%)</th>
<th>Packing density</th>
<th>Splitting tensile strength (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1</td>
<td>10</td>
<td>56</td>
<td>0</td>
<td>34</td>
<td>0</td>
<td>2.23</td>
<td>7.5</td>
<td>0.829</td>
<td>0.16</td>
</tr>
<tr>
<td>F2</td>
<td>7</td>
<td>69.2</td>
<td>0</td>
<td>23.8</td>
<td>0</td>
<td>2.13</td>
<td>5.5</td>
<td>0.792</td>
<td>0.16</td>
</tr>
<tr>
<td>F3</td>
<td>4</td>
<td>82.4</td>
<td>0</td>
<td>13.6</td>
<td>0</td>
<td>2.02</td>
<td>3.5</td>
<td>0.754</td>
<td>0.19</td>
</tr>
<tr>
<td>F4</td>
<td>10</td>
<td>66.4</td>
<td>0</td>
<td>23.1</td>
<td>0.5</td>
<td>2.2</td>
<td>6.8</td>
<td>0.824</td>
<td>0.35</td>
</tr>
<tr>
<td>F5</td>
<td>10</td>
<td>76.3</td>
<td>0</td>
<td>12.6</td>
<td>1.1</td>
<td>2.16</td>
<td>6.2</td>
<td>0.819</td>
<td>0.49</td>
</tr>
</tbody>
</table>

Table C.2.3: Mechanical properties of the mixes at the optimum proctor
The results show that only limited performances in splitting tensile strength are reached with the mixes in the first series, where no quick lime is added. This can be explained in F1 by important swelling. The graphic C.2.3 shows that at the end of curing, cracks have appeared in the cylinders as well as brown zones which are typical of CaO hydration in the slag aggregates. In F3, the low performance is probably due to the low content of fly ash. Moreover, in that series, it is possible that even if pozzolanic reactivity is accelerated by the increase of curing temperature on one hand, the diffusion of the quick lime liberated from the slag aggregate is not accelerated on the other hand. The consequence may then be that the fly ash particles are not in contact with lime and that pozzolanic reaction does not occur much in the first week.

In the second series, F5 reach 0.5 MPa in tensile strength. This result is encouraging as theoretical swelling is around 6000 $10^{-6}$. As it can be seen in figure C.2.4, no cracks was visible on the sample after curing.

![Figure C.2.3: Pictures of the F1 mix after curing. Segregation is due to the gap graded skeleton. Cracks are visible as well as brown scaling due to quicklime hydration in slag particles](image-url)
In a first approach, we intended to use the steam machine presented in figure C.2.1 to make the swelling measurements in accelerated conditions. This apparatus allows a steam curing at 100°C and was already equipped with a displacement indicator. Yet after several trials it appears that the steam was not able to go through the sample because of the low porosity of the samples and that the displacement observed were due to a piston effect and not to swelling. Then a new way of measuring the swelling was tested.

Using the procedure classically used for concrete samples, the idea is to include two steel inserts on in the two faces of a Ø16x 32 cm. The distance between these two inserts will then be measured at different ages with the measurement device presented in the figure C.2.5.

Figure C.2.4: Pictures of the F5 mix after curing. Segregation is due to the gap graded skeleton. No cracks are visible.

Figure C.2.5: Tool realized for length measurement of Ø16x 32 cm of well graded aggregate. The zero is made thanks an invar bar. The contact with the inserts of the sample or the bar is ensured by two steel balls.
Compared to concrete we have here two main difficulties:
- samples are compacted and not cast. A new device must be design to be included in the mould of the VCEC compacting machine, to be able to put the insert in the samples;
- measurements have to be started at a young age, when the well grade aggregate has almost no tensile strength. So the anchorage of the insert has to be well studied.

The graphic C.2.6 describes the system developed to include the inserts during compaction:
- a steel plate (9) is included in and empty millboard mould to which an insert (8) is screwed;
- the well-graded aggregate is included in the mould;
- another steel plate (7) with an insert is put on the material;
- the material is compacted with the piston (4).

Figure C.2.6: Principle adopted to include the insert while compacting

After several tests, the insert shown in figure C.2.7 was selected. It is made of stainless steel with a conic form with flat parts. This geometry allows a good filling of granular material near the insert, avoids movement and rotation of the insert at early age. A preliminary test has shown this new version of inserts is well anchored and allows a stable evaluation of the sample length.
C.2.4 Conclusions

On the basis of the bibliographical analysis we have selected a set of constituents which could be relevant to produce well-graded aggregates with a swelling behaviour and mechanical performances similar to that of cement treated well-graded aggregates. We also have described a method potentially useful to make the design of such mixes.

Preliminary tests were done in hot conditions (higher than 80°C) in order to accelerate the mechanical performance increase as well as swelling in order to verify the feasibility of the innovation proposed. Unfortunately we met technical difficulties to develop a reliable free swelling test adapted to such well graded aggregate mixes. A promising prototype has been designed and tested but it still needs improvements.

Promising results were obtained as far as mechanical performance were concerned but we were not able to quantify the swelling of the mixes though swelling of some samples was obvious (crack due to lime hydration were visible).

The feasibility of the proposed concept could then not be verified during this project and need more research.

Figure C.2.7: Selected version of insert, with flat plates on the conic part, is well anchored
C.3 Innovation 2.2: Roadway perception technology using the infrared know-how

Nowadays, the progress made out in the field of on board electronics and sensors (computer, camera, etc…) favours the emergence of assistance systems for driving road perception. Research works undergone in the field of image processing applied to stereoscopic image acquired on board of a vehicle allow obstacle detection. Furthermore, recent research works lead to the possibility of computing a visibility distance under foggy conditions, by using a simplified extinction light model coupled to a specific image processing algorithm. Nonetheless, the efficiency of these works depends on the information available inside the image and they are all based on the use of imaging system in the visible spectrum. As an example, to spread out in night conditions distance visibility computing and obstacle detection, you’d have to take into account how the vehicle can light up the roadway assuming that no bad weather conditions will be encountered. Traffic condition at night (lighting interference) and weather forecast at night or in daylight (rain, fog, snow, sunny nightfall on wet pavement) do not favour part of these methods based on the use of the visible spectrum.

So, even if overhang in research are observed as well in the field of on board sensors as on the road vision perception models under more or less favourable weather conditions, there remains an investigation field, which to our knowledge was poorly examined, to increase the efficiency of the roadway perception device. It’s the potential of the infrared spectrum. In this field, one will note the appearance of vehicles (top-of-the-range) marketed with infrared vision device. But, in situ performances of such systems remain dependent on the intrinsic and extrinsic properties of the road. Results available in literature are frequently presented with a qualitative analysis made on infrared images after treatment. First results are available with active infrared systems (infrared system coupled with vehicle headlights), but results analysis still remain a qualitative analysis of the image produced. To our knowledge, no investigation on the properties of the road infrastructure in the infrared spectrum has been published. So this aspect in the infrared vision for automotive application has to be investigated to see if the performance of such vision system could be enhance by acting on the infrastructure.

The innovation developed in the present work package focuses on the study of the pavement and road sign thermo-optics properties in the infrared spectrum applied to road perception under various weather conditions (restricted to fog). To reach this aim, measurement methods to characterise, in the infrared spectrum, used materials or new participative ones (spectral and directional thermo optics properties) were examined, taking into account the fact that road perception will be made by on board infrared vision device. A simplified transmission model was developed and used to evaluate the enhancement of performance that could be reached with on board infrared vision by acting on the infrastructure. Few tests were done to compare simulation with experimental results on real test site and in laboratory. These works leaned on the experience of LCPC and its partners (LRPC of Angers, Autun, Clermont-Ferrand and Nancy) in simulation and measurement experimentation available in the visible spectrum and the know-how in infrared system applied to winter time experimental pavement monitoring of some test site in France.

We can summarise the overall objectives of our research investigation in this innovation task by the following sentences:
- Enhance safety for drivers
- Acting on road infrastructure material’s to turn them cooperative for on board infrared vision system
- Reducing on road trials
- Enhance knowledge in roadway perception through on board infrared vision systems in bad weather conditions and/or during night time

C.3.1 State of the art
Looking at recent completed European projects on infrared vision in automotive1,2, it could be said that its use had mainly focused on the detection of animals, pedestrians and obstacles at night or under poor weather conditions with degraded visibility. For instance, published works3 used fusion algorithm and image processing applied to data acquired with different on-board sensors including infrared system. To our knowledge, for standard infrared on-board detector available on the market for automotive application, pedestrian detection and shape recognition research works were developed
for a set of distance to detectors ranging from 5 to 25 m. The infrared focal plane array (IRFPA) was of 320 x 240 sensitive elements (e.g., see 2) with a pitch of 45μm. Nonetheless, it was also shown that the use of fusion algorithm with visible spectrum images enhanced performances in the detection of pedestrians. But in those approaches, infrastructure thermo-physical properties had not been investigated to enhance the road driver visibility as, for instance, through the use of cooperative materials for on board infrared vision system. To drive the research in such a direction, investigation on infrastructure behaviour in the thermal infrared domain required the evaluation of the radiation heat balance of the whole system (i.e. the road and its environment) at different periods of the day and for different infrared-vision-system configurations. Considering what was done in the domain of teledetection, commercial tools existed, but investigations on thermo-optical properties were also mandatory due to the increase of the spatial resolution of new sensors. Software for radiation heat balance in enclosure were also available. Nevertheless, these tools were more dedicated to heat transfer or airborne vision approaches than for infrared vision applied to automotive.

