Winter Damage of Porous Asphalt

Case study using a meso-mechanics based Tool for Lifetime Optimization of PA

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ABSTRACT

With an increasing population in the Netherlands, people started to live relatively close to the primary road network. This led to major noise hindrance issues. As a solution it was decided to apply porous asphalt surfaces on the primary road network. These types of surface layers have a relatively open structure compared to traditionally applied dense asphalt mixtures. Application of porous surfaces brings along their first major advantage: noise reduction. A second major advantage of porous asphalt layers is an increased safety during rainfall. Due to its open structure water is stored and moved horizontally within the layer which reduces splash and spray effects and thus increases the visibility of drivers during rainfall. On the other hand the major disadvantage of porous asphalt layers is durability. The decisive factor for the relative short lifetime of porous asphalt is the loss of aggregates from the surface, also known as ravelling. This type of distress leads to a rough surface and decreases the material’s noise reduction potential. Further on the loosened particles cause damage to cars.

In the winter ravelling develops at a much higher speed. This results in totally damaged sections as was noticed during the winter of 2009/2010 in the Netherlands.

In this research a recently developed Lifetime Optimization Tool for porous asphalt was used to find out why different sections of the primary road network showed this type of excessive damage. Therefore LOT required information about the load, geometry and the response of these failed porous asphalt sections. In this research eight different sections were studied. The required input for LOT was determined directly from these eight sections. The results showed that in the winter the main cause for this increase in damage is caused by the reduced relaxation potential of the mortar of the mixture.

Further on the calculated performance of the eight different sections was compared with the observed performance during the winter of 2009/2010 and it was shown that they were in good agreement with each other. From this it was concluded that the Lifetime Optimization Tool is capable of explaining winter damage of porous asphalt concrete.
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1. INTRODUCTION

1.2 General

The Netherlands is a densely populated country. As a result most of the human activities take place relatively close to the primary road network. Due to this, environmental issues arise which have to be taken into consideration with respect to human well being. Two issues are especially addressed in the Netherlands, i.e. air pollution and traffic noise.

With respect to traffic noise hindrance, the policy of the government is best reflected in the obligation to apply porous asphalt (PA) on the primary road network.

Porous asphalt is a type of mixture that consists of relative coarse aggregates bound together by a mixture of sand, filler and bitumen hereafter called mortar.

After laying and compaction this results in a structure with a relative large amount of interconnected voids. Compared with traditionally applied dense asphalt concrete (DAC), porous asphalt has a very open structure with void contents around 20 %. See figures 1 and 2.

![Figure 1: Comparison of two types of asphalt concrete [Hagos 2008]](image)

![Figure 2: Porous asphalt](image)
This relative large amount of voids has two major advantages:

- noise reduction
- increased visibility of drivers during rainfall

Traffic noise results from a combination of aerodynamic effects in combination with mechanical effects. At driving speeds above 50 km per hour, the noise generated by contact between tire and pavement surface becomes dominant.

Due to its open structure the application of porous asphalt has proved to be an excellent measure to reduce this noise generation. Compared to dense asphalt concrete noise reductions up to 3 dBA can be achieved, while higher noise reductions up to 6 dBA can be achieved by application of a different type of porous asphalt e.g. two layer porous asphalt.

The relative high amounts of interconnected voids also allow rainwater to be stored and moved horizontally in the porous asphalt mixture. This results in almost no presence of water at the surface which leads to:

- reduced splash and spray effects
- no aquaplaning
- no reflection of light

These factors increase the visibility of drivers during rainfall.

Besides the mentioned advantages, the main disadvantage of applying porous asphalt surface layers is durability e.g.: a relative short lifetime. In most cases the decisive factor for this relative short lifetime is the loss of stones from the surface also known as ravelling. See figure 3.

Ravelling results in a rough surface which results in an increase of traffic noise and further on the loosened aggregates cause damage to cars.

Figure 3: Loss of stones of porous asphalt
Taking into account the structure of the mixture it can be concluded that ravelling is caused either by cracking of the mortar itself, cohesive failure, or by failure at the interface between stone and mortar, adhesive failure. See figure 4.

Figure 4: Failure modes in porous asphalt [Kringos 2007]

In the past various empirical approaches have been tried to improve the ravelling resistance of porous asphalt [CROW 1996]. Due to the empirical nature of all the approaches, a fundamental understanding of the ravelling phenomenon is not yet available.

In utilizing a mechanistic approach to understand the ravelling behavior, recently, a meso-mechanistic tool (LOT) has been developed at the Road and Railway Engineering section from the Delft University of Technology to understand the bulk behavior of porous asphalt mixtures based on the component material behavior. This tool gives insight into the mechanical phenomena that take place in the mixture during the passage of a wheel load. The tool uses finite element models which translate information about the load, geometry and material behavior into stress and strain signals. Based on these signals the damage accumulation of the cohesive parts, the mortar, and adhesive parts, the mortar-aggregate interface, of the mixture can be determined. This approach thus enables the design of durable porous asphalt mixtures.
During the winter of 2009/2010, excessive ravelling developed at different sections of the Dutch primary road network. The Lifetime Optimization Tool, LOT, was used to explain the reason why these sections showed significant ravelling damage \cite{Huurman 2009}. The results showed that at low temperatures, the main cause for this excessive ravelling was the reduced relaxation behavior of the mortar in relation to pavement deflection and temperature stresses. This resulted in a rapid increase of damage at the interface between stone and mortar, the adhesive zones.

1.2 Initiation of this research

The Lifetime Optimization Tool was found capable of giving insight into the ravelling behavior of porous asphalt in general. In a previous study \cite{Huurman 2009}, LOT was used to explain the excessive ravelling of porous asphalt during the winter period. The results of this study indicated that excessive raveling is caused mainly by the reduced relaxation potential of the mortar. The findings were however more general since the calculations considered a general experimental case, which was a simplified version of the reality. Several assumptions were made concerning the information that LOT requires.

This research is aimed at validating the possible general explanation that LOT gives for winter damage by making use of actual pavement information gathered from the damaged sections of the Dutch primary road network. This research contributes to the existing body of literature by making use of actual pavement information, which has not been done by any previous research using LOT.
1.3 Research objectives

The objective of this research is to validate the LOT theory on its capability to explain the excessive ravelling that develops during the winter. Many opinions exist concerning the cause of winter damage of porous asphalt. These opinions are used as input for the development of theories and models.

This research is not aimed at combining these theories and models or trying to give another explanation for winter damage, but to explicitly validate one theory: *excessive ravelling in the winter is caused mainly by reduced relaxation potential of the mortar.*

The objective of this research is to find valid answers for the following questions:

- Is the proposed theory correct? Does poor relaxation behavior cause excessive ravelling?
- If not, why?
- If yes, how can LOT then be put in practice to avoid such failures on the Dutch motorways in the future?

In order to answer the before mentioned questions an understanding needs to be developed concerning the following sets of questions:

Set 1:
- What is winter damage of porous asphalt?
- What is LOT?
- How was LOT used to explain winter damage?

Set 2:
- What are other theories that explain winter damage?
- What does LOT not consider?
- What is done currently to improve ravelling?
1.4 Outline of the report

First attention is paid to the first set of sub questions. These questions are answered in chapter two. The answers to the second set of sub questions follow from the literature review which is presented in chapter three of this report.

Based on the obtained insight from chapter two and three, a research approach is adapted which is presented in chapter four. Chapter four discusses which path is followed in order to obtain the required information. The obtained required information is presented in chapter five while chapter six discusses the results of this research. In chapter seven and eight results are presented from additional performed experiments while the conclusions and recommendations of this research are summarized in chapter nine.
2. BACKGROUND ON THE TOPIC

2.1 General

This research is aimed at validating the explanation that resulted from a previous conducted research were LOT was used to explain winter damage of porous asphalt. The aim of this chapter is to get the reader familiar with the topic. In this chapter the focus lies on providing answers for the first set of sub-questions:

- What is winter damage?
- What is LOT?
- How was LOT used to explain winter damage?

2.2 Winter damage of porous asphalt

As mentioned in the introduction the main issue with porous asphalt is durability. The decisive factor that influences the durability is the loss of stones from the surface, ravelling.

This loss of stones occurs due to failure at the weakest link within the stone-stone contact region. In this region two zones can be distinguished: the cohesive zones (the mortar) and the adhesive zones (interface mortar and stone).

Due to the open structure of porous asphalt, these zones are strongly influenced by climatic factors. Oxygen, UV radiation, temperature etc. cause hardening of the mortar. As a result, the material behaves stiffer and looses its viscous properties. This results in a reduced relaxation potential and more elastic behavior. In the winter when temperatures drop the relaxation potential decreases even further.

As a result stresses that develop in these zones are not relaxed away sufficiently and damage starts to initiate at a higher speed until failure occurs.

As soon as the first stone dislodges from the surface, ravelling develops at a much higher speed since the stones are less supported by their “neighbor”. This results in totally damaged sections which effect human safety and brings along unwanted expenses.
From above the following is concluded:

- due to its open structure, the cohesive and adhesive zones in porous asphalt mixtures are strongly influenced by non mechanical effects, which results in hardening
- hardening of the materials results in a reduced relaxation potential
- especially at low temperatures, this reduced relaxation potential causes the different zones to be stressed heavily which results in the initiation of damage

Figure 5 shows the average temperatures during a 24h period from the last three winters. It can be seen that during the last winter temperatures were significantly lower then the two previous winters in the Netherlands.

![Temperature behaviour during a 24h period, during the last winters](image)

**Figure 5: Temperature behaviour during a 24h period, during the last winters**

These relatively low temperatures during the winter of 2009/2010 manifested itself in the form of damaged sections which showed more severe ravelling than usual. See figures 6 and 7.
It becomes clear that in the winter, when temperatures are low, the circumstances are such that they cause a rapid increase in loss of stones which results in totally damaged sections. This is called winter damage of porous asphalt.

To find out why this type of failure occurred during the last winter, the Lifetime Optimization Tool was used. Before discussing the results of that study, first attention is paid to LOT. In the next paragraph the principle of LOT is discussed.
2.3 LOT

2.3.1 Background

Generally the design strategy in road engineering is based on a response calculation made by application of one of numerous linear elastic or visco-elastic multi layer program codes. The calculation translates information about load, geometry and material behavior i.e. stiffness, into material response, i.e. stress and strain. The calculated stress and/or strain values are then used as input in e.g. fatigue relationships which are determined by means of laboratory tests, to determine the design life of the pavement structure. The structural design is modified in case the design does not meet the required design life. Hereafter the process is repeated until the structural design meets the design demands.

In LOT the same principle is adapted but in this case the structure to be designed is not the pavement itself but the porous asphalt mixture. Ravelling, the loss of stone from the road surface, is the decisive factor for the service life of porous asphalt and is directly related to the performance of the porous asphalt mixture itself and not much dependant on the structural design of the pavement. Since ravelling is taking place at the surface of the asphalt layer it becomes more an in mixture problem then a structural problem. Therefore in LOT the porous asphalt mixture is modeled and not the pavement structure itself.

The Lifetime Optimization Tool (LOT) is a meso mechanistic tool that gives insight into the development of stresses and strains in a porous asphalt mixture due to loading by making use of finite element models. The next subparagraph discusses the principle of the tool in general. More detailed information can be found elsewhere, [Huurman 2008].
2.3.2 Principle

2.3.2.1 Introduction

As mentioned in the previous paragraph LOT is based on the triangular relation between geometry, load and material behaviour i.e. stiffness. If information about these three parameters is known any porous asphalt mixture can be analyzed with LOT. This paragraph discusses how this information is used in LOT.

2.3.2.2 Geometry

Until now it becomes clear that ravelling is related to binding failure within the stone contact region. Therefore the potential types of failure in stone contact regions must be considered. In LOT the stone contact region consist of different parts, see figure 8.

![Figure 8: Schematic illustration of idealized stone-stone contact](Mo 2010)

Within LOT two types of failure are distinguished:

1) Failure through the bitumen-rich interlayer or adhesive zone: this implies that bonding fracture occurs very close to stone surface and the fractures surfaces are still coated with bituminous material after failure

2) Failure through the pure mortar or cohesive zone: this implies that the location of fracture is not close to stone surface; the fractures surfaces are coated by mortar.
Figure 9 shows that these two types of failure do really occur in practice:

![Figure 9: Ravelled aggregates indicating the type of failure](image)

In LOT these zones are loaded in fatigue. This indicates that information is required about stress levels and the strength of these parts to calculate the fatigue life. The stress levels of these parts depend on the mixture geometry, the applied load on the mixture and the stiffness of mixture components.

The stresses and strains that develop in these zones due to loading are obtained by making use of finite element models. Within LOT three types of finite element models can be used:

- 2D idealized
- 3D idealized
- 2D photo/scan

For this research only the 2D idealized model is of interest. Selection of the other two models requires computation power and time, which jeopardizes the timely completion of this study. Another, more important reason is that this research is aimed at validating an explanation which followed from a study [Huurman 2009] where the 2D idealized model was used. Therefore only the 2D idealized model is used in this research.
Figure 10 gives an illustration of the 2D idealized model.