Thus, to investigate possible modifications of road infrastructure to enhance infrared vision for drivers, the research work approach developed in NR2C couples numerical simulations (in infrared and visible spectrum) with specific experiments as laboratory characterizations of road material infrared radiative properties, on roads and in fog tunnel infrastructure visibility in night conditions.

### C.3.2 Study of measurement methods in situ for infrared properties

The knowledge of infrared emissivity of pavements is mandatory to get a proper understanding of road weather phenomena occurring on pavement surface. This is an important parameter for thermal exchanges between pavement and atmosphere for road surface status forecast. Some models have indeed shown the relevance of emissivity on pavement surface temperature. A change of 0.92 to 0.98 could roughly induce a 1.3°C temperature change according to the CESAR/GELS model, that could have incidences in winter maintenance. This parameter is also important in thermographic techniques since emissivity allows to go from a luminosity temperature to a surface temperature. Such knowledge relied so far on literature data. Some studies aimed at the determination of physical properties of the various pavement materials in France, along with other materials used in the road infrastructure. An experimental setup has been designed to measure the total directional emissivity in the 1-20 μm spectral band, using a 5°C-amplitude thermal modulation technique. The undertaken work has consisted in determining the experimental conditions to measure this parameter, and evaluate the influence of factors such as the thermal modulation frequency. Then some measurements were done with the experimental setup and a FLIR S65 infrared camera.

Experimental apparatus for total directional emissivity measurement (fig. C.3.1) has shown a good ability in the measurement of this parameter in the 1-20 μm spectral bandwidth. The thermal modulation frequency and the measurement duration could be adjusted to be adapted to on-site measurements. The repeatability of the measurement is correct. Measurements have been done on a large range of emissivity values, with various surface composition and roughness.

![Figure C.3.1: Description of the experimental device (left) – Photography of the apparatus (right)](image)

The emissivity measurement does not depend on modulation frequency, which changes the measurement duration. The greater is the frequency, the shorter is the duration. Nevertheless, the
thermal inertia of the infrared source has to be considered. The greater the frequency, the shorter the available time for the infrared source to dissipate the energy accumulated during the heating phase. A shorter frequency could also affect the thermal balance of the infrared source due to its inertia, affecting the accuracy of the measurement. This could cause a temperature drift, this latest not being ambient anymore. A 12.5 mHz appeared to be a good compromise, with an acceptable measurement duration. A calculation module is included in the control of the measurement device at the end of each measurement period. Once the emissivity has reached a stable value, the measurement could be considered as completed. Measurements run on materials of road infrastructure materials (Fig. C.3.2 and Table C.3.1) have lead to values ranging around 0.95 (except steel parts which were 0.27). Complementary measurements obtained with a FLIR S65 infrared camera have given a directional emissivity of a semi-granular pavement of 0.85 within the 8-14 µm spectral band. This emissivity was stable with temperature and considered directions (15° et 75° with respect to an horizontal plane).

![Figure C.3.2: Examples of road samples – M2 (left) and M3 (right)](image)

**Table C.3.1:** Measurement result examples between 1-20 µm

<table>
<thead>
<tr>
<th>Sample</th>
<th>Alumina K x (10^3 K mV^-1)</th>
<th>Black paint εvalidation</th>
<th>Road sample M2</th>
<th>Road sample M3</th>
</tr>
</thead>
<tbody>
<tr>
<td>M2</td>
<td>6.30</td>
<td>0.72</td>
<td>6.22</td>
<td>6.30</td>
</tr>
<tr>
<td>M3</td>
<td>6.42</td>
<td>0.72</td>
<td>6.22</td>
<td>0.72</td>
</tr>
<tr>
<td>ΔT (°C)</td>
<td>3.69 ± 0.04</td>
<td>3.73 ± 0.02</td>
<td>3.69 ± 0.02</td>
<td>3.70 ± 0.02</td>
</tr>
</tbody>
</table>

Roughness and surface composition could be linked to wearing of the pavement because of the traffic, in particular with semi-granular structures. Some measurements run on such a structures of increasing roughness coated with a material of known emissivity (Table C.3.2) have shown an emissivity change that could go up to 100% with respect to a smooth surface.

**Table C.3.2:** Surface composition incidence on its emissivity

<table>
<thead>
<tr>
<th>Material composition</th>
<th>Measured emissivity (ε)</th>
<th>Emissivity variation (%)</th>
<th>ΦBF (%)</th>
<th>ΦDI (%)</th>
<th>Computed emissivity</th>
</tr>
</thead>
<tbody>
<tr>
<td>black adhesive tape</td>
<td>0.95 ± 0.01</td>
<td>0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Al foil with black</td>
<td>0.76 ± 0.01</td>
<td>19</td>
<td>81</td>
<td>19</td>
<td>0.76</td>
</tr>
<tr>
<td>adhesive tape cross</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>stripes</td>
<td>0.37 ± 0.01</td>
<td>61</td>
<td>44</td>
<td>56</td>
<td>0.42</td>
</tr>
</tbody>
</table>

![File: Deliverable2.2 Synthesis report-Rev5.doc](image)
Furthermore, some artificial surface composition heterogeneities have been created, and could created 60% emissivity variation compared to homogeneous surface (Table C.3.3). A great emissivity contrast is mandatory to obtain a great emissivity variation.

<table>
<thead>
<tr>
<th>Material</th>
<th>Emissivity</th>
<th>Standard deviation</th>
<th>Emissivity variation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silver paint</td>
<td>0.34</td>
<td>0.01</td>
<td>0</td>
</tr>
<tr>
<td>on a smooth surface</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silver paint</td>
<td>0.42</td>
<td>0.01</td>
<td>22</td>
</tr>
<tr>
<td>on a smooth pavement</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silver paint</td>
<td>0.69</td>
<td>0.02</td>
<td>101</td>
</tr>
<tr>
<td>on a rough pavement</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table C.3.3: Roughness incidence on the emissivity

The developed measurement system and results presented in the two last tables will be useful to predict effect of modified emissivity, for instance by adding local patches on pavement surface using a reflective paint in the infrared, on to its perception through an infrared vision system. Nevertheless, the spectral bandwidth used for measurement is still too large with regards to the more restricted spectral bandwidth of infrared vision systems on the market.

C.3.3 Study of a simplified model for atmospheric infrared transmission in fog for road meteorological conditions

A numerical model based on Mie theory\textsuperscript{9,10} has been developed. It incorporated several macroscopic fog properties\textsuperscript{11}: scattering, absorption, and extinction coefficients. The model was used to compute fog optical properties with experimental data (water droplet size distribution) acquired in fog tunnel. Furthermore, to reproduce basic scenes in the visible spectrum experimentally realized in a fog tunnel, simulations based on Monte-Carlo technique have been developed. Among the laws used to simulate particle size distribution, the gamma modified distribution of the droplets is the most used in fog case,

\[
dN\left(\frac{d}{r}\right) = n(r) = a r^{(\alpha)} e^{-b r^\gamma} \left[ cm^{-3} \cdot \mu m^{-1} \right]
\]

where \(n(r)\) is the number of droplets per classes of radius \(r\), and \(a, \alpha, b\) are parameters used to adjust the model on to observations. A typology for two types of fog, advection (convection with moist air over a cool surface) and radiation, with parameter values could be found in Ref. 12. These parameter values are reported in Table C.3.4.

<table>
<thead>
<tr>
<th>Type of fog</th>
<th>Model</th>
<th>(a)</th>
<th>(\alpha)</th>
<th>(b)</th>
<th>(r_m(\mu m))</th>
<th>(N(cm^{-3}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>advection</td>
<td>1</td>
<td>0.027</td>
<td>3</td>
<td>0.3</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.0659</td>
<td>3</td>
<td>0.375</td>
<td>8</td>
<td>20</td>
</tr>
<tr>
<td>radiation</td>
<td>3</td>
<td>2.37</td>
<td>6</td>
<td>1.5</td>
<td>4</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>607.5</td>
<td>6</td>
<td>3</td>
<td>2</td>
<td>200</td>
</tr>
</tbody>
</table>

Table C.3.4: Parameters to perform fog water droplets size distributions

Models 1 and 2 represent heavy fog for meteorologists (Visibility = 110 m and 160 m). Although the total particle number \(N\) is low, their mode radius (10 and 8 \(\mu m\)) implies a strong scattering. Models 3
and 4 represent moderate fog conditions for the different types (Visibility = 200 and 330m), with greater particle number (N = 100 and 200 cm⁻³) but droplet size much smaller (rₘ = 4 and 2 μm).