![Adhesive zones in the model](image1)

*Figure 10: Adhesive zones in the model [Huurman 2008]*

In order to model the geometry of the porous asphalt mixture, see figure 11, the following information is required:

- Size of the stones
- Density of the stones
- Percentage of the stones
- Density of the aggregates in the mortar
- Density of the bitumen
- Percentage of the bitumen
- Percentage of the voids

![From mixture to model](image2)

*Figure 11: From mixture to model*

Based on these seven parameters any porous asphalt mixture can be modeled within LOT. For a detailed description of the procedure, reference is made to Huurman 2008. After the mixture is modeled, the next step in LOT is assigning material properties i.e. stiffness of the cohesive zones and adhesive zones.
2.3.2.3 Stiffness of the cohesive and adhesive zones

As mentioned in the beginning of this chapter, within LOT two zones are of interest, the cohesive and the adhesive zones. In order to calculate stresses and strains in these zones, first these zones have to be assigned with a stiffness value. The stiffness of the cohesive zones, the mortar, is obtained from so called frequency sweep test performed on mortar samples. See figure 12.

\[ \tau = \frac{2 \cdot T}{\pi \cdot r^3} \]  
\[ \gamma = \frac{r \cdot \theta}{h_{\text{eff}}} \]

**Figure 12: DSR test setup, left: mortar sample, right: setup DSR machine**

The frequency sweep tests are done using the DSR (Dynamic Shear Rheometer) machine. An oscillating torque is applied to the sample and the strain in the sample is measured and translated into the shear modulus e.g. stiffness. The following formulas are used:

\[ \tau = \frac{2 \cdot T}{\pi \cdot r^3} \]  
\[ \gamma = \frac{r \cdot \theta}{h_{\text{eff}}} \]

*where:*
\[ \tau = \text{shear stress (MPa)} \]
\[ T = \text{applied torque (N.mm)} \]
\[ r = \text{specimen radius (3 mm)} \]
\[ \gamma = \text{shear strain} \]
\[ \theta = \text{measured deflection angle (rad)} \]
\[ h_{\text{eff.}} = \text{specimen effective height (12.742 mm)} \]
After the shear stress and shear strain are calculated, the complex shear modulus is determined with the formula:

\[
G^* = \frac{\tau}{\gamma} = |G^*|e^{i\delta} = G' + G''
\]  
\[\delta = \arctan\left(\frac{G'}{G''}\right)\]  

where:
- \(G^*\) = complex shear modulus (MPa)
- \(G'\) = real part of the complex shear modulus, elastic component (MPa)
- \(G''\) = imaginary part of the complex shear modulus, visco component (MPa)
- \(\delta\) = phase angle, shift between stress and strain during oscillation (degr.)

In LOT the stiffness of the mortar is modeled by means of visco-elastic model. This indicates that the measured DSR data has to be translated first into a visco-elastic model. This procedure is not discussed here and will be explained in chapter four of this report.

Within LOT the adhesive zones are modeled as very thin zones, 0.01 mm. As a result there is hardly any material that may deform and as such the adhesive zone is modeled by using a high stiffness. An estimate of the stiffness follows from the stiffness values from the cohesive zones. The following formulas are used:

\[
K_n = \frac{E^*}{d}
\]  
\[
K_s = \frac{G^*}{d} \text{ with } E^* = 2(1 + \nu) \times G^*
\]

where:
- \(K_n\) = normal stiffness of the adhesive zone (MPa/mm)
- \(K_s\) = shear stiffness of the adhesive zone (MPa/mm)
- \(d\) = thickness of the adhesive zone, 0.01mm
- \(G^*\) = shear complex modulus of mortar (MPa)
- \(E^*\) = uniaxial tensile complex modulus of mortar (MPa)
- \(\nu\) = Poisson's ratio, \(\nu=0.45\)
The calculated stiffness values are then assigned to the model. The next step is modeling the load. This is discussed in the next paragraph.

2.3.2.4 Load

After the stiffness values of the cohesive and adhesive zones are assigned to the model, the response of these zones under the action of loading can be calculated after the load is modeled. In LOT three types of loadings can be distinguished:

- Traffic
- Pavement deflections
- Temperature fluctuations

Traffic

In LOT traffic loads, the load of a tire to the pavement surface is applied via forces acting on individual stones. For this purpose use is made of measured data of contact stresses that develop between tire and pavement structure [Groenendijk 1998, de Beer et al 1997]. This led to the development of two signals illustrated in figure 13.

![Figure 13: Applied load signals in LOT](image)

In figure 13, function a represents the vertical stress while function b represents the shear stress. More information about these signals can be found in Huurman 2008. In LOT the vertical stress and shear stress are calculated out of the contact stress between tire and pavement surface. For this purpose use is made of table 1.
Table 1: Transfer functions to calculate bulk stresses

<table>
<thead>
<tr>
<th></th>
<th>Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical stress (MPa)</td>
<td>(0.025 \times \sigma_{\text{contact}})</td>
</tr>
<tr>
<td>Shear stress (MPa)</td>
<td>(0.3 \times \sigma_{\text{contact}})</td>
</tr>
</tbody>
</table>

The next step is translating the shear stress and vertical stress into shear and vertical forces acting on the individual stones. In the 2D idealized model the area on which the load is applied on each stone is known; see area A in figure 14.

![2D idealized model, A indicates the footprint area](image)

Figure 14: 2D idealized model, A indicates the footprint area

Multiplying the vertical and shear stresses with this area, results in forces working on the surface of the model. In this way traffic loadings are simulated in LOT. Also see figure 15.

![Principle of translating bulk stresses in to forces on particles](image)

Figure 15: Principle of translating bulk stresses in to forces on particles
Pavement deflections
In LOT pavement deflections are fed to the model as prescribed deformations at model boundaries. These deflections follow from measured deflection values using Falling Weight Deflection measurements or simply from calculations using multi-layer analysis programs. Simulation of these deflections, results in additional stresses in the cohesive and adhesive parts of the mixture.

Temperature fluctuations
In standard LOT simulations the stresses and strains in the cohesive and adhesive zones follow from traffic loadings and pavement deflections. In the winter a third type of loading occurs, temperature fluctuations. These temperature fluctuations want to shrink or expand the mixture. As a result additional stresses develop in the cohesive and adhesive parts. To save computational time, the stones of the 2D idealized model are modeled as rigid bodies in LOT. That means that they can’t deform. However to include the effect of additional stresses due to deformations that follow from temperature fluctuations, a second model is used where stones are modeled as physical bodies.

The same geometrical input and stiffness values are assigned to this model. The procedure is summarized hereafter:

- generate model using same geometrical input and stiffness values
- stones are modeled as physical bodies
- calculate the expansion of mortar and stone due to a temperature difference, use expansion coefficients of stone and mortar
- feed this expansion to the model as prescribed movements at model boundaries
- Select stresses at points of interest.

In this way stresses that results from all the three types of loading, are included within LOT calculations.

After all the required information about the geometry, load and material behavior is assigned to the model the simulations are performed. The stresses and strains are then selected and used as input for damage models.
During the development of LOT two damage models were developed one for the cohesive zone and one for the adhesive zone. Both models are discussed briefly hereafter. For detailed information reference is made to Huurman 2008.

### 2.3.2.5 Damage models

In LOT the cohesive zones and adhesive zones are loaded in fatigue. Therefore damage models were developed on the basis of numerous fatigue tests. In practice a porous asphalt layer is subjected to a complex state of stresses meaning no pure shear or tension but rather a combination. This indicated that fatigue tests had to be performed using different stress signals and not only sinusoidal signals which are commonly used in classical fatigue tests.

For the cohesive zones, mortar fatigue tests were performed on cylindrical samples, see figure 16.

![Mortar samples and test setup](image)

**Figure 16: Mortar samples (left) and mortar fatigue test setup (right) [Mo 2010]**

Interpretation of test results led to the following model:

\[
N_f = \left( \frac{W_{\text{initial \_ cycle}}}{W_0} \right)^{-b} \quad \text{with} \quad W_{\text{initial \_ cycle}} = \sum W_y = \int \sigma_y d\varepsilon_y
\]

where:
- \(W_0, b\) = model parameters, determined during testing
- \(\sigma, \varepsilon\) = stresses and strains that follow from simulations

\[\text{(7)}\]
Chapter 2 Background on the topic

For the adhesive zones fatigue tests were performed on stone-bitumen-stone samples, see figure 17.

![Adhesive zone fatigue testing setup](image)

**Figure 17: Adhesive zone fatigue testing setup [Huurman 2008]**

Interpretation of test results led to the following damage model for the adhesive zones:

\[
D = \left(\frac{\sigma_{et}}{\sigma_0}\right)^{n_0} \quad \text{for} \quad \sigma_{et} > 0, \quad D = 0 \quad \text{for} \quad \sigma_{et} \leq 0 \quad \text{with} \quad \sigma_{et} = \sigma_n + \frac{\tau}{\tan \phi}
\]  \hspace{1cm} (8)

where:
- \(D\) = rate of damage accumulation (-/s)
- \(\sigma_{et}\) = equivalent tensile stress (MPa)
- \(\sigma_n\) = adhesive zone normal stress (MPa)
- \(\tau\) = adhesive zone shear stress (MPa)
- \(\phi\) = friction angle (degr.)
- \(n_0\) = model parameter (-)
- \(\sigma_0\) = model parameter (MPa)

The stresses and strains that follow from the LOT simulations are then used as input for the damage models. Hereafter the damage development can be calculated in the cohesive and adhesive parts and based on this, insight can be obtained about the performance of any modeled porous asphalt mixture.
2.3.3 Evaluation

Paragraph 2.2 focused on the principle of LOT. As explained in this paragraph, LOT requires information about the geometry and material behavior of the cohesive and adhesive zones of a porous asphalt mixture. Further on the loading on the mixture is required. Using the simulation results, it is possible to calculate the damage development in the mortar and at the interface between mortar and stone.

From this it is concluded that LOT focuses on the critical stone-stone contact region where failure may occur firstly. This indicates that LOT explains the initiation of ravelling while visible ravelling damage in reality tends to develop over a longer period. This means that the life expectancy computed by LOT is not directly related to the actual lifespan of porous asphalt. However a good correlation between these two can be expected when the evolution of ravelling is taken into account. Finally it should be mentioned that LOT was validated successfully by means of accelerated pavement tests. Detailed information regarding the validation of LOT can be found in the main LOT report [Huurman 2008].

In a previous conducted study LOT was used to explain ravelling of porous asphalt in the winter [Huurman 2009]. The next paragraph discusses which required information was used and the explanation that followed from that study.
2.4 LOT + winter damage

2.4.1 Introduction

As mentioned in the introduction, during the last winter extreme ravelling developed at different sections of the primary Dutch road network. LOT was used to explain why these sections showed this type of damage [Huurman 2009]. The previous paragraph already stated that LOT requires information about the geometry, load and material behaviour. This paragraph discusses which information was used in the LOT simulations during the study of Huurman 2009 and the obtained results. For detailed information and procedures, reference is made to the paper which is included in the appendices of this report.

2.4.2 Geometry

In this study the 2D idealized model was used. The geometrical input used for generation of the 2D model followed from a representative Dutch porous asphalt mixture from the Dutch standards (RAW). See table 2.

<table>
<thead>
<tr>
<th>Table 2: Used geometrical input for explaining winter damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equivalent stone radius</td>
</tr>
<tr>
<td>Stone density (kg/m³)</td>
</tr>
<tr>
<td>Percentage of stone (%)</td>
</tr>
<tr>
<td>Mineral in mortar density (kg/m³)</td>
</tr>
<tr>
<td>Bitumen density (kg/m³)</td>
</tr>
<tr>
<td>Bitumen percentage (%)</td>
</tr>
<tr>
<td>Void content (%)</td>
</tr>
</tbody>
</table>

2.4.3 Stiffness values

The stiffness values assigned to model were derived from laboratory tests performed during the development of LOT. Properties of an SBS modified mortar were assigned to the cohesive zones. The assigned stiffness of the adhesive zones followed from the properties of the SBS modified mortar.
2.4.4 Load

Three types of loadings were simulated:

- Pavement deflections
- Traffic
- Temperature fluctuations

**Pavement deflections**

Pavement deflections were calculated using a multi-layer linear elastic program, WESLEA. This program requires information about the structural design of the pavement i.e.: layer thicknesses and stiffness values of each layer and the load on the pavement. For this purpose use was made of a representative Dutch structural design. See table 3.

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (mm)</th>
<th>Poisson’s ratio</th>
<th>Stiffness (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>-10 °C</td>
</tr>
<tr>
<td>PA</td>
<td>50</td>
<td>0.35</td>
<td>10475</td>
</tr>
<tr>
<td>DAC</td>
<td>200</td>
<td>0.35</td>
<td>20950</td>
</tr>
<tr>
<td>Unbound base</td>
<td>225</td>
<td>0.4</td>
<td></td>
</tr>
<tr>
<td>Sand sub base</td>
<td>1000</td>
<td>0.4</td>
<td></td>
</tr>
<tr>
<td>Subgrade</td>
<td>∞</td>
<td>0.4</td>
<td></td>
</tr>
</tbody>
</table>

Information about the load is given in table 4.

**Table 4: Used input for WESLEA**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Length wheel patch (mm)</td>
<td>170</td>
</tr>
<tr>
<td>Diameter wheel patch (mm)</td>
<td>330</td>
</tr>
<tr>
<td>Contact area (mm²)</td>
<td>56100</td>
</tr>
<tr>
<td>Wheel load (kN)</td>
<td>50</td>
</tr>
<tr>
<td>Tire pressure (MPa)</td>
<td>0.891</td>
</tr>
</tbody>
</table>

Using the information from table 3 and 4, deflections were calculated up to 2 m away from the centre of the model. It was assumed that 15 m away from the centre the model experiences no effect of the tire, meaning that at 15 m the deflection was set to zero.
Using this information, an exponential function was fitted to the calculated data to estimate the complete deflection bowl starting from the centre up to 15 m away. Figure 18 illustrates how the deflection bowl is simulated in LOT.