However, these models do not correspond to experimental conditions in Clermont-Ferrand fog tunnel where meteorological visibility could be as low as 6 m with a higher number of particles per classes of radius n(r). Figure C.3.3. (left) presents water droplet size distributions measured in fog tunnel for different meteorological visibility, "V".

**Figure C.3.3:** Water droplet distribution measured data (left), extinction coefficients computed (right)

Measurement values have been obtained after complete saturation in water droplet in the fog tunnel and acquired at different times during fog natural dissipation. Visibilities "V" reported were measured with a transmissionmeter located in the fog tunnel. Extinction coefficient is then computed for different wavelengths using the following formula,

\[ K_{ext} = 10^{-3} \int_0^\infty Q_{ext} \left( \frac{2\pi}{\lambda}, m \right) \pi^2 n(r) dr \left[ km^{-1} \right] \] (2)

where \( Q_{ext} \) is extinction efficiency depending on wavelength \( \lambda \), droplet radius \( r \) and refraction index of the medium \( m \) (e.g., see 13). Results obtained for different water droplet size distributions (i.e., meteorological visibility) and wavelength ranges from visible to far infrared are shown in Figure C.3.3. (right). Dashed curves represent scattering contribution for the total extinction. In visible range, extinction and scattering coefficients could be merged (absorption negligible). However, for the far infrared range absorption is no more negligible.

In the visible spectrum, scene image simulations were based on Monte-Carlo technique. The developed simulator asked to choose wavelength and fog droplet radius. From these two parameters, the phase function was calculated and scattering directions were distributed according to probability density. Influence of water droplet size of each fog type for a same meteorological visibility could then be studied. Figure C.3.4. shows simulation results for meteorological visibilities of 22 m and 30 m, with 1 μm water droplet radius, corresponding to experimental conditions available in the fog tunnel.
Visual analysis of simulated images presented in Figure C.3.4 shows a rapid fall down of the contrast between road sign panel (placed at 30 m) and background of the scene.

C.3.4 Development of an infrared simulation tool for simplified road geometries

Simulation tool developed for simplified road geometry vision with an IRFPA incorporates three main steps to generate the incidence map that could be viewed through an infrared camera looking at the road. Nevertheless, at that stage of our development we didn’t introduced a fourth step to determine thermal contrast detected by taking into account IRFPA detector technology (quantum or thermal) as proposed by G. Paez and M. K. Scholl14 in their works.

The first step consisted in meshing the real world (the road and its surrounds) according to the location of the infrared camera in the scene and its internal properties. It was done by projection onto the ground, assumed as a plane surface15, of each sensitive element of the IRFPA studied. Displacement of the optical center and optical deformation were neglected when mesh was generated.

The second step is the positioning of the road and its equipment in the real world and linking it to the mesh generated in the first step. To simplify this work, the geometry of the road was constrained to linear segments. Three dimensional objects that were eventually introduced on the road scene were reduced to parallelepipeds.

The third (and last) step was the evaluation of the incidence received from the scene, by each sensitive element of the IRFPA in its spectral operating range. Each element of the scene was modeled as a diffuse–gray plane surfaces in the wavelength band and whole scene was supposed to be at the local thermal equilibrium. The radiosity method16 was used to compute radiation heat balance at road scene level. As night vision was the topic of interest, Sun incidence was not taken into account in the energy balance. The problem was reduced to heat radiation exchange in a virtual enclosure with or without 3D objects. In the first approach, radiation coming from vertical object was neglected; the sky was also taken into account and assumed to behave as a blackbody.

The local incidence on a sensitive element (cell) of IRFPA was then determined using the following relation between net radiation, configuration factor, transmission coefficient and incidence:

\[ E_{ij,\lambda_1 \rightarrow \lambda_2} = \tau J_{ij,\lambda_1 \rightarrow \lambda_2} F_{ij} \left[ Wm^{-2}\right] \]  

with \( \lambda_1, \lambda_2 \) spectral band, \( \tau \) transmission coefficient, \( F_{ij} \) configuration factor between surface of road viewed and IRFPA cell, \( J_{ij,\lambda_1 \rightarrow \lambda_2} \) net radiation of road surface viewed.

Different approaches were proposed in literature to compute the configuration factor16 that intervene in Eq. (3) for local incidence estimation. Among them there is the contour integration method17, based on...
the application of the Stoke’s theorem. It was chosen for implementation. It reduced the multiple integration over a surface area to a single integration around the boundary of the area. Its expression for the configuration factor of $A_2$ (surface viewed in the road scene) to $A_1$ (IRFPA sensitive element considered) is:

$$ F_{21} = \frac{1}{\pi A_2} \int \cos \theta_1 \cos \theta_2 \frac{dA_1 dA_2}{r^2} = \frac{1}{2 \pi A_2} \int \int \left( \ln ||S|| dx_1 dx_2 + \ln ||S|| dy_1 dy_2 + \ln ||S|| dz_1 dz_2 \right) $$

Where $\ln ||S|| = \ln \left( \sqrt{(x_2 - x_1)^2 + (y_2 - y_1)^2 + (z_2 - z_1)^2} \right)$ is the natural logarithm of distance between locations coming from mathematical solution proposed by E.M. Sparrow\textsuperscript{17}. The use of analytical solutions and of hemicube method were also investigated to reduce the computing time.

Figure C.3.5 shows different steps of the simulation for simplified road scene and LWIR camera.

![Figure C.3.5: Road scene generation (left), net radiation map (middle), Incidence map (right)](image)

Such tool, even though, is not completely achieved (lack of 3D objects insertion in the final radiation heat balance), is enough developed at that stage to evaluate potential solutions by acting on thermo-optical properties at pavement surface level. It allows us to reduce on road trials.

C.3.5 Trials

C.3.5.1 On road trials

The outside test site chosen to make first experimentation is the small regional airport of Autun. It has the advantage of having a large width test track with a straight long part (runaway) and experimentations at night are facilitated by absence of flight. The second interesting aspect is that no vertical vegetation is present on the shoulder of the track which is close to the conditions we have implemented in the simulation tool developed. Figure C.3.6 show images of this test site in the visible and in the infrared.

![View of the airport test track](image) ![infrared image without object](image)

**Figure C.3.6:** View of the test site in the infrared and the visible spectrum

The infrared images acquired during all these trials were made using an infrared camera in Band III (FLIR S65). Furthermore, pavement surface temperature and atmosphere temperature and relative humidity were also measured. Different infrared and visible images were acquired for different
distances between various targets (pedestrian, vehicle, pedestrian + vehicle) and infrared camera, but also for different locations of targets in the width of the pavement and with or without adding vehicle headlights. For instance Figure C.3.7 show images acquired (visible and infrared spectrum) for a distance between camera and pedestrian of 50 m.

<table>
<thead>
<tr>
<th>Distance 50m – Pedestrian at right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Infrared – no headlight</td>
</tr>
<tr>
<td>Infrared – dipped headlight</td>
</tr>
<tr>
<td>Infrared - full headlight</td>
</tr>
<tr>
<td>Visible – no headlight</td>
</tr>
<tr>
<td>Visible – dipped headlight</td>
</tr>
<tr>
<td>Visible - full headlight</td>
</tr>
</tbody>
</table>

**Figure C.3.7:** View of the test site in the infrared and the visible spectrum

Images acquired on this test site were also used to validate part of the infrared vision simulation tool developed. Figure C.3.8 show an infrared image acquired on this test site, its associated road scene construction and the resulting incidence map on the detector (in logarithmic scale) computed using data measured, at local thermal equilibrium, during trials.

**Figure C.3.8:** View of the test site in the infrared and the visible spectrum

Figure C.3.9 shows for this same road scene incidence map computed in one case by acting on the emissivity of the roadside in orange on left image (set to 0.2) and in the other case by acting on its temperature (set to 10°C over the surface temperature of the carriageway).

**Figure C.3.9:** Simulation with Roadside modified properties – IR spectral bandwidth III : 7.5-13 μm
Except for the sky where, we do not introduce the radiative effect of the surrounding mountains at the horizon, a good agreement in the general distribution of the signal issued from the element of the scene simulated is observed by comparison with the experiment. Simulation tool is then used to evaluate signal that can be available by acting on thermo-optical properties of the pavement surface, for instance by changing its emissivity or its temperature.

C.3.5.2 Trials in fog tunnel

The fog tunnel\(^{18}\) (Figure C.3.10 (left)) consisted of a facility 30 m long, 5.5 m wide, and 2 m high, producing artificial fog in monitored conditions (see Figure C.3.10 (right)). This equipment allowed to study the factors that affect visibility in fog, both during day and night time. An observation station consisting of the front of a car with headlights allows for both photometric measurements and personal observation by individuals seated in the driving position.