![Figure 18: Example of pavement deflections fed as a whole to the model [Huurman 2009]](image)

**Traffic**

During the LOT calculations, a free rolling wheel of 50 kN travelling at a speed of 76.5 km/h was simulated. A total of 10000 equivalent 50 kN wheel passages were assumed. During a 24 hour period, different distributions were assumed:

- 15% passing during the night with 85% passing during the day
- 33% passing during the night with 66% passing during the day
- 1% passing during the night with 99% passing during the day

**Temperature fluctuations**

Temperature stresses were calculated as explained in the previous paragraph. Only the fluctuations during a 24 h period were considered. The following expansion coefficients were used:

<table>
<thead>
<tr>
<th>Table 5: Linear thermal expansion coefficients for stone and mortar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stone (Sandstone/Greywacke)</td>
</tr>
<tr>
<td>αL</td>
</tr>
<tr>
<td>6.6 *10e-6/°C</td>
</tr>
</tbody>
</table>
During these simulations the temperature fluctuations during a 24 h period were assumed to show a sinusoidal movement. This resulted in temperature stresses which were distributed according to the assumed sinusoidal fluctuation. See figure 19.

![Figure 19: Calculated temperature stresses during a 24h period](image)

**2.4.5 Damage models**

As stated in the previous paragraph the stresses and strain that follow from the simulations are used as input for the damage models. Both damage models require model parameters. For both damage models, model parameters were used which were obtained during the development of LOT.

For the cohesive zones the following parameters were used:

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>W0 (MPa)</th>
<th>n</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.981</td>
<td>2.643</td>
</tr>
<tr>
<td>10</td>
<td>0.363</td>
<td>2.986</td>
</tr>
</tbody>
</table>

For the adhesive zones, the damage model parameters were determined for two types of stone, Greywacke and Bestone, during the development of LOT. In the study of Huurman 2009, the average values of the obtained parameters for both types of stone were used. See table 7.
Table 7: Damage model parameters used for the adhesive zones

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>σ₀ (MPa)</th>
<th>n</th>
<th>θ (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-10</td>
<td>20.63</td>
<td>2.74</td>
<td>70.2</td>
</tr>
<tr>
<td>0</td>
<td>12.18</td>
<td>5.14</td>
<td>25.1</td>
</tr>
<tr>
<td>10</td>
<td>10.56</td>
<td>3.56</td>
<td>31.9</td>
</tr>
<tr>
<td>20</td>
<td>8.35</td>
<td>3.17</td>
<td>35.5</td>
</tr>
</tbody>
</table>

2.4.6 Results

Finally using all the assumed information LOT simulations were performed at 4 different temperatures: -10°C, 0°C, 10°C and 20°C.

The simulation results are shown in figure 20.

![Figure 20: Relative daily damage as function of average daily temperatures](image)

Based on the trends plotted in figure 20 it was observed that in the winter a porous asphalt mixture is subjected to two types of loading:

- Strain controlled loadings:
  - Due to temperature fluctuations the material wants to shrink or expand. Since the pavement cannot shrink or expand, this movement is counteracted by opposite strains which results in additional stresses.
- Under the action of traffic the pavement deflects. These deflections depend on the structural design of the pavement. It is known that the contribution of the porous asphalt mixture to the structural stiffness is very limited. This indicates that the surface layer cannot limit deflections even when it becomes very stiff. From this it was concluded that pavement deflections result in a type of loading that is best described by strain controlled.

- Stress controlled loadings:
  - Due to contact between tire and surface forces develop. The principle of equilibrium of forces results in reaction forces on the surface of the porous asphalt layer. This type of loading is thus mainly stress controlled.

Further on the results showed that at relative low daily temperatures, the main reason for excessive ravelling was the reduced relaxation potential of the mortar i.e. the cohesive zones. This effect caused the adhesive zones to be stressed heavily, resulting in failure of these parts. Temperature fluctuations during the day caused an increase of stress in these zones.

At relative low daily temperatures this effect causes a rapid increase of damage. See #2 in figure 20. This rapid increase of damage seemed to correlate with the excessive damage which occurred at different sections of the primary network during the last winter.

Further on the best performance was obtained at 0 °C while at higher temperatures it was observed that the material is flexible enough to relax away stresses that follow from pavement deflections and temperature fluctuations. It was observed that at higher temperatures the strength of the material became important in offering resistance against damage development.
2.4.7 Conclusions

Based on this study with LOT, the following conclusions are drawn:

- The tool is capable of explaining ravelling damage occurring during the winter
- The main cause of winter damage is the reduced relaxation potential of the mortar
- It was observed that in the winter the strain controlled loadings require relaxation potential while the stress controlled loadings required strength
- It was observed that in the winter the rapid increase of damage was mainly influenced by the strain controlled loadings. From this it was concluded that the actual strength of the cohesive and adhesive zones is not required.
- The optimum performance of porous asphalt in cold weather conditions depends on the availability of a mortar that remains viscous at low temperatures with good adhesion strength at high temperatures

2.5 Evaluation

This chapter discussed the first set of sub-questions. Until now we have seen that LOT seems to explain winter damage. However one should realize that the explanation was based on results obtained using a general experimental case.

Further on other opinions and theories exist which give an explanation for winter damage. And finally it can also be argued that within LOT some effects are not taken into account such as: the effect of moisture, salt used in winter etc.

Therefore prior to formulating the research approach a literature review was performed to find out what other theories and explanations exist with regard to winter damage and which methods are currently used to prevent winter damage. These topics are discussed in the literature review presented in the following chapter.

Chapter three provides answers to the second set of sub-questions.
3. LITERATURE REVIEW

3.1 Introduction

Ravelling of porous asphalt has been a hot issue in the Netherlands for many years now. This indicates that there are many theories which try to explain ravelling of porous asphalt. As part of this research a literature review was performed to find out what these other theories are. In this chapter first attention is paid to ravelling of porous asphalt. It is important to understand what ravelling is in order to decide if a theory is capable or not to give an explanation. After this, two theories are discussed tend to give an explanation for winter damage. Hereafter currently used methods are described and finally this chapter ends with an evaluation of the literature review. Based on this evaluation and the conclusions summarized in paragraph 2.4.7, a research approach is adapted which is presented in chapter 4.

3.2 Ravelling of porous asphalt

3.2.1 General

The most significant and by far the most decisive factor for the durability of porous asphalt is ravelling. Porous asphalt has a high percentage of voids and is characterized by a high percentage of coarse aggregates in the mixture. The bituminous mortar binding the aggregates together is believed to be subjected to tensile, compressive and shear forces.

In the process of stress development, the material is subjected to damage which is accumulated till ravelling (loss of aggregate from the surface) occurs. Considering the open structure of porous asphalt concrete, ravelling can happen in a cohesive or adhesive failure mode. Cohesive failure occurs in a sudden or progressive breaking of the mortar while adhesive failure occurs in the stone-mortar interface. See figure 21.
The resistance of a porous asphalt mixture to ravelling is mainly influenced by:

- **Aggregate type**: the adhesion properties are influenced by the texture of the stone surface. Generally a rougher surface will result in a stronger adhesive bond. Also the size and shape play an important role with respect to in mixture deformations. Another important factor is the chemical composition of the aggregate. During the LOT research it was observed that the Bestone aggregate showed a better performance then the Greywacke aggregate. This proved that the type of aggregate is an important factor.

- **Bitumen content**: the amount of bitumen determines the thickness of the mortar layers surrounding the stones. In case of low bitumen contents the mortar layers are thinner and easily influenced by climatic factors. The thin mortar layers also decrease the adhesive bond between stone and mortar since stones are not embedded sufficiently in the mixture.

  When increasing the binder content other measures have to be taken to prevent dripping of the bitumen during transport. For this purpose use is made of additives such as cellulose fibers.

- **Properties of the mortar**: due to the open structure the mortar is hardened due to climatic factors. This increases the stiffness and under loading conditions the material is not flexible enough to relax stresses and therefore micro-cracks develop in the material. It was already observed that this factor is dominant in the winter.
Climate conditions: usually ravelling develops several years after construction. However during the last two winters in the Netherlands, excessive ravelling was noticed on different sections of the road network. This indicates that climate conditions, especially temperatures are an important factor, since it influences the material behavior.

Construction conditions: the circumstances during the construction of porous asphalt are important to ensure a qualitative good layer. In case the material has cooled down too much it becomes hardly impossible to compact. Over compacting will damage the material and create weak spots within the layer. Important factors are: thermal isolation of the mixture during transport, transportation time and temperature.

Further on results from previous research [Miradi 2009], indicated that other factors such as traffic, percentage of coarse aggregates in the mixture, voids content and the number of cold days also contribute to the ravelling resistance of porous asphalt. See figure 22.

![Figure 22: Two bar diagrams giving the five most influential variables explaining PA ravelling after 5, left, and 8, right, years of service [Miradi 2009]](image)

So from above it’s concluded that the durability of porous surface layers strongly depends on the magnitude of the bond strengths of the cohesive and adhesive zones, the degree to which they are influenced by aging effects (hardening), the mixture composition, the magnitude of stress imposed on the pavement and workmanship. Hereafter two theories are discussed which tend to explain ravelling of porous asphalt in the winter.

31
3.2.2 The effect of moisture

Moisture damage in asphalt mixtures can be defined as the progressive functional deterioration of a pavement mixture by loss of the adhesive bond between the asphalt mortar and the aggregate surface and/or loss of the cohesive resistance within the asphalt mortar principally by the action of water [Kiggundu and Roberts 1988]. Adhesive (within the aggregate-mortar bond) and cohesive (within the mortar) failures are the last step in the process that starts with different modes of moisture transport and results in the generation of moisture damage.

This damage results in separation of the aggregates and the binder which is known as stripping.

Many studies have been performed to collect, describe and measure the moisture susceptibility of asphaltic mixtures. Most of these are aimed at a comparative measure of moisture damage, either via visual observations from field data, laboratory tests or mechanical tests.

In her research, Kringos [Kringos 2007] developed a micro-mechanical finite element model to simulate combined mechanical and moisture induced damage in asphaltic mixtures. The model assigns an elastic material property to the stones and an elasto-visco-plastic material behaviour for the mortar that couples moisture and mechanical effects together.

Moisture diffusion into the mortar layer, moving towards the aggregate-mortar interface and mortar erosion, due to high water pressures caused by the pumping action of traffic, are identified as the main moisture –induced damage processes. These damage processes are implemented in a finite element program, named RoAM (Ravelling of Asphalt Mixtures) which was developed in the Section of Structural Mechanics of Delft University of Technology, as a subsystem of the finite element system CAPA – 3D.
The model enables the simulation of moisture infiltration due to pressure and moisture-gradient driven processes but it requires desorption characteristics as well as the diffusion and the dispersion coefficients to be known. The two damage processes distinguished before require controlling parameters which are summarized below:

- Moisture diffusion, the controlling parameters are:
  o Moisture diffusion coefficient
  o Maximum moisture capacity

- Mortar erosion, the controlling parameters are:
  o Mortar desorption coefficient
  o Mortar diffusion coefficient
  o Mortar dispersion coefficients
Methodologies were developed in order to determine these parameters which relate the moisture content and erosion to mechanical weakening. Although this model is expected to be able to explain processes that lead to ravelling of porous asphalt there are some points that could be argued:

- First of all determination of this parameters requires micro-scale laboratory testing on a large scale. This requires a lot of time and one should realize that in practice other factors may induce ravelling before moisture damage occurs.

- Further on mechanical tests are required to get insight into the effect of moisture on the mechanical response of the moisture-induced mixture components. In the model of Kringos the effect of moisture on the aggregate-mortar bond was determined by pure tension tests. The test setup consisted of a steel stub coated with a thin film of mortar bonded to aggregate. One can imagine that these tests are far from reality and therefore are considered not sufficient to explain excessive ravelling.

- In the model pavement deflections and temperature effects are not taken into account. Both parameters are considered important to understand ravelling of porous asphalt in the winter.

- Another aspect could be the presence of moisture in the mixture. Since ravelling occurs at the surface of the top layer, the question could be raised if there is enough moisture present at the surface to start the processes that Kringos summarized in her research. In case there is no moisture present this would indicate that according to this model no ravelling would occur. It is believed that moisture damage of porous asphalt is a type of distress that occurs at the bottom of the porous asphalt layer and not specifically at the surface where ravelling occurs.

- Finally the most important point is that Kringos model is not validated by means of full scale pavement tests.
3.2.3 The effect of freeze-thaw cycles

As mentioned earlier micro cracks will develop in the porous asphalt due to a combination of loading and the increased hardness of the mortar due to ageing. At low temperatures this effect increases and micro cracks develop at a high rate. During frost–thaw cycles water goes inside these micro cracks. When it starts to freeze this water expands in these cracks. Since the stone-stone contact region behaves to brittle to relax this expansion, micro cracks start to grow. For every freeze-thaw cycle this effect increases and finally results in failure in the stone–stone contact region. As the frost-thaw cycles occur frequently damage increases at a higher rate.

This idea seems logical however there is one point that could be argued. First of all, in case of frost, de-icing salts are applied on the pavement surface. It is known that the de-icing salts decrease the freezing point of water. In case temperatures remain above this freezing point in the winter this would indicate that no freezing would occur and therefore micro cracks would not expand. Nevertheless it is fair to say that frost-thaw cycles cause an increase of damage but it should be pointed out clearly that the cause of winter damage lies somewhere else. To the author’s knowledge, the frost-thaw cycle effect could be identified as a secondary process which becomes important after the development of micro cracks due to hardening has occurred.

3.2.4 Summary

It becomes clear that many factors have to be taken into consideration when porous asphalt is applied. On the other hand it seems that all the factors are known but the problem seems quantifying them e.g. which factor has major contribution, which not etc. Generally it is concluded that increasing the durability of porous asphalt mixtures can be described as designing mixtures with a high level of ravelling resistance. For this purpose different methods are possible.

In the next paragraph some methods are discussed which are currently used in the Netherlands.
3.3 Currently used methods to improve ravelling

Improving the durability of porous asphalt layers has been a hot issue in the Netherlands for many years. It seems that all the factors that contribute to the ravelling phenomenon are understood but somehow it is difficult to quantify them.

With the currently used DBFM (Design Build Finance Maintenance) contracts, contractors have to make sure that their maintenance costs stay low and that the traffic lanes are available. Poor availability of traffic lanes can cause contractors to pay severe fines.

As a result the contractors try to improve mixtures and study their performance by means of laboratory tests. Usually for porous asphalt the following tests are performed:

- **The Rotating Surface Abrasion Test (RSAT):**
  The aim of developing this test was to simulate the loading on an asphalt layer in the laboratory in such a way that after testing the same damage image is obtained which occurs at the end of the surface life in practice. Therefore the test plate is loaded intensively by a driving tire in such a way that besides normal forces, also shear forces are developed. This test is used to determine the durability/ravelling sensitivity of porous asphalt.

*Figure 25: Rotating Surface Abrasion Test setup [Voskuilen 2005]*
- The Indirect Tensile Test (ITT):
This test is used to determine the splitting strength, which is an indication of the tensile strength of the material, and can be performed at different temperatures. The test is used in the European standards to determine the water sensitivity of porous asphalt mixtures. Therefore tests are performed on dry sample and on conditioned samples. The ratio between these two gives the water sensitivity of the tensile strength of the material. This ratio is used as an indicator to study the effect of mixture modifications. In this way different mixtures can be compared with each other.

![Figure 26: Principle of the ITT test [Khedoe 2006]](image)

- The Semi Circular Bending test (SCB):
The semi-circular bending test is used to determine the fracture toughness of specimens. By performing tests on samples of different mixtures, insight can be obtained about the performance of each individual mixture. The fracture toughness is used as an indicator to study the effect of mixture modifications and like the ITT test different mixtures can be compared with each other.

![Figure 27: Setup of the SCB test](image)
- The Cantabro loss test:
The Cantabro test is used to determine the resistance of test samples to damage resulting from a combination of actions including abrasion and impact in a rotating steel drum. The failure type in this test is completely different than what occurs in reality. Therefore the Cantabro loss test is only suitable to compare different mixtures with each other.

![Cantabro Rotating Drum](image1)

*Figure 28: The Cantabro rotating drum [Voskuilen 2005]*

- The Aachener Ravelling Tester:
In this test the surface of a porous asphalt test slab is loaded by two oscillating tires in order to develop grain losses at the surface. The amount of loss is used as an indicator of the mixtures resistance against particle loss under traffic loading.

![Aachener Ravelling Tester](image2)

*Figure 29: The Aachener Ravelling Tester*
It’s obvious that all the discussed methods provide information about the effect of mixture receipt modifications such as:

- different stone types
- different bitumen types/modifications
- different fillers
- application of additives

However the questions still remains if these laboratory results can directly be translated into solutions for durable porous asphalt mixtures. At this point experience and human judgments (often subjective) play important roles in translating laboratory behavior into field behavior. Nevertheless contractors are still confronted with the risks they have to take at this point.

It is believed that studying porous asphalt on a macro scale, as done in these tests, will not provide sufficient insight into the ravelling phenomenon since:

- Ravelling occurs at the surface of the layer which is subjected to a complex state of stresses indicating: no pure tension, compression or shear but rather a combination.

- It is impossible to take into account all the factors that influence the material in practice during laboratory testing such as:
  - Non mechanical loadings
  - The effect of workmanship
  - Compaction effort

- Studying the ravelled material that piles up on the pavement shoulders indicates clearly that failure occurs within the mortar or at the interface between mortar and stone. Approaching ravelling on the scale of mortar and adhesive zones would provide a far more better fundamental understanding.
3.4 Evaluation of the literature review

The aim of the literature review was to obtain sufficient insight and provide solid answers for the second set of sub questions. It becomes clear that many factors combined together contribute to the performance of porous asphalt.

For many years all these factors were known, and the main problem seemed to be the poor insight into the effect of each factor. On the other hand one should realize that in practice a lot is going on which can not be completely understood at the moment.

Within LOT the following effects are not included:

- the effect of moisture
- the effect of frost-thaw cycles/ de-icing salts
- workmanship
- degree of compaction

However in contrast to this LOT was validated successfully by means of full scale pavement test [Huurman 2008].

This is an indication that LOT is capable to explain ravelling of porous asphalt.

With regard to winter damage the Lifetime Optimization Tool was able to give a general explanation for the aggressive ravelling in the winter.

The aim of this research is to validate this general conclusion by studying specific cases. Therefore the focus lies on using the LOT theory and not on combining LOT with other theories.

Based on the performed literature review and the conclusions drawn in chapter two, a research approach is adapted which is presented in the next chapter.
4. RESEARCH APPROACH

4.1 General

Research can have three purposes: exploratory, descriptive or explanatory. Research with the purpose to explore a topic is usually used to familiarize the researcher with a certain topic. This occurs when the researcher examines a new interest or when the subject of the study is relative new [Babbie 2007].

Exploratory studies are done for three purposes:

- to satisfy the researcher’s curiosity and desire for better understanding,
- to test the feasibility of undertaking a more extensive study,
- to develop methods to be employed in any subsequent study.

In descriptive research the researcher strives to describe a social phenomenon. According to Babbie, one observes the phenomenon and then describes what was observed.

The research presented in this report has the purpose to explain and is consequentially an explanatory research. An explanatory research such as this one provides: “reasons for phenomena, in terms of causal relationships.” [Babbie 2007].

The purpose of this research is to explain if brittle behavior is indeed the main cause for aggressive raveling in the winter. Research can have two approaches: deductive or inductive. In a deductive approach, research is used to test theories. In inductive research theories are developed from the analysis of research data.

This research draws hypotheses from a previous conducted research, which was performed in order to explain raveling in the winter by making use of a specific theory. This research can therefore be classified as deductive research. It does not try to develop a new theory but merely tries to test one theory and provide possible differences between the expected outcome (from the theory) and the research findings.

This research is aimed at validating an explanation that the LOT theory with regard to winter damage. This explanation was based on a general case and is already discussed in detail in chapter two.
In this research this explanation will be validated by considering different cases. This indicates that the first step is selecting the cases i.e. different sections of the primary road network.

The input that LOT requires (load, geometry and response) is directly obtained from these sections. This information is then used in simulations, which results in calculated stresses and strains throughout the whole porous asphalt mixture. The aim of this research is to validate the theory that damage occurs in the adhesive zones of the porous asphalt mixture in the winter. Therefore only the stresses and strains in the adhesive zones are of interest. After the results of the simulations are obtained, only the adhesive damage model is used to calculate damage development. At the end the calculated performance is compared with the field performance of the selected sections during the winter of 2009/2010. See figure 30.

This chapter discusses how the above mentioned procedure is executed, how the information is obtained, which considerations are taken into account and what assumptions are taken.
Chapter 4 Research approach

Figure 30: Outline of the research
4.2 The specific cases

It was found that the main reason for aggressive raveling was the reduced relaxation potential of the mortar. It was assumed that old sections which have been exposed to climatic factors for a longer time will show a higher stiffness value compared to relative young sections which have been exposed for a shorter period. Nevertheless many relative young sections showed severe raveling damage during the last winter while some old sections didn’t show significant raveling damage. If the explanation from LOT is followed, this would indicate that the relative young sections should have a stiffer mortar compared with the relative old sections. This is somehow confusing since one expects young ones to be more flexible and the old ones to be stiff. Therefore two types of sections are chosen:

- relative old sections that showed no significant raveling damage during the last winter
- relative young sections that showed severe raveling damage during the last winter

For each type 4 different sections of the Dutch primary road network were selected.

4.3 Geometry

For this research use will be made of the 2D idealized model for the following reasons:

- **Time constraint:**
  Using the 3D idealized model would require a lot of computation time
  Modeling each mixture separately by means of a photo/scan would require a tremendous amount of time. Modeling one single mixture using the photo/scan procedure takes approximately 2 weeks of time, this included knowledge transfer. Modeling all eight sections would require almost 4 months. This is considered not feasible for completion of this research in time.
• **Obtaining insight:**
  During the development of LOT it was already found that the 2D idealized model is capable enough to provide sufficient insight \cite{Huurman2008, Huurman2009, Mo2010}.

• **Practical reasons:**
  It is expected that in the future LOT will be used by contractors to improve their porous asphalt mixtures. Since contractors tend to be more practical then theoretical, usage of the 2D idealized model is considered to be the best option from a practical point of view. By proving with this research, that the 2D idealized model is capable enough of explaining winter damage, the interest of companies to implement LOT within their design methods, should increase.

### 4.4 Load

For this research three loading types are considered:

• **Traffic:** in this research a free rolling wheel travelling at a speed of 76.5 km/h will be simulated. The signals used to simulate this are already discussed in chapter two. In the study of Huurman 2009, a total of 10000 standard axle loads of 100 kN were assumed. Further on different traffic distribution were assumed during a 24h period. In this research the number of axle loads and the distribution during a 24 period follow from actual traffic measurements.

• **Pavement deflections:**
  Since Falling Weight Deflection (FWD) measurements were unavailable for the selected sections, the pavement deflections were calculated using WESLEA. In the previous study were LOT was used to explain winter damage, a representative structural design was used in WESLEA calculations. In this research the actual structural information of the selected sections is used in WESLEA. Further on in the previous study deflections were calculated up to 2 m and an exponential function was fitted in this data assuming that 15 m away the deflections are zero. In this research the deflections are calculated up to 15 m away from the centre.
• **Temperature fluctuations:**
The procedure how temperature stresses are included in damage calculations is already explained in chapter two. In the study of Huurman 2009, temperature stresses were distributed over a 24 h period assuming a sinusoidal movement. In this study the temperature stresses are distributed assuming the actual movement of temperature during a 24 h period. For this purpose use is made of climate data from the KNMI.

### 4.5 Material behaviour

As mentioned in chapter two it is only required to determine the stiffness of the mortar in LOT. The response of the adhesive zones is calculated out of the stiffness from the mortar. Unlike concrete for example, bituminous materials show visco-elastic behavior, meaning that when a constant stress or strain is applied on the sample, the stiffness is decreasing in time. See figure 31.

![Figure 31: Visco-elastic behaviour, left: applied constant strain; right: decreasing of stiffness in time [Jansen 2006]](image)

To describe this behavior many visco-elastic models are available such as Burgers, Maxwell, Prony, Boltzmann, Kelvin-Voight. In the finite element package which is used for simulations, ABAQUS, the Prony series is implemented. Therefore in LOT the mortar response is described by the Prony series.

The Prony series is derived from the generalized Maxwell model which consists of a spring and n Maxwell elements connected parallel. See figure 31.
Chapter 4 Research approach

Figure 32: Generalized Maxwell model

The response characteristics of the mortar need to be determined as a function of time and temperature. For short loading times (high frequencies) it is almost not possible to determine the visco-elastic properties using a relaxation test such as the DTT (Direct Tension Test). Therefore use was made of the DSR (Dynamic Shear Rheometer) machine which is fitted for this purpose.

In ABAQUS it’s not possible to export the DSR data directly one to one. Therefore the data is first transformed into Prony parameters.

Procedure:
The generalized Maxwell model describes the modulus in time domain with the function:

\[ E(t) = \sum_{i=1}^{n} E_i \exp[-t/\tau_i] \]  \hspace{1cm} (9)

By making use of Laplace transforms, this function can be transformed into a function describing the modulus within a frequency domain:
Chapter 4 Research approach

During the DSR test the stiffness, loss modulus and storage modulus are measured within a frequency domain.

By assuming a number of \( E_i \) and \( \tau_i \) values the stiffness, loss and storage modulus can be calculated using formulas 10, 11 and 12.

Using optimization programs, such as Excel or MATLAB, the error between the calculated and the measured stiffness, loss and storage modulus is minimized by optimizing the assumed \( E_i \) values and \( \tau_i \) values.

After the set of \( E_i \) and \( \tau_i \) values is determined, the time dependant stiffness of the material can be calculated using formula 9.

\[ E^*(\omega) = \sum_{i=1}^{n} \frac{E_i \omega \tau_i}{1 + i \omega \tau_i} \]  

(10)

\[ E'(\omega) = \sum_{i=1}^{n} \frac{E_i \omega^2 \tau_i^2}{1 + \omega^2 \tau_i^2} \]  

(11)

\[ E''(\omega) = \sum_{i=1}^{n} \frac{E_i \omega \tau_i}{1 + \omega^2 \tau_i^2} \]  

(12)

where:

\( E^*(\omega) = \) frequency dependant stiffness (MPa)

\( E'(\omega) = \) storage modulus (MPa)

\( E''(\omega) = \) loss modulus (MPa)

\( \tau = \) time constant

\( \omega = \) frequency (rad/s)

4.6 The simulations

After all the required information regarding the geometry is obtained, the 2D idealized mesh is generated using the LOT input generator. This input generator produces an input file. The input file is ran using ABAQUS 6.9.1. After the simulations the stresses at the points of interest can be exported to Excel and used for damage calculation.
4.7 Damage calculation

For this research only the stresses in the middle of the model at the surface will be used for calculation of damage. Only the stresses at the interface between stone and mortar, the adhesive zones, in contact points 1 and 4 are considered (see figure 34). The following considerations led to this decision:

- It was found that the most accurate results are obtained in the centre of the model. When the behavior of the adhesive zone is of interest, the equivalent tensile stress that follows from the normal stress and shear stress becomes of interest. It was found in previous simulations that the highest stresses occur in the points 1 and 4 [Mo 2010].
- This research focuses purely on winter damage. Preliminary results explaining winter damage, indicated that aggressive ravelling occurred due to failure of the adhesive zones. This was caused by the brittle behavior of the mortar. Since this research is aimed at validating these results, only the adhesive zones are of interest.
- When calculating the temperature stresses, it’s obvious that in longitudinal direction of the pavement, the highest normal stresses will be obtained in the horizontal contact points.