Figure C.3.10: Fog tunnel facility scheme (left), Infrared images: MWIR (middle) and LWIR (right)

During experiments water droplet size distribution was measured. Two transmissionmeters, located at 15 and 30 m, were also used to determine the evolution of the meteorological visibility during trials.

Figure C.3.11 present the scene studied in the fog tunnel under night vision conditions. Infrared cameras were placed directly in the fog tunnel which is more closed to road application even though we acquired images in static mode. During trials, the heated panel was located at 27 m for MWIR II, 28 m for LWIR and 29 m for photopic imagery (visible spectrum). The base of the panel was located at 94 cm from the ground level and the upper part of the legs of the little worker drawing at 120 cm. The panel is an equilateral triangle of length 1 m. It is recovered by a 3M reflective wearing material (micro marble type) and the little worker drawing is also compound of a black 3M non reflective material glued on its surface.

Figure C.3.11: Road work panel heated by an halogen lamp (left), Road scene viewed from the control room (middle) and Infrared cameras implantation in the fog tunnel (right)

To illustrate image enhancement using infrared vision, some images reported in Figure C.3.12 for different meteorological visibility show what can be seen in the visible spectrum only using car headlight in foggy night conditions. The circular panel located at 15 m is non active and the road works panel is still at 29 m but not visible on the images. Level of illumination measured on the panel are very low, less than 5 cd/m². That’s why for low visibility the integration for measurement can reach
10 s due to low signal recorded by the calibrated detector (Photopic camera). For on road application typical integration time required is closed to one millisecond.

**Figure C.3.12:** Images acquired in the visible spectrum with the Videophotometer with car Headlights and for different meteorological visibilities: left \( V = 20 \text{ m} \) – middle \( V = 15 \text{ m} \) – right \( V = 12 \text{ m} \)

Such images in the visible spectrum have to be compared with infrared ones presented hereafter on Figure C.3.13 to Figure C.3.15 for meteorological visibilities of same range, with and without a thermal excitation on the road works panel.

**Figure C.3.13:** Infrared images: MWIR in grey levels (non active panel on left and active on right) LWIR in pseudo colour levels (non active panel on left and active on right) – \( V = 22 \text{ m} \)

**Figure C.3.14:** Infrared images: MWIR in grey levels (non active panel on left and active on right) LWIR in pseudo colour levels (non active panel on left and active on right) – \( V = 16 \text{ m} \)

**Figure C.3.15:** Infrared images: MWIR in grey levels (only for active panel) LWIR in pseudo colour levels (non active panel on left and active on right) – \( V = 12 \text{ m} \)

Globally, we observed an enhancement of the visibility from photopic vision by using infrared vision on active thermal panels placed in foggy atmosphere. On the other hand, the panel placed at 13 m and which is not thermally active drive to infrared images where it is less or more included in the background signal. Furthermore, the reflective surface put on this surface (square piece of aluminium) didn’t induce a particular contrast in presence of fog. As previously seen for on road trials, no influence of the artificial lighting in fog tunnel or from the car headlights was observed. In infrared bandwidth used, lighting spots of the tunnel and rectangular target, when they are under electrical alimentation, are just particular intensive radiative area in infrared image. The wearing of the road works panel and particularly its information is still readable in infrared bandwidth II, more easily when thermal excitation
is applied. For infrared band III information is lost but hot panel remains visible for lower meteorological visibility distance, to preserve information the thermal excitation must be done with appropriated coding distribution. Such trials can also take benefit of using optical configurations adapted to each detector to have a same field of view to facilitate comparison between each spectral bandwidth.

Figure C.3.16 presents the evolution of the active panel or target signal transmission versus the evolution of the meteorological visibility. One will observe that for thin water droplet size 5% of the signal is transmitted for a visibility of 12 m in LWIR spectral band and at 20 m in MWIR. For the other water droplet size such value fall down to 22 m in LWIR spectral band.

Figure C.3.16: Infrared signal transmission evolution versus meteorological visibility measured for different trials (two fogs, targets and infrared bandwidth)

So, efficiency of infrared vision in foggy night conditions also depends on the type of fog, but thermal active panel favour its eye detection on the scene.

C.3.6 Conclusions and recommendations

Results obtained with the emissivity measurement apparatus has been used to make numerical simulations with the infrared simulation tool developed for simplified road geometries. Nevertheless, the spectral bandwidth used for measurement is still too large with regards to the more restricted spectral bandwidth of infrared vision systems on the market. Some accurate measurements could be undertaken with an experimental setup with a proper sensor in the 7-14 µm spectral bandwidth, and in various directions. Based on the Mie theory, calculations of absorption and scattering coefficients were developed from experimental water droplet size distribution data acquired in a fog tunnel. Monte Carlo simulation, valid when absorption is negligible (i.e., in visible range), allowed to see the influence of each fog type on road sign perception: decrease of the contrast with decrease in meteorological visibility was observed, but also increase of backscattering with decrease of water droplet size in foggy night conditions. The infrared simulation tools developed, even though, it is not completely achieved at that stage (lack of 3D objects insertion in the final radiation heat balance), allows to evaluate potential solutions by acting on thermo-optical properties at pavement surface level and permits to reduce on road trials. For instance, simulations permit to evaluate the size of cooperative elements of infrastructure required to be perceptible on infrared images by taking account the characteristics of on-board infrared vision system used. Experiments on road site and in fog tunnel allowed us to validate
numerical simulations. They also have permitted to verify that improving contrast on infrared images by generating a thermal excitation on infrastructure typical elements had to be favour in front of reducing their emissivity in foggy night conditions. Recovering energy from road could be a sustainable solution to generate active thermal elements for on board infrared vision system.
C.4  Innovation 2.3: New pavement maintenance technique aiming at enlarging the overall conditions of application

C.4.1 Organisation and tasks

The first step in this research is to identify the climatic parameters which have an incidence on the pavement maintenance works, on the quality of the mixtures that are placed and finally on the behaviour of pavement. Some of these climatic parameters lead to a reduction of the mechanical properties of the material and consequently on the pavement performance. So it will be necessary to identify the linkage between the climatic factors and the mechanical properties of the layers after placing. In terms of product it will be necessary to distinguish the components (bitumen, hydraulic binder…) and the final product (layer).

The main goal is to formulate some “realistic” proposals considering the requirements and the specifications of the innovation and the foreseen advantages and to produce a guide providing information aiming at reducing the effects of the weather conditions on the maintenance works, and consequently the risk of having poor pavement mechanical properties leading to a poor behaviour of the road structure.

C.4.2 Rating trees

C.4.2.1 Climatic factors and materials

In order to implement this rating tree it has been necessary to identify the climatic parameters, which do not allow the continuation of the job sites. For this task we have used the specifications in the contracts.

These parameters are the following:

- Wind  Cold  Rain  Snow  Storm  Night

It has been necessary to add Heat and Heat and Loading. These factors are not yet considered as negative parameters on job sites, but the high temperatures in 2006 summer oblige us to integrate them.

Note: During the bibliography research it has not been easy to find some information about the weather conditions (or weather restrictions) in official guidelines for the road industry.

After this identification, due to different processes of manufacturing it was necessary to distinguish bituminous (it is necessary to heat and dry the aggregates in order to get the same temperature as thus of the bitumen which is added) and hydraulic (aggregates are used at ambient temperature and water is added for the setting of the binder and sometimes in order to improve the workability during the mix phases) bounded materials. The manufacturing or these materials are different.

C.4.2.2 Influence of climatic parameters

The most original part of this work is the identification of the relationship between the climatic parameters and the properties of the materials.

The properties of the materials can be shared in two classes:

Mechanical properties

- They are linked with the behaviour of the materials and the life duration of the pavement. The most well-known criteria are the stiffness (characterized by the elastic modulus) and the fatigue behaviour (characterized by the allowed strain at $10^6$ cycles).
Safety properties

- One of the two most important criteria are the increasing voids content due to a lack of workability of the mix and the presence of cracks (longitudinal or transversal).

- They are linked with surface characteristics of the road. We find evenness, lack of skid resistance, risk of rutting.
- And the risk of accident when there are fumes due to the contact of moisture (or rain) with hof asphalt mixes.