Figure 33: 2D idealized model, black arrow indicating the selected stone.
For this research only the adhesive zone damage model is of interest:

For $\sigma_{et} > 0$, $D = 0$ for $\sigma_{et} \leq 0$ with $\sigma_{et} = \sigma_{n} + \frac{\tau}{\tan \phi}$

\[ D = \left( \frac{\sigma_{et}}{\sigma_{0}} \right)^{n_{0}} \]

where:

- $D$ = rate of damage accumulation (-/s)
- $\sigma_{et}$ = equivalent tensile stress (MPa)
- $\sigma_{n}$ = adhesive zone normal stress (MPa)
- $\tau$ = adhesive zone shear stress (MPa)
- $\phi$ = friction angle (degr.)
- $n_{0}$ = model parameter (-)
- $\sigma_{0}$ = model parameter (MPa)

First of all, the model translates the combined action of normal and shear stresses which follow from the different simulations, into an equivalent tensile stress by using the Mohr-Coulomb failure criterion. The damage development of any stress signal is then determined by the sum of various damage increments:

\[ D(t) = \sum_{i=1}^{m} \left[ \frac{\sigma_{x}(t_{i+1}) + \sigma_{x}(t_{i})}{2} \cdot \Delta t \right] \]

where:

- $D(t)$ = the accumulated damage at time $t = \sum_{i=1}^{m} t_{i}$
- $\sigma_{x}(t_{i})$ = the equivalent tensile stress at time $t_{i}$ (MPa)
- $\sigma_{x}(t_{i+1})$ = the equivalent tensile stress at time $t_{i+1}$ (MPa)
- $\Delta t$ = the time increment (s)
This means that if the stresses of the adhesive zones during a 24 h period are known, the damage development in these zones in a 24 h period can be calculated. In this research each hour is divided in time steps of approximately 7 minutes. In these 7 minutes, the stresses that follow from temperature fluctuations, traffic and pavement deflections are summed used as input in the above damage model. Dependent on the distribution of temperature stresses and the distribution of traffic during this 24h period the damage development in each step varies. The total damage follows from accumulating the damage in each time step. Table 8 and 9 show the model parameters of the adhesive zone damage model, determined for two types of stone, Greywacke and Bestone. In this research the average values of both tables are used as model parameters for the adhesive zone damage model.

**Table 8: Damage model parameters for the adhesive zones, Bestone**

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>σ₀ (MPa)</th>
<th>n</th>
<th>θ (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-10</td>
<td>18.91</td>
<td>2.57</td>
<td>74.2</td>
</tr>
<tr>
<td>0</td>
<td>13.75</td>
<td>4.33</td>
<td>27.1</td>
</tr>
<tr>
<td>10</td>
<td>14.89</td>
<td>3.01</td>
<td>32.6</td>
</tr>
<tr>
<td>20</td>
<td>7.51</td>
<td>3.33</td>
<td>38.2</td>
</tr>
</tbody>
</table>

**Table 9: Damage model parameters for the adhesive zones, Greywacke**

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>σ₀ (MPa)</th>
<th>n</th>
<th>θ (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-10</td>
<td>22.34</td>
<td>2.9</td>
<td>66.2</td>
</tr>
<tr>
<td>0</td>
<td>11.93</td>
<td>5.36</td>
<td>24</td>
</tr>
<tr>
<td>10</td>
<td>8.18</td>
<td>3.83</td>
<td>32.4</td>
</tr>
<tr>
<td>20</td>
<td>9.98</td>
<td>2.77</td>
<td>39.6</td>
</tr>
</tbody>
</table>

This chapter discussed in detail the research boundaries, assumptions and how the research is executed. All the necessary input for LOT was determined by testing material retrieved from failed sections of the Dutch primary road network. The obtained input information is presented in the next chapter.
5. INPUT DETERMINATION

5.1 General

This research is aimed at validating the meso mechanical approach on its capability to explain excessive ravelling of porous asphalt in the winter. For this purpose use is made of the Lifetime Optimisation Tool. As explained in the previous chapter the 2D idealized model is used in this research. For generation of the 2D model information about the geometry was required. To determine the stresses in the adhesive zones, the response and load of these zones was also required. This chapter discusses the obtained information and how this information was determined. See figure 35.
Chapter 5 Input determination

Figure 35: Outline of chapter 5
5.2 Sections of the primary road network

As stated in the introduction of this report, eight sections of the Dutch primary road network were selected:

- four relative young pavements which showed severe ravelling damage during the winter of 2009/2010
- four relative old pavements which showed no significant ravelling damage during the winter of 2009/2010

Out of these sections several cores were drilled. These were cored in the heaviest loaded traffic except for the A4_1997 and the A12_1987, see table 10. First of all the material had to be representative indicating that it must have been subjected to traffic since it was observed that ravelling is more severe in the traffic lanes. Therefore it was decided to drill the cores from the traffic lanes. Secondly porous asphalt has a very open structure. This open structure has its advantages as mentioned in the introduction but on the other hand these voids can easily be clogged by dirt. Since the action of traffic causes some pumping effect it is assumed that in the wheel track of the traffic lane, the material would be less clogged compared to the other locations in the lane. Later in this report an explanation will be given why it was important to select cores which were less clogged.

**Table 10: Overview of the selected sections**

<table>
<thead>
<tr>
<th>Section</th>
<th>Date of construction</th>
<th>Location (km)</th>
<th>Lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>A15</td>
<td>2006</td>
<td>74.6 - 74.9</td>
<td>1HRR,3RR,RS</td>
</tr>
<tr>
<td>N9</td>
<td>2002</td>
<td>80.0 - 81.4</td>
<td>1RL,1RR</td>
</tr>
<tr>
<td>A200</td>
<td>2002</td>
<td>4.8 - 5.2</td>
<td>1HRR,2RR</td>
</tr>
<tr>
<td>N3</td>
<td>2004</td>
<td>1.3 - 1.5 / 3.8 - 4.5</td>
<td>1HRR,2RR,RS</td>
</tr>
<tr>
<td>A4</td>
<td>1997</td>
<td>51.65 - 52.1</td>
<td>HRR 1RR</td>
</tr>
<tr>
<td>A15</td>
<td>1995</td>
<td>117.8 - 118.5</td>
<td>1HRR,2RR,RS</td>
</tr>
<tr>
<td>A12</td>
<td>1992</td>
<td>118.2 - 119.4</td>
<td>HRL-2RL</td>
</tr>
<tr>
<td>A12</td>
<td>1987</td>
<td>31.27 - 30.58</td>
<td>1HR-L,1R-L</td>
</tr>
</tbody>
</table>
The terms in the outer right column from table 10 indicate which spot the cores were drilled.

In the Netherlands the reference point is Amsterdam. When driving towards Amsterdam the main road is marked as HRL, while driving away from Amsterdam is marked as HRR. In the Netherlands the main roads are separated by a traffic separator. Each main road is divided into 2 or three, in some case 4 traffic lanes. RR indicates the traffic lanes adjacent to the traffic separator where 1 stands for the first, 2 stands for the second or middle and 3 stands for the outer right traffic lane. The term RS indicates that the cores were drilled in the wheel track of the traffic lane. See figure 36.

---

**Figure 36: Explanation of the terms from table 7**
5.3 Load

In the previous chapter it was already mentioned that in this research three types of loading are considered:

- pavement deflections
- temperature fluctuations
- traffic

Hereafter the different loading types are discussed separately.

5.3.1 Pavement deflections

Pavement deflections were calculated using WESLEA. WESLEA is a multi-layer linear elastic analysis program which allows calculating the stresses, strains and displacements due to a wheel load, at different locations in a pavement structure. For this, the model requires thicknesses of the different layers and the Young’s modulus values of the different layers.

Note that by using these types of analysis program, the real behavior of the different materials is simplified since they don’t behave linear elastic in reality. Granular materials show stress dependent behavior and asphalt mixtures show visco-elastic behavior. In the winter temperatures are low and asphalt mixtures tend to behave more elastic.

However the assumption that these materials behave linear elastic is acceptable in case stresses and deflections in the structure remain low.

From the selected sections the structural design e.g.: layer thickness, and type of materials used, was gathered from Rijkswaterstaat (RWS). Missing information was gathered from the drilled cores. For this reason the cores were drilled at least until the base layer.

At some cores the base layer material was still attached. This proved to be very important since it indicates what kind of material was used below the asphalt layers.
Figure 37: Cores indicating the presence of a sand cement base layer [A15_1995]

Figure 38: Cores indicating the presence of a blast furnace slag base layer [N3_2004]

Figure 39: Cores indicating the presence of a sand layer below the asphalt package [A12_1987, A200_2002, and A15_2006]
Chapter 5 Input determination

Figure 40: Cores indicating the presence of a granular base layer [A4_1997]

The thickness of the asphalt layer was measured according to the procedure described in NEN-EN 12697-36.

Figure 41: Measurement of layer thickness

For practical reasons, the asphalt layers below the porous asphalt layer were considered to be one layer. The stiffness value for this layer was determined from the Dutch design method for asphalt pavements [RWS 1998]. To obtain the stiffness at -10 °C, the stiffness curve was extrapolated from 0°C to -10 °C. See figure 42.
Figure 42: Stiffness of dense asphalt concrete as a function of temperature

For calculating deflections of the porous asphalt layer using WESLEA it was required to assign a stiffness value for this layer.

Due to its structure, it is uncommon to perform bending or creep tests on porous asphalt. The mixture has an open structure held together by a relatively small amount of mortar. During testing the mortar fails extremely rapidly due to lack of lateral restraints.

This indicated that the stiffness of the porous asphalt layer had to be estimated as accurate as possible.

In pavement design the load spreading strongly depends on the bending stiffness of the different layers. From basic applied mechanics it is known that the bending stiffness is related to the product $E \cdot h^3$, where E is the elastic modulus and $h$ the layer thickness.

According to the Dutch design manual, an equivalent value of 0.8 is used for porous asphalt. This means that in structural design calculations, a porous asphalt layer of 50 mm equals a dense asphalt layer of 40 mm.

This led to the conclusion that the stiffness of the porous layer equals approximately half the stiffness of the dense layer.

Finally the thicknesses and stiffness values of the different layers below the asphalt package were assumed by making use of design manuals [RWS 1998, VNC 1996].
Table 11 gives an example of the input that is used in WESLEA. The input of the remaining sections can be found in the appendices.

### Table 11: Input used for WESLEA, A15_2006

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (mm)</th>
<th>Poisson’s ratio</th>
<th>Stiffness (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>-10 °C</td>
</tr>
<tr>
<td>PA</td>
<td>41</td>
<td>0.35</td>
<td>10475</td>
</tr>
<tr>
<td>STAC</td>
<td>333</td>
<td>0.35</td>
<td>20950</td>
</tr>
<tr>
<td>Sand sub base</td>
<td>1000</td>
<td>0.4</td>
<td>100</td>
</tr>
<tr>
<td>Subgrade (clay)</td>
<td>∞</td>
<td>0.4</td>
<td>55</td>
</tr>
</tbody>
</table>

Using the information from table 11, the deflections due to one wheel load were calculated up to 15 meter from the centre of the load. A wheel load from 50 kN was applied by a super single tire. From previous studies [Mohan 2007] it was concluded that the use of super single tires on the non driven axles of trucks is increasing in the Netherlands. In the following years it is expected that also the driven axles of trucks will be assembled with super single tires.

The tire used in this study is a Goodyear 425R65 super single.

The width is 330 mm and the assumed length of the wheel patch is 170 mm. This leads to an average contact stress of 0.891 MPa, see table 10.

Figure 43 gives an illustration of the deflections bowl calculated with WESLEA.

### Table 12: Used input for WESLEA

<table>
<thead>
<tr>
<th>Length wheel patch (mm)</th>
<th>170</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter wheel patch (mm)</td>
<td>330</td>
</tr>
<tr>
<td>Contact area (mm²)</td>
<td>56100</td>
</tr>
<tr>
<td>Wheel load (kN)</td>
<td>50</td>
</tr>
<tr>
<td>Tire pressure (MPa)</td>
<td>0.891</td>
</tr>
</tbody>
</table>
5.3.2 Temperature fluctuations

As explained in chapter two temperature fluctuations during a 24 h period are also considered in the simulations. For this purpose a small 2D idealized model was build were stones are modeled as physical bodies. See figure 44.

![Figure 44: Model used for calculation of temperature stresses](image)

The expansion or shrinkage caused by a difference in temperature was calculated using the thermal coefficients listed in table 10.

<table>
<thead>
<tr>
<th></th>
<th>Stone (Sandstone/Greywacke)</th>
<th>Mortar</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha_L$</td>
<td>$6.6 \times 10^{-6} ^\circ C$</td>
<td>$2.5 \times 10^{-5} ^\circ C$</td>
</tr>
</tbody>
</table>
This expansion was then fed to the model as prescribed movements at outer edges of the model. The simulated movement was derived from the actual temperature movements measured during the last winter. Figure 45 shows the temperature fluctuations measured during the winter of 2009/2010 in the Netherlands. For practical reasons data measured in the centre of the Netherlands was used.

According to figure 45 the lowest temperature was measured the 19th of December 2009. Figure 46 shows the temperature fluctuations of this day during a 24 h period.

For this research, simulation of temperature behavior during a 24h period had to represent the whole winter period. Therefore the temperature for each hour was assigned to be the average of 89 days (December, January, and February).
Figure 47 shows the average fluctuations during a 24 h period for the winter of 2009/2010.

![Figure 47: Measured and modelled temperature fluctuations](image)

For reasons of simplicity a function was fitted to this data. This would make the simulation process easier. See figure 48.

![Figure 48: Measured and modelled temperature fluctuations](image)
Finally this function was fed to the model and in this way the deformation of the model is in accordance with the actual temperature behavior. Meaning that when the temperature drops, the material shrinks and tensile stresses develop. As the temperature increases the material wants to expand and compressive stresses develop in the stone–stone contact region. In figure 45 it can be observed that the difference of temperature varies from day to day. In this research a value of 6°C is adapted as the temperature difference during a 24 h period.

Figure 49 gives an example of calculated temperature stresses at four different average daily temperatures assuming a temperature difference of 6°C.

![Figure 49: Calculated temperature stresses A12_1992, dt =6°C](image-url)
5.3.3 Traffic

The last type of loading considered in calculations was traffic. For this research traffic information was only available for two different locations. These measurements were carried on:

- RW012: Wageningen – Oosterbeek, 2008 in both directions
- RW015: Deil – Meteren, 2008 in both directions

Since the measurements were carried out in both directions, this led to 4 measured amounts:

- direction Wageningen
- direction Deil
- direction Oosterbeek
- direction Meteren

Table 14 gives the number of the measured trucks during a 24 h period on a working day. In these tables no distinction is made between the amounts of trucks per traffic lane. The bold numbers indicate the total number of trucks measured in a 24 h period on all traffic lanes.
Table 14: Measured trucks in 4 directions, during 24 hours

<table>
<thead>
<tr>
<th>Direction</th>
<th>Deil</th>
<th>Wageningen</th>
<th>Meteren</th>
<th>Oosterbeek</th>
</tr>
</thead>
<tbody>
<tr>
<td>0:00:00</td>
<td>1:00:00</td>
<td>46</td>
<td>68</td>
<td>49</td>
</tr>
<tr>
<td>1:00:00</td>
<td>2:00:00</td>
<td>58</td>
<td>59</td>
<td>41</td>
</tr>
<tr>
<td>2:00:00</td>
<td>3:00:00</td>
<td>43</td>
<td>59</td>
<td>40</td>
</tr>
<tr>
<td>3:00:00</td>
<td>4:00:00</td>
<td>61</td>
<td>80</td>
<td>53</td>
</tr>
<tr>
<td>4:00:00</td>
<td>5:00:00</td>
<td>145</td>
<td>174</td>
<td>90</td>
</tr>
<tr>
<td>5:00:00</td>
<td>6:00:00</td>
<td>390</td>
<td>435</td>
<td>221</td>
</tr>
<tr>
<td>6:00:00</td>
<td>7:00:00</td>
<td>579</td>
<td>602</td>
<td>422</td>
</tr>
<tr>
<td>7:00:00</td>
<td>8:00:00</td>
<td>505</td>
<td>425</td>
<td>433</td>
</tr>
<tr>
<td>8:00:00</td>
<td>9:00:00</td>
<td>440</td>
<td>383</td>
<td>377</td>
</tr>
<tr>
<td>9:00:00</td>
<td>10:00:00</td>
<td>513</td>
<td>441</td>
<td>446</td>
</tr>
<tr>
<td>10:00:00</td>
<td>11:00:00</td>
<td>556</td>
<td>434</td>
<td>487</td>
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<tr>
<td>11:00:00</td>
<td>12:00:00</td>
<td>554</td>
<td>426</td>
<td>509</td>
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<tr>
<td>12:00:00</td>
<td>13:00:00</td>
<td>531</td>
<td>422</td>
<td>525</td>
</tr>
<tr>
<td>13:00:00</td>
<td>14:00:00</td>
<td>520</td>
<td>421</td>
<td>539</td>
</tr>
<tr>
<td>14:00:00</td>
<td>15:00:00</td>
<td>520</td>
<td>428</td>
<td>608</td>
</tr>
<tr>
<td>15:00:00</td>
<td>16:00:00</td>
<td>504</td>
<td>414</td>
<td>639</td>
</tr>
<tr>
<td>16:00:00</td>
<td>17:00:00</td>
<td>466</td>
<td>376</td>
<td>584</td>
</tr>
<tr>
<td>17:00:00</td>
<td>18:00:00</td>
<td>361</td>
<td>294</td>
<td>483</td>
</tr>
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<td>18:00:00</td>
<td>19:00:00</td>
<td>298</td>
<td>238</td>
<td>416</td>
</tr>
<tr>
<td>19:00:00</td>
<td>20:00:00</td>
<td>238</td>
<td>190</td>
<td>313</td>
</tr>
<tr>
<td>20:00:00</td>
<td>21:00:00</td>
<td>163</td>
<td>151</td>
<td>209</td>
</tr>
<tr>
<td>21:00:00</td>
<td>22:00:00</td>
<td>120</td>
<td>123</td>
<td>150</td>
</tr>
<tr>
<td>22:00:00</td>
<td>23:00:00</td>
<td>86</td>
<td>100</td>
<td>112</td>
</tr>
<tr>
<td>23:00:00</td>
<td>0:00:00</td>
<td>66</td>
<td>80</td>
<td>72</td>
</tr>
<tr>
<td>0:00:00</td>
<td>0:00:00</td>
<td>7764</td>
<td>6826</td>
<td>7816</td>
</tr>
</tbody>
</table>

From the available information it was concluded that an average of approximately 7500 trucks are passing on these roads during a 24 h period.

In this research it assumed that 100 % of these trucks are driving in the right traffic lane. Further on an average of 3.5 axles per truck is assumed; this led to a total of 26299 axles.

All the simulations in this research consider a standard axle load of 100 kN.

The next step was to translate these 26299 occurring axle loads into standard 100 kN axle loads.

For this purpose use is made of table 15. Table 15 shows the axle load distribution that is used in the Netherlands as input for the thickness design of concrete pavements. This distribution is also suggested for the design of asphalt pavements [Molenaar 2006].
Table 15: Axle load frequency distribution (%) for two types of motorways [Houben 2006]

<table>
<thead>
<tr>
<th>Axle load group (kN)</th>
<th>Average wheel load (kN)</th>
<th>Heavily loaded motorway (%)</th>
<th>Normally loaded motorway (%)</th>
<th>Average (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20-40</td>
<td>15</td>
<td>20.16</td>
<td>14.8</td>
<td>17.5</td>
</tr>
<tr>
<td>40-60</td>
<td>25</td>
<td>30.56</td>
<td>29.5</td>
<td>30.1</td>
</tr>
<tr>
<td>60-80</td>
<td>35</td>
<td>26.06</td>
<td>30.2</td>
<td>28.1</td>
</tr>
<tr>
<td>80-100</td>
<td>45</td>
<td>12.54</td>
<td>13.5</td>
<td>13</td>
</tr>
<tr>
<td>100-120</td>
<td>55</td>
<td>6.51</td>
<td>7.91</td>
<td>7.21</td>
</tr>
<tr>
<td>120-140</td>
<td>65</td>
<td>2.71</td>
<td>3.31</td>
<td>3.01</td>
</tr>
<tr>
<td>140-160</td>
<td>75</td>
<td>1</td>
<td>0.59</td>
<td>0.8</td>
</tr>
<tr>
<td>160-180</td>
<td>85</td>
<td>0.31</td>
<td>0.09</td>
<td>0.2</td>
</tr>
<tr>
<td>180-200</td>
<td>95</td>
<td>0.12</td>
<td>0.01</td>
<td>0.07</td>
</tr>
<tr>
<td>200-220</td>
<td>105</td>
<td>0.03</td>
<td>0.01</td>
<td>0.02</td>
</tr>
</tbody>
</table>

First the occurring numbers of axles are distributed per axle load group, Ntot. The number of axles per group is then translated into number of equivalent 100kN axle loads, Neq.

This is also known as the load equivalency concept. This concept makes it possible to calculate the damaging effect of a particular axle load relative to a standard axle load. The number of equivalent axle load repetitions can be calculated with:

\[
N_{eq} = \left(\frac{P}{P_{st}}\right)^m \cdot N_{tot,P}
\]  

(15)

where:

- \(N_{eq}\) = number of equivalent passages of the wheel load considered
- \(P\) = wheel load to be considered (kN)
- \(P_{st}\) = reference wheel load (kN)
- \(N_{tot,P}\) = number of repetitions of the considered wheel load
- \(m\) = damage factor

The value of \(m\) depends on which type of damage is considered [Molenaar 2006]:

- damaging effect of various axle loads relative to each other in terms of fatigue of the asphalt layer, then: 3<\(m<6\)
- effect on fatigue in a cement treated layer, then: 7<\(m<10\)
- effect on the loss of serviceability, then: \(m=4\)
Since excessive ravelling of porous asphalt in the winter leads to a reduced availability of the road network and a decrease of the noise reduction effect, this can be translated into a type of loss of serviceability. Therefore a value of 4 is assumed for \( m \).

Assuming a standard axle load of 100 kN and \( m = 4 \), led to approximately 11450 equivalent 100 kN axle loads. See table 16.

**Table 16: Calculation of equivalent axle loads**

<table>
<thead>
<tr>
<th>Wheel load (kN)</th>
<th>Freq.distr. (%)</th>
<th>Ntot.</th>
<th>Neq.</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>17.5</td>
<td>4602</td>
<td>37</td>
</tr>
<tr>
<td>25</td>
<td>30.1</td>
<td>7916</td>
<td>495</td>
</tr>
<tr>
<td>35</td>
<td>28.1</td>
<td>7390</td>
<td>1774</td>
</tr>
<tr>
<td>45</td>
<td>13.1</td>
<td>3445</td>
<td>2260</td>
</tr>
<tr>
<td>55</td>
<td>7.21</td>
<td>1896</td>
<td>2776</td>
</tr>
<tr>
<td>65</td>
<td>3.01</td>
<td>792</td>
<td>2261</td>
</tr>
<tr>
<td>75</td>
<td>0.8</td>
<td>210</td>
<td>1065</td>
</tr>
<tr>
<td>85</td>
<td>0.2</td>
<td>53</td>
<td>439</td>
</tr>
<tr>
<td>95</td>
<td>0.07</td>
<td>18</td>
<td>240</td>
</tr>
<tr>
<td>105</td>
<td>0.02</td>
<td>5</td>
<td>102</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td><strong>11450</strong></td>
<td></td>
</tr>
</tbody>
</table>

The last step was to distribute this 11450 equivalent 100 kN axle loads over a 24 h period.

Therefore use is made from traffic density distributions which were calculated during the earlier mentioned traffic measurements.

Figure 50 gives the average of the four measured traffic distributions during a 24 h period.

**Figure 50: Traffic distribution during a 24h period.**
5.4 Geometry

As mentioned in chapter 4, the simulations were carried out using the 2D idealized model. In order to model a porous asphalt mixture using the 2D idealized model, the following information is required:

- Average stone diameter of the mixture
- Density of the stone fraction
- % stone
- Density of the mineral in the mortar
- Density of the bitumen
- % bitumen on 100 % aggregate
- % voids

Since no mixture receipts of the different sections were available it became obvious that all the drilled cores had to provide this information in some way.

As shown in table 10 majority of the cores were drilled in the right traffic lane in the outer wheel track. This was done to obtain cores that were less clogged since traffic has some cleaning effect.

Since porous asphalt is an open mixture one can imagine that this mixture is vulnerable to clogging with dirt. This dirt contains organic material, fine sand, rubber etc.

When determining the volumetric composition of the mixture it is necessary that it represents the mixture as it was during the construction phase. Therefore it was necessary to get the cores clean first. By not cleaning the cores, this would affect the maximum density and this would finally result in non representative void contents.

Therefore the cores were first cleaned first and after that the composition was determined by means of extraction.
Chapter 5 Input determination

Cleaning of the cores

The main reasons for cleaning the cores were:

- **Calculation of voids content**: for porous asphalt the void content is determined with the formula:

\[
\text{Airvoids} (%) = \frac{\rho_{\text{max}} - \rho_{\text{bulk}}}{\rho_{\text{max}}}
\]

(16)

*where*:
\[
\rho_{\text{max}} = \text{maximum density (density of the mix) (kg/m}^3\text{)}
\]
\[
\rho_{\text{bulk}} = \text{bulk density (density of the sample) (kg/m}^3\text{)}
\]

The bulk density is calculated using the formula:

\[
\rho_{\text{bulk}} = \frac{M_{\text{sample}}}{V_{\text{sample}}}
\]

(17)

*where*:
\[
M_{\text{sample}} = \text{mass of the porous asphalt core (kg)}
\]
\[
V_{\text{sample}} = \text{volume of the porous asphalt core (m}^3\text{)}
\]

The volume of the core is simply calculated by multiplying the height with the area. This is done according NEN –EN 12697-5.

The mass is measured by weighing the sample. In case of clogged cores, this would lead to a higher value for the bulk density. According to formula 16 this would lead to a lower void content. This would effect the mesh generation and finally the results.

- **Response of the mortar**: for this research the mortar response was determined from the material selected directly from the core. In case of dirty cores, this could result in a material containing all kinds off dirt i.e.: organic material, rubber, pieces of glass etc. This would influence the response measurements of the material and therefore it was decided to clean the cores first.
Before cleaning the cores, a CT scan was performed in order to see the effect of cleaning. See figure 51. The red arrows indicate the presence of dirt.

![Figure 51: The porous asphalt core before cleaning (right) and CT scan of the core (left)](image)

The first attempt to clean the cores was done by using an ultrasonic bath. See figure 52.

![Figure 52: Ultrasonic bath setup](image)

First the core was soaked in water for 12 hours. After being placed for approximately one hour in the ultrasonic bath, no loosening of the dirt was visible. It became clear that this particular ultrasonic bath was not sufficient enough to clean these extremely clogged cores. Therefore this procedure was aborted.

The second approach was by using water pressure. For this purpose a rubber was used, clamped around the core with a water hose to be mounted on a standard water pipe. See figure 53.
The core was then placed on a set of sieves in order to collect all the removed dirt. The pipe was opened maximum and in this way a pressure is created on top of the core.

After every 15 minutes the retained dirt on the sieves was selected, and then the core was placed under water pressure again. Finally when no retained dirt was visible on the set of sieves, it was assumed that the core was clean.

After this the core was sent for a second CT scan.

The second CT scan showed that the top and bottom part of the core was clean. However in the centre of the core, the pores were still filled with dirt. See figure 53.