C.4.3 Rating tree for bituminous material

During this phases different rating trees have been elaborated taking into account the different climatic parameters mentioned in C.4.2.1

For example only three of them: cold, rain and heat are described in this summary because they are the most important and frequent on the job site

C.4.3.1 Cold

![Rating tree for bituminous material](image)
C.4.3.2 Rain

- Decreasing of temperature of air and bituminous materials
- Presence of water in the materials
- Uncompressible material in the asphalt mix
- Difficulty of compaction
- Decreasing of voids content
- Decreasing of evenness
- Decreasing of life duration

Decreasing of temperature of air and bituminous materials leads to:
- Increasing of bitumen viscosity
- Loss of workability of the bituminous materials
- Increasing of voids content
- Decreasing of evenness
- Decreasing of life duration

Decreasing of voids content results in:
- Decreasing of modulus $E$
- Decreasing of allowable strains $\varepsilon_6$

Decreasing of evenness causes:
- Decreasing of life duration

Decreasing of life duration leads to:
- Uncomfort
- Risk for safety

C.4.3.3 Heat before loading

- Works on the upper layer before allowable temperature of the lower layer
- Traffic on the road before allowable temperature of the pavement

Works on the upper layer before allowable temperature of the lower layer results in:
- Risk of over-compaction of the lower layer
- Decreasing of voids content
- Risk of rutting
- Decreasing of life duration

Risk of rutting leads to:
- Uncomfort
- Risk for safety

Traffic on the road before allowable temperature of the pavement results in:
- Risk of over-compaction of the upper layer (wearing course)
- Excess of bitumen at the top of the wearing course
- Risk of bleeding

Excess of bitumen at the top of the wearing course causes:
- Lack of skid resistance
- Risk of safety
C.4.4 Rating tree for hydraulic bounded materials

For this type of materials it was easy to summarize the influence of climatic parameters on a single rating tree.

C.4.4.1 Cold, rain and heat

C.4.5 Solution for bituminous material

C.4.5.1 Cold

When the outside temperature is lower than allowed in the specification (or known by the state of the art) the bitumen viscosity increases. To solve this problem one way is to maintain the workability of asphalt mix by adding wax or zeolite.
An other consequence of the cold is the lack of binding between two widths of mix. A solution can be found with a preformed adhesive strip used on a job site in 2005.

C.4.5.2 Rain
When it is raining, the rain cools the asphalt mix (see above) and there is a production of foam or vapor. This is due to the large range of temperature between the asphalt mix (160 to 170°C) and thus of the rain (10 to 15°C). To solve this problem one way is to reduce the temperature of the asphalt mix by adding wax or zeolite.

C.4.5.3 Loading before cooling
When asphalt mix is always hot, its mechanical performances are low. Its modulus (depending on temperature) is decreasing.

The time necessary to reach the optimal temperature (60°C) may be long. By reducing the asphalt temperature during the manufacturing, it is possible to gain faster the optimal characteristics and avoid early rutting.

Reference documents are documents which fall in the following two categories:
either they are explicitly mentioned in the text of the Project Quality Plan;
or they do not contain binding requirements.

C.4.6 Conclusions

C.4.6.1 Bituminous bounding materials
The solutions provided for enlarging the overall conditions of application have been founded only for the following climatic parameters:

- Cold
- Rain
- Wind
- Heat and loading

Some consequences of climatic parameters such as water on support, film of water on tack-coat have not been solved with original solutions.

In the case of storm or snow the best way is to wait... better weather conditions.
C.4.6.2 Hydraulic bounding materials
No original solutions have been provided for enlarging the overall conditions of application due to the fact that there is no means to smooth influence of cold, rain or wind.

C.4.6.3 Trends
The opportunity of enlarging the overall condition of pavement materials application is not an authorization for working when the risk of failure or damage is too high. It is only a controlled opportunity.
C.5 Innovation 2.4: Improving the mechanical properties of a low noise section

SOFTBLOC – Silent poroelastic overlay fixed on tailored cement block pavement

C.5.1 Justification and background to the innovation

The innovation consists of a poroelastic material fixed on paving blocks. A poroelastic road material, in the forms we currently know, consists of an aggregate of rubber granules or fibres, sometimes supplemented by sand, stones or other friction-enhancing additives. The rubber can be either from scrap tyres or “new” rubber. It further consists of a binder to hold the mix together. Currently, the binder which was tried is polyurethane, with a binder contents tried so far ranging between 5 and 17% by weight. In combination with paving blocks this surface combines the excellent noise reducing qualities of the poroelastic material with the possibility to manufacture a large part of the road off site and thus avoiding problems with poor adhesion, which was a major obstacle in previous field tests with poroelastic road materials.

Previous experiences with poroelastic road material were summarized in a report, SILVIA Project Report SILVIA-VTI-005-02-WP4-141005. The report can be downloaded from the website: http://www.trl.co.uk/silvia/. To summarize: Early on it was proven that it is not too difficult to produce a poroelastic surface with excellent internal durability. Later test has also confirmed these results and have also demonstrated that the durability towards for example studded tyres is much better for poroelastic road surface (PERS) than conventional pavement materials. On the contrary, the problems encountered in some of the early trials with poor adhesion between PERS and the base layer were also the cause of failure in some later field trials. Laboratory trials on adhesion and some of the later field trials have shown that the adhesion could be made sufficiently strong, if enough binder is applied to the surface and proper attention is taken to cleaning of the base layer prior to adding the binder. The risk of having poor adhesion could probably be completely avoided if the binding is done in a controlled environment. Wet friction was and still is a major obstacle for poroelastic road material. The initial wet friction of the recently produced PERS was more than sufficient. However, the excellent wear resistance of rubber has the adverse effect, that the friction enhancing materials added to the mixes were worn away faster from the surface than the rubber.

C.5.2 Identification of key points to improve

In many of the previous experiments the lack of adhesion between poroelastic and underlaying layers was the reason for failures. This is also the reason for choosing a pavement structure where poroelastic layer is layed on concrete (paving) blocks. In this way the critical gluing can be made in more controlled and proper manner and environment.

Taking into account problems and reasons for failure of test fields in Japan, we have identified and selected some key point topics for improvement. The foundation for the blocks (bedding layer) has to be improved, intending to avoid the Japanese failure.

The previous experiments are showing that a friction characteristic of the poroelastic surface still needs to be improved. It is expected that this can be done by adding new friction-enhancing additives to the poroelastic mixture. The friction enhancing material that will be tested will be very hard and wear resistant material. The influence of the hardness of the poroelastic material on the friction properties will also be tested. Recently there have been indications that comparatively hard poroelastic material has better durability of the wet frictional properties compared to soft versions of the material.

The research in this innovation was focused on solving two major problems with this type of surface:

- The wet skid resistance must be maintained at an acceptable level for a reasonable operating time for a typical road condition
- The stability of the system of blocks and stabilizing (bedding) layer must be sufficient for a reasonable operating time and for a mix of light and heavy vehicles
C.5.3 Wet skid resistance

A laboratory test scheme was set up to study means for having an interlocking block surface covered with a poroelastic material that (together with the blocks) is both resistant to rutting and has a durable wet friction. For improving the frictional properties, a test will be conducted to study the influence on the durability of friction of binder hardness, addition of silicon carbide to the mix and pre-treatment of the rubber crumbs.

For simulation of the polishing effect of rolling tyres on a road surface, the VTI pavement testing machine will be used. The pavement testing machine is a circular track with a diameter of approximately 5 metres. Four or six wheels are rolling on this surface at a speed up to 70 km/h (see Fig. C.5.1). The power to the traction of the wheels is delivered by electrical engines attached on each of the wheel axles. The load on each wheel is adjustable but is usually fixed at 450 kg. The wheels do not follow a circular track; rather there is a lateral movement of the wheels to simulate a realistic distribution of wheel paths. The test can be made in dry or wet conditions. When wet conditions are used, fresh water is constantly sprinkled over the surface. The evolution of the friction will be tested with the British Pendulum method and by manually pushed equipment developed at VTI which measures the friction on a partially slipping wheel pushed at a speed of approximately 5 km/h.

![Figure C.5.1: The Pavement Testing Machine at VTI.](image-url)
C.5.4 Raw material

Rubber granules.
The maximum density of the rubber granules was determined with the method EN 12697-5. The density was 1.141 g/cm³.
Since the grading curve of the rubber granules proved to be very narrow a combination of a large number sieves with nominal sizes around 1-5 mm was used. The grading curve of the rubber granules is presented in figure C.5.2. The sizes of the granules ranged from 1.4 mm to 4.0 mm.

![Grading curve of the rubber granules (PV).](image)

**Figure C.5.2:** Grading curve of the rubber granules (PV).

Hardened rubber granules
The hardness of tire rubber is usually about 50-60 Shore A. The hardness of the rubber could to some extent be adjusted to higher values if the rubber is artificially aged. In order to check the influence of rubber hardness towards the durability to friction, a part of the rubber granules were artificially aged in a forcefully ventilated oven at 90°C for 12 hours. The granules were kept in 5 L buckets without lids. The granules were noticeable harder but the hardness could not be checked with a shore meter as the granules are too small to make any measurement with a conventional Shore meter meaningful.

Polyurethane
Two types of polyurethane binders were tested.
The first one, Flexilon 1109, is a prepolymerized MDI polyurethane produced by Rosehill Ltd. UK. The density of this binder is 1.10 g/cm³ and the hardness of the cured product matches the hardness of rubber, e.g. approximately 60 Shore A. This binder is referred to as the soft binder in this report.
The other product tested, was specially produced for our tests. The Swedish representative for Lagomat/Elastogran/BASF. The product was a two component polyurethane binder with a design hardness of the cured product of approximately 90 Shore A. This specially formulated product didn’t have any name and is referred to as the hard binder, in this paper.
Catalyst
A substituted morpholine compound was used as catalyst to decrease the curing time.