This method was promising so the core was cleaned again using the same procedure. This time the core was washed from both sides. After one hour the core was turned upside down. And this procedure was repeated until no dirt was visible anymore on the different sieves. Then it was assumed that the core was clean.

After this step the cores were scanned again using the CT scanner.

The final scan showed that there was still some dirt inside the core. See figure 54, red arrows indicate the presence of dirt.
So finally it was decided that even this procedure was not effective enough and another solution had to be found.

![CT scan of the core after cleaning](image)

*Figure 54: CT scan of the core after cleaning (red arrows indicate the presence of dirt)*

It was clear that in order to get inside the centre of the core; the core had to be crumbled in such a way that the core was not damaged. Therefore it was decided to heat up the core and then press it using a manual actuator press.

First the cores are placed in an oven at 85 degrees for 30 minutes. After this the core is pressed using a manual hydraulic pressure bench. To avoid crushing, rubber plates are placed on top and bottom of the core. See figure 55. After this the cores is pulverized further by hand.
After the cores were completely crumbled the dirt had to be separated from the mixture. This raised the following question: what is considered as dirt?

It was assumed that the material which was selected during washing is material that is present in the voids in the form of dirt. By sieving this material more insight would be obtained about the particle size of this fraction. Therefore the dirt that was retained on the different sieves while cleaning using water was sieved using the following set of sieves:

2 mm – 1mm – 0.5mm - 125µm – 63 µm
The sieve analysis showed that almost 100% of the dirt was smaller than 2 mm while approximately between 80 and 90% of the dirt was smaller than 1 mm. It was decided arbitrary to use the 1 mm sieve to separate dirt from the material. This indicated that after crumbling the cores, all the material smaller than 1 mm is considered as dirt.

On the basis of this decision table 17 gives an overview of the calculated amount of dirt for various sections.
Table 17: Calculated percentage of dirt for cores from different sections

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Before (gr.)</td>
<td>1245.7</td>
<td>1464.1</td>
<td>1980.9</td>
<td>1480</td>
<td>1298.9</td>
<td>1331.8</td>
</tr>
<tr>
<td>After (gr.)</td>
<td>1200.7</td>
<td>1440.7</td>
<td>1820.3</td>
<td>1420.3</td>
<td>1248.6</td>
<td>1288.4</td>
</tr>
<tr>
<td>Dirt (gr.)</td>
<td>45</td>
<td>23.4</td>
<td>160.6</td>
<td>59.7</td>
<td>50.3</td>
<td>43.4</td>
</tr>
<tr>
<td>Dirt (%)</td>
<td>3.6</td>
<td>1.6</td>
<td>8.1</td>
<td>4.0</td>
<td>3.9</td>
<td>3.3</td>
</tr>
</tbody>
</table>

One of the important geometrical input parameters is the percentage of voids. In order to calculate the voids content, first the maximum density of the mixture has to be known (formula 16).

The maximum density of the mixture is determined according NEN-EN 12697-5. In the Netherlands the use of water is the official procedure to determine the maximum density. Nevertheless use was made of methyleenchloride because after determining the maximum density, the next step was determining the composition of the mix by means of extraction. By using methyleenchloride the mixture would already be soaked in the solvent, which would make the extraction process easier.

The extraction process was done using a rotating evaporator set up. For a detailed description of the procedure, reference is made to NEN-EN 12697-3.

![Figure 58: Rotating evaporator setup used for extraction of the bitumen](image)

After extraction the mixture is divided into bitumen, sand, stone and filler. LOT requires the density of the stone (fraction >2mm) and density of the mineral in the mortar.

According to previous research [Muraya 2007] the aggregate skeleton of porous asphalt mixtures only consist of aggregates larger then 0.5 mm.
Chapter 5 Input determination

This means that the mortar in porous asphalt mixtures can be defined as a mixture of
bitumen and aggregates smaller then 0.5 mm.
Determining the density of such a small fraction would require a tremendous amount
of time and effort.
For practical reasons the mineral in mortar is considered to be the fraction smaller
then 2mm in this research. This would make it far easier to determine the density.
Table 18 shows the composition of the different sections obtained after the extraction
procedure while table 19 shows the recommended grading for a standard porous
asphalt mixture according to the Dutch standards.

Table 18: Grading of the different mixtures

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>C 16.0</td>
<td>0.53</td>
<td>2.92</td>
<td>2.08</td>
<td>0.47</td>
<td>1.36</td>
<td>0.57</td>
<td>2.6</td>
<td>0.93</td>
</tr>
<tr>
<td>C 11.2</td>
<td>23.11</td>
<td>21.04</td>
<td>33.71</td>
<td>19.56</td>
<td>23.61</td>
<td>25.91</td>
<td>25.38</td>
<td>19.66</td>
</tr>
<tr>
<td>C 8.0</td>
<td>57.34</td>
<td>52.72</td>
<td>69.24</td>
<td>56.36</td>
<td>56.53</td>
<td>55.42</td>
<td>48.83</td>
<td>56.43</td>
</tr>
<tr>
<td>C 4</td>
<td>75.97</td>
<td>73.1</td>
<td>86.89</td>
<td>84.87</td>
<td>77.31</td>
<td>75.86</td>
<td>72.04</td>
<td>76.28</td>
</tr>
<tr>
<td>2 mm</td>
<td>81.04</td>
<td>80.18</td>
<td>88.73</td>
<td>87.45</td>
<td>82.92</td>
<td>82.09</td>
<td>81.47</td>
<td>83.48</td>
</tr>
<tr>
<td>1 mm</td>
<td>84.26</td>
<td>84.32</td>
<td>90.09</td>
<td>89.62</td>
<td>85.83</td>
<td>84.7</td>
<td>84.12</td>
<td>86.69</td>
</tr>
<tr>
<td>0.500 mm</td>
<td>87.56</td>
<td>87.32</td>
<td>91.14</td>
<td>91.53</td>
<td>89.17</td>
<td>88.27</td>
<td>86.32</td>
<td>88.89</td>
</tr>
<tr>
<td>0.250 mm</td>
<td>89.97</td>
<td>89.33</td>
<td>93.69</td>
<td>93.39</td>
<td>91.17</td>
<td>90.59</td>
<td>88.59</td>
<td>90.5</td>
</tr>
<tr>
<td>0.063 mm</td>
<td>94.13</td>
<td>93.14</td>
<td>95.65</td>
<td>95.57</td>
<td>94.84</td>
<td>94.23</td>
<td>93.84</td>
<td>94.42</td>
</tr>
<tr>
<td>% bitumen on</td>
<td>4.51</td>
<td>3.29</td>
<td>3.03</td>
<td>3.6</td>
<td>4.34</td>
<td>3.84</td>
<td>3.6</td>
<td>3.72</td>
</tr>
</tbody>
</table>

Table 19: Recommended grading for PA 0/16 mixture according to the Dutch
standards

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>C 16.0</td>
<td>0</td>
<td>5</td>
<td>±2</td>
<td>0</td>
</tr>
<tr>
<td>C 11.2</td>
<td>15</td>
<td>30</td>
<td>±8</td>
<td>15</td>
</tr>
<tr>
<td>C 8.0</td>
<td>50</td>
<td>65</td>
<td>±7</td>
<td>50</td>
</tr>
<tr>
<td>C 4</td>
<td>65</td>
<td>±4</td>
<td>85</td>
<td>85</td>
</tr>
<tr>
<td>2 mm</td>
<td>85</td>
<td>±4</td>
<td>85</td>
<td>85</td>
</tr>
<tr>
<td>1 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.500 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.250 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.180 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.063 mm</td>
<td>95.5</td>
<td>±1</td>
<td>95.5</td>
<td>95.5</td>
</tr>
<tr>
<td>% bitumen on</td>
<td>4.5</td>
<td>±0.5</td>
<td>4.5</td>
<td>4.5</td>
</tr>
</tbody>
</table>

When the values from table 18 are compared with the values from table 19 it can be
concluded that the overall grading of the mixtures fits reasonably between the
recommended values.
In some cases the requirements are not met however the differences are not that significant. However the major difference that can be noticed directly is the relative low amount of bitumen content for all the cases except the A15_2006 and the A4_1997.

At first instant it was believed that during cleaning of the cores some of bitumen was being washed away.

Therefore one mixture (A12_1992) was extracted completely without cleaning the material, to find out whether the cleaning procedure was the cause for this significant small amount of bitumen percentage.

![Sieve analysis](image)

**Figure 59: Comparison of grading curves, A12_1992**

**Table 20: Obtained bitumen percentage after extraction A12_1992**

<table>
<thead>
<tr>
<th>Material</th>
<th>Bitumen content in (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cleaned</td>
<td>3.46</td>
</tr>
<tr>
<td>Dirty</td>
<td>3.73</td>
</tr>
</tbody>
</table>

Next to this, the dirt that was selected during the trial cleaning procedures was sent for an extraction to find out whether there was some bitumen present in the dirt. The results of this extraction are shown in table 21

**Table 21: Results obtained after extraction of dirt**

<table>
<thead>
<tr>
<th>Material</th>
<th>Mass (gr)</th>
<th>Percentage (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Filler, material &lt; 63 µm</td>
<td>4.2</td>
<td>11.4</td>
</tr>
<tr>
<td>Material &gt; 63 µm</td>
<td>30.6</td>
<td>82.9</td>
</tr>
<tr>
<td>Bitumen</td>
<td>2.1</td>
<td>5.7</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>36.9</strong></td>
<td><strong>100</strong></td>
</tr>
</tbody>
</table>
Based on the obtained result presented in table 18, figure 59, table 20 and table 21, the following conclusions were drawn:

- Cleaning the cores and considering all the material less then 1 mm as dirt does not cause significant changes in the grading of the mixture

- The extraction results in table 18 indicate that the separated dirt contains some bitumen and some fine material. However for both the amount is relative very low.

- The presence of bitumen in the dirt indicates that during the service life of the porous asphalt mixture, some kind of erosion process takes place which results in reduced amounts of bitumen in the mixture. This seems to be in accordance with the mortar erosion process that Kringos describes in her research. However the amount of recovered bitumen from the dirt was very small.

- Considering the conclusion above it is strongly believed that the relative poor amount of bitumen of the different cases is directly related to poor construction efforts.

The values presented in table 15 were used to calculate the average stone diameter.

The following formula was used:

$$D = \frac{\sum_{i=1}^{n} D_i \cdot fr_i}{\sum_{i=1}^{n} fr_i}$$

$$R = \frac{D}{2}$$

*Where:* 

- $D$ = equivalent grain diameter (mm) 
- $R$ = equivalent grain radius, i.e radius of the modelled stone particles (mm) 
- $n$ = number of fractions in the stone particle grading, i.e $D > 2$ mm (-) 
- $i$ = fraction counter (-) 
- $D_i$ = diameter of stone particles in the $i_{th}$ fraction (mm) 
- $fr_i$ = size of the $i_{th}$ fraction (m/m)
For the bitumen density a value of 1030 kg/m³ was adapted for all the cases. For porous asphalt mixtures the main type of bitumen used is a pen 70/100 bitumen which has a density that lies between 1025 and 1030 kg/m³. Further on most of the bitumen types used in road engineering have density that generally lies between 1020 and 1040 kg/m³. Based on this information it was assumed that adapting a value of 1030 kg/m³ is a quite fair decision. Table 22 shows the final obtained information that was used to generate the 2D idealized model of the different sections.

\begin{table}
\centering
\begin{tabular}{|c|c|c|c|c|c|c|c|c|}
\hline
\hline
Average stone diameter (mm) & 8.24 & 7.94 & 8.53 & 7.49 & 8.17 & 8.26 & 7.78 & 7.95 \\
\hline
Bitumen density (kg/m³) & 1030 & 1030 & 1030 & 1030 & 1030 & 1030 & 1030 & 1030 \\
\hline
Density mineral in mortar (kg/m³) & 2700 & 2633 & 2718 & 2587 & 2612 & 2634 & 2640 & 2617 \\
\hline
Density stone (kg/m³) & 2743 & 2728 & 2723 & 2732 & 2659 & 2656 & 2705 & 2644 \\
\hline
% stone & 84.3 & 84.3 & 88.7 & 87.5 & 83.7 & 84.7 & 84.1 & 86.7 \\
\hline
% bitumen on 100% aggregate & 4.51 & 3.29 & 3.03 & 3.6 & 4.34 & 3.84 & 3.6 & 3.72 \\
\hline
% air voids & 20.0 & 20.8 & 21.8 & 21.8 & 23.1 & 20.6 & 19.1 & 21.3 \\
\hline
\end{tabular}
\caption{Obtained geometrical input for LOT}
\end{table}
5.5 Response

5.5.1 General

After the required information about the load and geometry was obtained, the last step was to determine stiffness properties of the cohesive and adhesive zones. It was already mentioned in chapter two that for LOT simulations only the stiffness of the mortar has to be determined. The stiffness off the adhesive zones is calculated out of the stiffness from the mortar.

The procedure is already discussed in chapter two. This paragraph discussed how the samples were prepared, the DSR test results and the final obtained stiffness of the cohesive zones.

5.5.2 Preparation of the mortar samples

For mortar testing using the DSR test setup, mortar columns had to be prepared. This was done by heating up the cleaned porous asphalt mixture to 175 °C for approximately one hour.

In paragraph 5.4 it was already mentioned why the cores had to be cleaned. If not, this would affect the maximum density and finally the void content.

A second reason why the cores had to be cleaned is because the mortar was retrieved from the mix by peeling it off from the aggregate particles using spatulas.

In case of dirt cores one can imagine that the dirt could get mixed up in the mortar and this could affect the response behavior.

The aim was to obtain the mortar as it is now and as how it was during the construction phase with respect to composition. This means that no dirt was allowed.

By stirring with the spatula in the mixture while it’s hot, material sticks on to it. This material is then selected and placed on a sieve in the oven at 175 °C.

During the development of LOT, mortar response tests were done on material containing grains smaller then 0.5 mm.