Friction enhancing material
Silicon carbide, or carborundum, was used to improve the wet friction of the poroelastic material. The material was delivered from Saint Gobain Abrasives. The product used was SIKA ABR IV F150 i.e. the carborundum granules passed a 0.104 mm sieve. The density of the material is 3.22 g/cm³.

C.5.5 Production of test material

The production of the test materials were done mixing all compounds in a single mixing pot followed by curing in a mould with a fixed volume. Having précis control of the amount of the mixed material and their densities enabled us to tune the air void content to the design value of 21%. The design value for the air void content was chosen to be rather low to what is possible, but since the purpose of the experiment polishing and the durability of wet friction, a high air void content could give less wear resistance and thus jeopardize the aim of the experiment.

Figure C.5.3: Production of test material.
Figure C.5.4: Production of test material.

Figure C.5.5: Production of test material. Releasing the finished material from the mould.
Figure C.5.6: Production of test material. Mounting the material in the PVM.

The following mixes were produced:

<table>
<thead>
<tr>
<th>Number &amp; Name</th>
<th>Rubber</th>
<th>Binder</th>
<th>Rubber content %(w/w)</th>
<th>Binder content %(w/w)</th>
<th>Silicon carbide content %(w/w)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 Hard, reference</td>
<td>Un-aged</td>
<td>Hard, two component</td>
<td>79</td>
<td>21</td>
<td>-</td>
</tr>
<tr>
<td>4 Hard, abrasive</td>
<td>Un-aged</td>
<td>Hard, two component</td>
<td>65</td>
<td>16</td>
<td>19</td>
</tr>
<tr>
<td>5 Hard abrasive and aged</td>
<td>Aged</td>
<td>Hard, two component</td>
<td>65</td>
<td>16</td>
<td>19</td>
</tr>
<tr>
<td>6 Soft, abrasive</td>
<td>Un-aged</td>
<td>Soft, one component</td>
<td>65</td>
<td>16</td>
<td>19</td>
</tr>
<tr>
<td>1 Rosehill abrasive</td>
<td>Not available</td>
<td>Not available</td>
<td>Not available</td>
<td>Not available</td>
<td>Not available</td>
</tr>
</tbody>
</table>

Two plates, A and B, were produced of each mix type and mounted in the PVM for polishing and wear tests.

C.5.6 Test description

Pavement testing machine PVM

The VTI Pavement testing machine (PVM) is a circular test track with a track diameter of 5.25 m. The machine can be used to do accelerated tests of tire road interactions. The machine has been used for accelerated road wear studies, accelerated traffic polishing studies and studies of emissions of particulate matters form tire road interaction etc. In this study the intention is to focus on traffic polishing, but wear will also be an issue. The wheels in the PVM are driven by electrical engines.
Beside the circular movement of the wheels, there are also lateral movements of the wheel, to make the wheel tracks more realistic.

During the test water was sprinkled over the surface with a rate of 6-8 L/minute, see figure C.5.7.

Figure C.5.7: The surfaces were sprinkled with water @ 6-8 L/minute during the accelerated traffic polishing tests.

The air temperature in the test hall was kept at 10-12°C during the tests. The PVM was fitted with four wheels with the dimensions 185/65 R15. The tires were inflated to a pressure of 2.5 bar. Each tire was pushed against the road surface with a load equal to 450 kg. Two types of tires were used. For the first 102 000 revolutions Nokian Nrhi tires was used. These tires are of normal summer type. After the first 102 000 revolutions the tires were shifted to Nokian Hakkapelitta Q tires and the PVM run for another 50 700 revolutions. These tires are used in winter time in the Nordic countries but they are not fitted with studs, rather the grip is accomplished with softer rubber composition in the tread and more sipes in the tread pattern. Such tires are commonly referred to as “friction tires”. The purpose of shifting the tires was to see if the tire type had a decisive role for the traffic polishing effect.

The speed was kept at 60 km/h for the first 23 000 revolutions but was then increased to 70 km/h. Higher speed increase the traffic polishing effect and the wear of the tires per revolution.

Friction tests
Friction coefficients was measured with a device developed at VTI called the Portable Friction Tester, PFT. The friction was measured on wet surfaces. The PFT is a small device, weighing 38 kg, which is pushed by the operator, see figure C.5.8, and measures the friction coefficient for a wheel at constant slip, i.e. the measuring wheel is rotating at a lower speed relative the road surface than the device traction wheels. The measuring wheel has a slip of 21%, i.e. the measuring wheel is rotating at 21% of the speed of the device traction wheels. The friction coefficient is measured at normal walking speed.
In Sweden, the wet friction coefficient should be above 0.5, although the device for measuring the normative friction coefficient is not the same as the PFT, rather a full size car equipped with a fifth measuring wheel. Friction was measured on the poroelastic material in the rolling direction of the PVM.

![Figure C.5.8](image-url)

**Figure C.5.8:** The Portable Friction Tester used for measuring the wet friction coefficient at walking speed on the poroelastic surfaces. In the figure the PFT is placed with the measuring wheel on a road marking.

### C.6 Results

The accelerated test was performed for in total 152 700 revolutions, i.e. the surfaces was run over by a tire for 610 800 times. The first 23 000 revolutions were done at 60 km/h and the rest at 70 km/h. Initially and at seven intervals the machine was stopped and the friction coefficient was measured in the rolling direction of the wheels, with the PFT. Already at the first stop at 23 000 revolutions both plates of one surface and one plate of another surface was broken down. There were tendencies on some of the other test plates that some of the material wasn’t durable enough. At the second stop at 45 000 revolutions there was in total 2 sections that was broken down and one plate of another section that was broken, see figure C.5.9 for an example of a broken test plate. From there and onwards the remaining plates were intact except for some wearing. All broken plates had to be replaced with dummy plates not to prevent the wheels to jump up and down when they passed the broken part of the track. At the end of the experiment the rut depth was measured with a straight edge. The rut depths are presented in table C.5.1.
Table C.5.1 Rut depth at the end of the accelerated testing in the PVM.

<table>
<thead>
<tr>
<th>Number &amp; Name</th>
<th>Rut depth plate A (mm)</th>
<th>Rut depth plate B (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 Hard, reference</td>
<td>broken</td>
<td>broken</td>
</tr>
<tr>
<td>4 Hard, abrasive</td>
<td>broken</td>
<td>broken</td>
</tr>
<tr>
<td>5 Hard abrasive and aged</td>
<td>broken</td>
<td>4.1</td>
</tr>
<tr>
<td>6 Soft, abrasive</td>
<td>0.5</td>
<td>1.2</td>
</tr>
<tr>
<td>1 Rosehill abrasive</td>
<td>2.3</td>
<td>1.1</td>
</tr>
</tbody>
</table>

Figure C.5.9 Broken plate at after 23 000 revolutions in the PVM.

The friction coefficients are presented in table C.5.2.
Table C.5.2: Evolution of wet friction coefficient in the accelerated polishing test. At 102 000 revolutions the tires were shifted from regular summer tires to friction tires.

The friction coefficients presented in table C.5.2 for the material with silicon carbide additions are very high. It is also obvious from the table that the silicon carbide addition did give the poroelastic material a lasting wet friction and the polishing action of the tires was weak for silicon carbide.

Some of the material had a very poor wear resistance and that the artificial ageing of the rubber only made poroelastic material perform worse. The best performing materials were the material produced by Rosehill with addition of abrasive material and the material produced with a soft one component binder with silicon carbide as friction enhancing material.
C.6.1 Stability of block pavement

Pavement structure and materials characteristics

Four different setups were tested. First, tests were performed on a structure where cement concrete blocks with a glued poroelastic overlay were placed onto a sand bedding layer. The second test setup was the same only that it included watering of the pavement structure. The third and fourth setups were parallel cases, except that the cement concrete blocks were placed onto a cementitious screed bedding layer. The pavement structures were prepared in a special wooden mould.

In total there were four test cycles. One cycle consisted of a piston-induced dynamic loading of the test structure, as can be seen in Figure C.5.10. There were 100,000 vertical loadings applied through a heavy vehicle tire, each time of maximum load of 35 kN. The loading equals 100,000 passes of 140 kN axle load. The vertical displacement of the structure was monitored and followed by three LVDTs, mounted on a framework and placed along the tire.

![Test assembly with a sketch showing the testing principle](image)

**Figure C.5.10:** Test assembly with a sketch showing the testing principle

The pavement structure can be seen from Figure C.5.11. Poroelastic material was cut in the shape of the cement concrete blocks and glued to them. Blocks were placed into a bedding layer (sand or cementitious screed, 5 – 6 cm), that was laid onto two asphalt layers (6 + 3 cm). At the bottom of the structure there was laid the unbound material layer (30 cm). When the concrete blocks with poroelastic cover were laid into the bedding layer, dry siliceous sand 0/2 mm was used to fill the spacing between each block and spacing between block assembly and the wooden mould.

![Structure of the tested pavement](image)

**Figure C.5.11:** Structure of the tested pavement

Poroelastic cover on cement concrete blocks

The poroelastic material that was used is named “Tokai” and is in more detail described in the EU-project SILVIA report no. SILVIA-VTI-005-02-WP4-141005 [3]. Originally it was prefabricated in rubber panels 1×1m² and imported from Japan. The same material was used for testing at the test site in a residential area in Stockholm. The panel thickness is of 30 mm and it is made of the rubber fibres. A polyurethane type binder was used as a binder for the mix. On the underside the panels have a square mesh (200×200 mm) of drainage channels. From panels there were pieces of the poroelastic material cut in the wavy shape of the standard commercial cement concrete blocks (see Figure C.5.12).
The critical gluing was done in a controlled and proper manner and environment – in a laboratory. The poroelastic material was glued to concrete blocks using a mixture of the two epoxy based adhesives. The mechanical properties - tensile strength and shear strength - of the new adhesive were tested before gluing the pieces together. For determining the shear strength, the joint between a piece of poroelastic material and concrete block was loaded in longitudinal direction. For determining the tensile strength, the samples were torn apart using the steel caps that were additionally glued to pieces of poroelastic material.

The test samples were cured for 72 hours in laboratory conditions (23°C, 50 % r.h.). A testing machine ZWICK Z100 was used for loading specimens, with a rate of loading of 10 mm/min. The determined shear and tensile strengths of 12 multiple specimens are shown in Table C.5.3.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Joint shear strength (MPa)</th>
<th>Break point deformation (mm)</th>
<th>Joint tensile strength (MPa)</th>
<th>Break point deformation (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.45</td>
<td>6.22</td>
<td>0.63</td>
<td>4.53</td>
</tr>
<tr>
<td>2</td>
<td>0.53</td>
<td>6.30</td>
<td>0.69</td>
<td>5.47</td>
</tr>
<tr>
<td>3</td>
<td>0.47</td>
<td>7.42</td>
<td>0.75</td>
<td>6.60</td>
</tr>
<tr>
<td>4</td>
<td>0.40</td>
<td>5.22</td>
<td>0.60</td>
<td>5.57</td>
</tr>
<tr>
<td>5</td>
<td>0.43</td>
<td>5.68</td>
<td>0.70</td>
<td>7.04</td>
</tr>
<tr>
<td>6</td>
<td>0.46</td>
<td>4.49</td>
<td>0.75</td>
<td>6.27</td>
</tr>
<tr>
<td>7</td>
<td>0.61</td>
<td>4.43</td>
<td>0.59</td>
<td>4.27</td>
</tr>
<tr>
<td>8</td>
<td>0.49</td>
<td>6.22</td>
<td>0.58</td>
<td>4.32</td>
</tr>
<tr>
<td>9</td>
<td>0.43</td>
<td>5.77</td>
<td>0.60</td>
<td>4.98</td>
</tr>
<tr>
<td>10</td>
<td>0.59</td>
<td>5.72</td>
<td>0.53</td>
<td>4.56</td>
</tr>
<tr>
<td>11</td>
<td>0.62</td>
<td>4.64</td>
<td>0.54</td>
<td>5.20</td>
</tr>
<tr>
<td>12</td>
<td>0.61</td>
<td>6.36</td>
<td>0.63</td>
<td>5.00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>average</th>
<th>st. deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint shear</td>
<td>0.51</td>
<td>0.08</td>
</tr>
<tr>
<td>strength</td>
<td></td>
<td>0.07</td>
</tr>
<tr>
<td>MPa</td>
<td></td>
<td>0.91</td>
</tr>
<tr>
<td>Break point</td>
<td>5.72</td>
<td>0.63</td>
</tr>
<tr>
<td>deformation</td>
<td></td>
<td>5.32</td>
</tr>
<tr>
<td>(mm)</td>
<td></td>
<td>0.91</td>
</tr>
</tbody>
</table>

**Table C.5.3: The results of the shear and tensile strength tests**

At both tests and with all the specimens, the weakest point was the poroelastic material. In all cases the breakdown happened in the poroelastic piece, whereas the joints between poroelastic material and concrete block remained in good conditions. The failure was in poroelastic material and not in the adhesive.

**Bedding layer**

As a bedding layer there were two options chosen. First option was to lay the concrete blocks in the sand 0/4 mm layer, the second one was to lay them in the cementitious screed layer.

The grading of the sand aggregate that was used for the bedding layer can be seen from Table C.5.4. The moisture content of five samples varied between 6.73 % and 7.72 %.

<table>
<thead>
<tr>
<th>% passing</th>
<th>8</th>
<th>4</th>
<th>2</th>
<th>1</th>
<th>0.71</th>
<th>0.50</th>
<th>0.25</th>
<th>0.125</th>
<th>0.090</th>
<th>0.063</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand 0/4</td>
<td>100</td>
<td>99</td>
<td>76</td>
<td>45</td>
<td>33</td>
<td>29</td>
<td>18</td>
<td>12</td>
<td>10</td>
<td>9.3</td>
</tr>
</tbody>
</table>

**Table C.5.4: Grading of the sand bedding layer**
For the cementitious screed bedding layer we have chosen a PflesterDrainmörtel GK 4 (porous mortar) product from Baumit company. The product was prepared and laid into the pavement according to the manufacturer’s directions. The moisture content of five samples varied between 4.36 % and 5.95 %.

**Asphalt layers**

There were two layers placed into the pavement structure: AC 8 of 3 cm layer over bituminous well graded crushed stone BD 22s layer (asphalt concrete base layer; 6 cm). The characteristics of the binder that was used in different mixes can be seen from Table C.5.5.

<table>
<thead>
<tr>
<th>Binder type</th>
<th>bitumen B50/70</th>
<th>bitumen B70/100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer</td>
<td>Bituminous crushed stone BD 22s</td>
<td>Asphalt concrete BB 8</td>
</tr>
<tr>
<td>Ball and ring test (°C)</td>
<td>51.2</td>
<td>46.2</td>
</tr>
<tr>
<td>Penetration (mm/10)</td>
<td>53.0</td>
<td>86.0</td>
</tr>
<tr>
<td>Index of penetration</td>
<td>-0.8</td>
<td>-0.9</td>
</tr>
</tbody>
</table>

**Table C.5.5:** Characteristics of binder in the asphalt mixes

Following tables show the soluble binder content, particle size distribution of mineral aggregates and other measured characteristics of both mixes.

<table>
<thead>
<tr>
<th>Pavement</th>
<th>Binder content</th>
<th>Passing / Sieve (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.09</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>% (m/m)</td>
<td>% (m/m)</td>
</tr>
<tr>
<td>1 (sand bed layer)</td>
<td>5.1</td>
<td>12.0</td>
</tr>
<tr>
<td>2 (screed bed layer)</td>
<td>5.2</td>
<td>12.4</td>
</tr>
</tbody>
</table>

**Table C.5.6:** Particle size distribution and binder content in BB 8 mix

<table>
<thead>
<tr>
<th>Pavement</th>
<th>Binder content</th>
<th>Passing / Sieve (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.09</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>% (m/m)</td>
<td>% (m/m)</td>
</tr>
<tr>
<td>1 (sand bed layer)</td>
<td>3.3</td>
<td>6.1</td>
</tr>
<tr>
<td>2 (screed bed layer)</td>
<td>3.5</td>
<td>6.9</td>
</tr>
</tbody>
</table>

**Table C.5.7:** Characteristics of BB 8 mix

<table>
<thead>
<tr>
<th>Pavement</th>
<th>Binder content</th>
<th>Passing / Sieve (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.09</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>% (m/m)</td>
<td>% (m/m)</td>
</tr>
<tr>
<td>1 (sand bed layer)</td>
<td>3.5</td>
<td>6.9</td>
</tr>
<tr>
<td>2 (screed bed layer)</td>
<td>3.5</td>
<td>6.9</td>
</tr>
</tbody>
</table>

**Table C.5.8:** Particle size distribution and binder content in BD 22s mix

<table>
<thead>
<tr>
<th>Pavement</th>
<th>Binder content</th>
<th>Passing / Sieve (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.09</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>% (m/m)</td>
<td>% (m/m)</td>
</tr>
<tr>
<td>1 (sand bed layer)</td>
<td>3.5</td>
<td>6.9</td>
</tr>
<tr>
<td>2 (screed bed layer)</td>
<td>3.5</td>
<td>6.9</td>
</tr>
</tbody>
</table>

**Unbound layer**

For the unbound layer the crushed stone aggregate 0/32 mm was used. The grading of mineral aggregate and other characteristics can be seen from tables C.5.10 and C.5.11. The fines content of the aggregate is 4 %.
<table>
<thead>
<tr>
<th>% passing</th>
<th>63</th>
<th>45</th>
<th>31.5</th>
<th>22.4</th>
<th>16</th>
<th>11.2</th>
<th>8</th>
<th>4</th>
<th>2</th>
<th>0.71</th>
<th>0.25</th>
<th>0.090</th>
<th>0.063</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sieve (mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand 0/4</td>
<td>100</td>
<td>94</td>
<td>92</td>
<td>78</td>
<td>65</td>
<td>53</td>
<td>43</td>
<td>29</td>
<td>19</td>
<td>10</td>
<td>6</td>
<td>4</td>
<td>4.0</td>
</tr>
</tbody>
</table>

Table C.5.10: Particle size distribution of the mineral aggregate for unbound layer

<table>
<thead>
<tr>
<th>Geometrical properties</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flakiness index</td>
<td>17</td>
</tr>
<tr>
<td>Shape index</td>
<td>18</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Fines content</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Assessment of fines</td>
<td></td>
</tr>
<tr>
<td>Sand equivalent test</td>
<td>45</td>
</tr>
<tr>
<td>Methylene blue test</td>
<td>1.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Resistance to fragmentation</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Los Angeles coef)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Resistance to wear</th>
</tr>
</thead>
<tbody>
<tr>
<td>(micro Deval)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Particle density and water absorption</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho$ Mg/m$^3$</td>
</tr>
<tr>
<td>WA$_{30}$ %</td>
</tr>
</tbody>
</table>

Table C.5.11: Characteristics of the mineral aggregate for unbound layer

**Test setup**

In total there were four test cycles. One test cycle consisted of a hydraulic piston-induced dynamic loading of the test structure. In total, 200,000 loading cycles (100,000 cycles without watering and another 100,000 cycles with watering the pavement) were applied to each of two pavement structures. When preparing the second pavement, the same concrete blocks were used. The blocks were rearranged in a way that the blocks on which the load was applied (central blocks) were moved to outer position and vice versa. The frequency of applying the load to the pavement surface was of 0.25 Hz. One loading cycle was performed in about 4 seconds, with minimal loading of 5 kN and maximal loading of 35 kN. The vertical displacement of the structure was monitored and followed by three LVDT sensors, mounted on a framework and placed along the test tire.

In the second and fourth test setup, water was poured into the pavement, to simulate a light rain and two heavy rainfalls. For the first, 8 litres of water per 1 m$^2$, were poured into the pavement in 8 hours. The heavy rain equals to 45 litres of water poured into the pavement (1 m$^2$ area) in 30 minutes.

Water was let to freely run out the pavement (and the wooden mould) and it was captured into containers and utensils. When performing the tests it was supposed that the “real” pavement, made on site, would have good drainage system helping water run off and away of the pavement.

**Results**

Figures C.5.13 and C.5.14 show the results for each of the four test setups: the pavement with sand bedding layer (pavement 1), the same pavement into which the water was poured into during the testing, the pavement with screed bedding layer (pavement 2), and again the same pavement that was watered. On each figure there are three lines, representing the result of deflection measurements of each of the three LVDT sensors.
It can be seen that the levels of deflections measured by sensors are considerably closer each to other by the pavement with screed bedding layer (porous mortar) than by the sand bedding layer pavement. This is attributed to higher stiffness and compactness of the screed layer, compared to the sand layer. Although the concrete blocks were laid into the sand layer and compacted as much as possible, it seems that the load applied to the pavement was distributed much more uniformly to lower layers through the screed bedding layer than through the sand layer.

When looking at the results for sand bedding layer pavement and comparing the deflections registered by sensor L1 to deflections by sensors L2 and L3, it can be seen that the first sensor values vary between the values of the remaining two. Since these two sensors were located to the opposite ends of the same block, we can assume different local deformations in the sand bedding layer. Deflections of the concrete blocks on screed bedding layer were relatively very uniform.
Figure C.5.15 shows the test results for each of the three LVDT sensors.

The results from all three sensors show that the pavement with screed bedding layer was deformed under load applied more than the pavement with sand bedding layer for first 100,000 cycles. From that point on, the trend changed radically and deflections of the screed bedding layer pavement stayed almost at the same level throughout next 100,000 loading cycles, like the screed has hardened and the layer was compacted to its maximal state. Contrary, the deflections of sand bedding layer pavement increased for all the loading time ending with the deflections (much) higher than by the other pavement.

Table C.5.12 shows the deflections at the beginning of loading and at the end of loading the pavement, minimal, maximal and the range of different pairs of deflections.
The effects of water poured into the pavements were three-fold. There was no effect observed for simulating the light rain. When simulating the extreme rain for the first time, there was a short-term effect on the pavement deflections increase. The result of the second extreme rain was in a considerable prolongation of duration of the effect or/and in the increase of deflection values. As can be seen from Table C.5.13, there was almost no difference in the effect of the two “heavy rains” on the pavement with screed bedding layer. Also, there was a limited effect of watering this pavement. Pavement with the sand bedding layer was affected in much different way. Increase in deflections caused by the first “heavy rain” was the same as for the other pavement, only that the duration was shorter. On the other hand, the second “heavy rain” affected the pavement for much longer time and with considerable increase in deflections compared to the first “heavy rain” and to the other pavement.
Table C.5.13: Effects of simulating rain

<table>
<thead>
<tr>
<th>bed layer</th>
<th>watering</th>
<th>range (cycles)</th>
<th>peak (cycles)</th>
<th>duration (hours)</th>
<th>effect (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>sand</td>
<td>first</td>
<td>23000</td>
<td>25300</td>
<td>2.5</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>second</td>
<td>66400</td>
<td>82300</td>
<td>17.7</td>
<td>0.05</td>
</tr>
<tr>
<td>screed</td>
<td>first</td>
<td>16900</td>
<td>23600</td>
<td>7.5</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>second</td>
<td>37700</td>
<td>44600</td>
<td>7.7</td>
<td>0.02</td>
</tr>
</tbody>
</table>

There was an imprint of the test wheel that was observed after finishing the tests on the pavements. The profile of the imprint was measured in five lines with results presented in Table C.5.14. It was found out that about a week time after finishing the tests on the pavement the poroelastic layer returned into its initial position making the wheel imprint not any more visible.

Table C.5.14: Wheel imprint profile lines

<table>
<thead>
<tr>
<th>Max deflections (mm)</th>
<th>Line 1</th>
<th>Line 2</th>
<th>Line 3</th>
<th>Line 4</th>
<th>Line 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>without water</td>
<td>1.66</td>
<td>1.77</td>
<td>2.65</td>
<td>3.96</td>
<td>2.30</td>
</tr>
<tr>
<td>pavement watering</td>
<td>2.92</td>
<td>2.14</td>
<td>2.84</td>
<td>4.33</td>
<td>2.63</td>
</tr>
</tbody>
</table>

Conclusions and recommendations

The aim of tests was to solve some problems of a pavement with poroelastic surface that were encountered during previous experiments. ZAG focused on the bedding layer onto which the cement concrete blocks were placed. A decision was also to choose a pavement structure where poroelastic layer is put on concrete (paving) blocks. In this way the critical gluing was made in more controlled and proper manner and environment, in laboratory. The entire pavement structure was designed to be as “strong” as possible to avoid bearing capacity failures, but the intention was also to have it as traditional as possible.

Although the concrete blocks were laid into the sand layer and compacted as much as possible, it seems that the load applied to the pavement was distributed much more uniformly to lower layers through the screed bedding layer than through the sand layer. This is attributed to higher stiffness and compactness of the screed layer, compared to the sand layer. Registered deflections of the pavement with concrete blocks on screed bedding layer were relatively very uniform, what was not the case with the sand bedding layer pavement. Even if the pavement with screed bedding layer has been deforming under load applied more than the pavement with sand bedding layer for the first part of experiment, the trend changed radically and deflections of this pavement stayed almost at the same level throughout the next part (when water was poured into the pavement). Contrary, deflections of the sand bedding layer pavement increased for all the loading time ending with the deflections (much) higher than the other pavement. There was a limited effect of watering the pavement with screed bedding layer and almost no difference in the effect of the two successive “heavy rains”. On the other hand, the second “heavy rain” affected the pavement with the sand bedding layer for much longer time and with considerable increase in deflections compared to the first one and to the other pavement.

All together the pavement where concrete blocks with poroelastic cover were placed into a cementitious screed layer has shown considerably better performance under applied conditions, compared to the sand bedding layer pavement. Considering these results it is advisable to continue with further experiments on this pavement. The research should be oriented to the field tests focusing on stability and suction forces under the typical traffic conditions.
D - References

Innovation 2.1A

[4] BRRC, Test method OCW/CRR-IX-02 "Determination of the resistance to cracking by fatigue"

Innovation 2.1B

Innovation 2.2


Innovation 2.3


Innovation 2.4