At first instance the selected material was placed on a 0.5 mm sieve.
Chapter 5 Input determination

Trial attempts to select material with grain smaller than 0.5 mm showed that this was very time consuming, and the material had to be exposed for a long time to high temperatures. This could cause some ageing effects and would result in a not fully representative mortar sample.

For practical reasons it was decided to use a 1 mm sieve.

Therefore in this study mortar is defined as a mixture of: bitumen, filler and sand smaller than 1mm.

![Figure 60: Procedure of sample preparation](image)

All the material that is passing the sieve is selected then placed in a special developed mold.

Note that the mold is preheated in the oven at 175 °C. This is done to exclude unwanted cooling down of the mortar. Since the mortar is very sticky, it is heated up extra on a hot plate for approximately 5 minutes to make it easy to fill into the mold.

Hereafter the mold with mortar is placed in the oven for 5 minutes at 175 °C to take away enclosed air bubbles. This also causes some setting of the material. So extra mortar is placed on top and the mold is placed again for five minutes at 175 °C. Finally the mold is taken out and left to cool down at room temperature. After this the mold is dismantled and the samples are inspected. Good samples should show a smooth surface, while bad samples show some air captivities.
5.5.3 The DSR Test

Frequency sweep tests were done using the DSR (Dynamic Shear Rheometer) machine. An oscillating stress is applied to sample and the rotational deformation of the sample is measured and translated into the shear modulus.

![DSR test setup](image)

*Figure 61: DSR test setup*

Use is made from the following formulas:

\[
\tau = \frac{2 \cdot T}{\pi \cdot r^2}
\]

(20)

\[
\gamma = \frac{r \cdot \theta}{h_{\text{eff}}}
\]

(21)

*where:*

\(\tau\) = shear stress (MPa)

\(T\) = applied torque (N.mm)

\(r\) = specimen radius (3 mm)

\(\gamma\) = shear strain

\(\theta\) = measured deflection angle (rad)

\(h_{\text{eff}}\) = specimen effective height (12.742 mm)

After the shear stress and shear strain are calculated, the complex shear modulus and phase angle are calculated using the formulas 22 and 23.
Chapter 5 Input determination

\[ G^* = \frac{\tau}{\gamma} = |G^*|e^{i\delta} = G' + G'' \quad (22) \]

\[ \delta = \arctan\left(\frac{G'}{G''}\right) \quad (23) \]

where:

- \( G^* \) = complex shear modulus (MPa)
- \( G' \) = real part of the complex shear modulus, elastic component (MPa)
- \( G'' \) = imaginary part of the complex shear modulus, visco component (MPa)
- \( \delta \) = phase angle, shift between stress and strain during oscillation (degr.)

All the above parameters are calculated automatically during testing by the machine’s software. Figure 62 gives an example of measured data during testing.

![Raw DSR data](image)

**Figure 62: Example of DSR test results, A12_1992**

From the results of the previous conducted study were LOT was used to explain winter damage, it was already concluded that especially the strain controlled loading types caused an increase in damage in the winter (see chapter two). Two types of strain controlled loadings were distinguished:

- temperature fluctuations
- pavement deflections

It is known that bituminous materials show elastic behavior at short loading times while at long loading times they show viscous behavior.
This behavior depends on the temperature. As a combination of ageing and low temperatures in the winter, the relaxation potential of the material has decreased and the loading types that occur at relative low frequencies (long loading times) will cause stresses to act for a longer period on the material. When this highly stressed material is loaded by a passing tire (relative high frequency) it can be concluded that damage develops rapidly.

This indicates that it is important that the material behavior is measured over such a frequency range which covers loading types of long time and short time.

The simulations performed in this research consider three loading types:
- Traffic, which occurs at a frequency of approximately 40 – 80 Hz
- Pavement deflections: simulation of the deflection bowl takes approximately 1.4 seconds of time. This leads to a frequency around 1 Hz.
- Temperature fluctuations: simulation of temperature movements consider a 24 period. This leads to a frequency of approximately 1.1 E-05 Hz.

Note that the mentioned frequencies are not absolute values but more or less an accurate estimation.

In order to obtain complete insight into the behavior of the mixture the DSR test would have to be performed over a frequency range from 1E-05 Hz up to 100 Hz.

Since it’s not economical to perform these tests over such a wide range of frequencies, the visco-elastic properties can be obtained for a much larger frequency domain using the time-temperature superposition principle.

The time-temperature superposition principle simply states that the effects of loading frequency and temperature are interchangeable. This means that the same material properties can be obtained either at low temperatures and low frequencies (long loading times) or at high temperatures and high frequencies (short loading times).

This implies that the response measured at a certain frequency, f, and temperature, T, can be shifted to a reduced frequency, red.freq, and a chosen reference temperature, Tₜ, for this purpose shift factors that control the shift in frequency as a function of temperature are required.
For this research the Williams-Landel- Ferry (WLF) equations are used to determine the shift factors:

\[
\log a_i = \frac{-C_1(T - T_i)}{(C_2 + T - T_i)}
\]

where:
- \(a_i\) = the shift factor relative to the reference temperature \(T_s\)
- \(C_1, C_2\) = empirically determined constants,
- \(T = \) the selected temperature (°K or °C)
- \(T_s = \) the reference temperature (°K or °C)

First the master curve is formed visually by assuming different values for \(C_1\) and \(C_2\). After this a model is fitted into this shifted curve and the error between model and visually shifted curve is minimized by optimization of the assumed \(C_1\) and \(C_2\) values. Hereafter the model fully represents the measured data and mastercurves can be constructed for any of the test temperatures.

At this point full insight can be obtained of the material at different loading times and temperatures.

The model used for constructing the mastercurves is the Christensen-Anderson model [Christensen et al 1992]:

\[
G^*(\omega) = G_s[1 + \left(\frac{\omega}{\omega_c}\right)^{\frac{\log^2 R}{\log^2}}]^{\frac{R}{\log^2}}
\]

\[
\delta(\omega) = 90\left\{1 + \left(\frac{\omega}{\omega_c}\right)^{\frac{\log^2 R}{\log^2}}\right\}
\]

where:
- \(G^*(\omega)\) = complex dynamic modulus (Pa)
- \(G_s\) = glass modulus (Pa)
- \(\omega_c\) = crossover frequency (frequency where the phase angles equals 45°), (rad / s)
- \(R\) = rheological index
- \(\delta\) = phase angle (degr.)

To ensure that the material behavior at 1.1 E-05 Hz would be measured, frequency sweep tests were performed for 7 temperatures: -10, 0, 10, 20, 30, 40 and 50 °C with a frequency range of 0.1 to 400 rad/s. See figure 63.
During testing it was observed that 50 °C was the highest temperature at which the samples could be tested. At higher temperatures the material started to flow downwards. See figure 63.

Figure 63: Sample flowing downwards at 60 °C

Figure 64 illustrates the principle of the shifting procedure and the important frequency ranges.

Figure 64: Example of shifting the raw data to construct the master curve, A12_1992_Tref=20°C

It can be seen in figure 64 that after shifting the material properties are obtained over a wide frequency range covering the three important frequency ranges.
Chapter 5 Input determination

After testing the mortar of all the different sections, and applying the described procedures and formulas, the following results were obtained:

**Table 23: Obtained model parameters**

<table>
<thead>
<tr>
<th>Section</th>
<th>$C_1$</th>
<th>$C_2$</th>
<th>$R$</th>
<th>$G_0$ (Pa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A15_2006</td>
<td>36.37</td>
<td>264.51</td>
<td>1.76</td>
<td>3.50E+09</td>
</tr>
<tr>
<td>N3_2004</td>
<td>35.41</td>
<td>251.01</td>
<td>2.32</td>
<td>4.80E+09</td>
</tr>
<tr>
<td>N9_2002</td>
<td>30.00</td>
<td>215.00</td>
<td>1.54</td>
<td>3.50E+09</td>
</tr>
<tr>
<td>A200_2002</td>
<td>34.32</td>
<td>240.07</td>
<td>1.53</td>
<td>4.00E+09</td>
</tr>
<tr>
<td>A4_1997</td>
<td>28.90</td>
<td>213.16</td>
<td>1.68</td>
<td>3.50E+09</td>
</tr>
<tr>
<td>A15_1995</td>
<td>35.30</td>
<td>257.70</td>
<td>1.80</td>
<td>4.20E+09</td>
</tr>
<tr>
<td>A12_1992</td>
<td>42.27</td>
<td>307.88</td>
<td>2.00</td>
<td>5.00E+09</td>
</tr>
<tr>
<td>A12_1987</td>
<td>31.33</td>
<td>226.61</td>
<td>1.57</td>
<td>3.70E+09</td>
</tr>
</tbody>
</table>

**Table 24: Calculated shift factors ($T_{reference} = 10^\circ C$)**

<table>
<thead>
<tr>
<th>Section</th>
<th>$10^\circ C$</th>
<th>$0^\circ C$</th>
<th>$10^\circ C$</th>
<th>$20^\circ C$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A15_2006</td>
<td>944.64</td>
<td>26.86</td>
<td>1.00</td>
<td>4.73E-02</td>
</tr>
<tr>
<td>N3_2004</td>
<td>1163.31</td>
<td>29.46</td>
<td>1.00</td>
<td>4.40E-02</td>
</tr>
<tr>
<td>N9_2002</td>
<td>1193.78</td>
<td>29.07</td>
<td>1.00</td>
<td>4.64E-02</td>
</tr>
<tr>
<td>A200_2002</td>
<td>1315.65</td>
<td>31.03</td>
<td>1.00</td>
<td>4.24E-02</td>
</tr>
<tr>
<td>A4_1997</td>
<td>981.80</td>
<td>26.45</td>
<td>1.00</td>
<td>5.07E-02</td>
</tr>
<tr>
<td>A15_1995</td>
<td>934.22</td>
<td>26.62</td>
<td>1.00</td>
<td>4.80E-02</td>
</tr>
<tr>
<td>A12_1992</td>
<td>864.68</td>
<td>26.25</td>
<td>1.00</td>
<td>4.68E-02</td>
</tr>
<tr>
<td>A12_1987</td>
<td>1077.46</td>
<td>27.94</td>
<td>1.00</td>
<td>4.74E-02</td>
</tr>
</tbody>
</table>

**Figure 65: Constructed mastercurves of the different sections @ -10^\circ C**
Figure 65 shows the constructed mastercurves for the eight selected sections at a reference temperature of -10 °C. It can be seen that the A12_1992 has the highest stiffness followed by the N3_2004 which is a relative young section. The mastercurves of the other sections are relative close to each other. The ellipse in figure 65 is highlighted in figure 66.

![Mastercurve @ -10 °C](image)

**Figure 66: Mastercurves in the low frequency range**

Out of the mortar which was selected out of the different cores, several samples were prepared for each section. For two sections the DSR test was repeated using different samples. The results are plotted in figure 67 and 68.

![Mastercurve @ -10°C](image)

**Figure 67: Mastercurves constructed for two sets of test results, A12_1992**
From the obtained results the following conclusions were drawn:

- The results indicate that in reality the ageing process differs from material to material. There is no clear trend indicating that the older materials show a higher stiffness. In fact the oldest section, the A12_1987, has a lower stiffness compared to the relative young sections. Studying these differences is far beyond this research; nevertheless it is strongly believed that the material compositions e.g. the type of bitumen and type of filler used in these sections cause these differences. Care should be taken with these issues when developing laboratory ageing protocols to simulate field ageing.

- The result plotted in figure 67 and 68 indicate that mortar samples prepared for a specific section, are quite homogenous. No significant difference is observed when test data is compared. This is an indication that the performing the DSR test on the mortar samples is reproducible.
To compare the different sections with each other, stiffness values are selected at three different frequencies. These are presented in table 25.

**Table 25: Stiffness values during three different types of loadings (T= -10°C)**

<table>
<thead>
<tr>
<th>Section</th>
<th>1E-5 Hz</th>
<th>1Hz</th>
<th>40 Hz</th>
</tr>
</thead>
<tbody>
<tr>
<td>A12_1987</td>
<td>1.11E+08</td>
<td>2.08E+09</td>
<td>2.82E+09</td>
</tr>
<tr>
<td>A12_1992</td>
<td>4.07E+08</td>
<td>2.86E+09</td>
<td>3.67E+09</td>
</tr>
<tr>
<td>A15_1995</td>
<td>1.13E+08</td>
<td>1.99E+09</td>
<td>2.84E+09</td>
</tr>
<tr>
<td>A4_1997</td>
<td>9.09E+07</td>
<td>1.77E+09</td>
<td>2.49E+09</td>
</tr>
<tr>
<td>N9_2002</td>
<td>1.54E+08</td>
<td>2.42E+09</td>
<td>3.17E+09</td>
</tr>
<tr>
<td>A200_2002</td>
<td>1.41E+08</td>
<td>2.12E+09</td>
<td>2.77E+09</td>
</tr>
<tr>
<td>N3_2004</td>
<td>2.32E+08</td>
<td>2.06E+09</td>
<td>2.89E+09</td>
</tr>
<tr>
<td>A15_2006</td>
<td>1.83E+08</td>
<td>2.00E+09</td>
<td>2.63E+09</td>
</tr>
</tbody>
</table>

It was already mentioned that within LOT the DSR data has to be transformed first into a visco-elastic model. Chapter 4 already discussed the procedure. Hereafter the results are presented.

### 5.5.4 Determination of the Prony parameters

In the finite element program used for simulations, the visco elastic behavior of the mortar is described by means of the Prony Series.

This indicates that the DSR data has to be transformed first into Prony parameters. The procedure is already described in chapter four and the reason in chapter two.

During the DSR test the complex, loss and storage modulus are automatically measured.

By assuming a number of values for the different parameters (see paragraph 4.5) the complex, loss and storage modulus can be calculated using the mentioned formulas.

By using an optimization program, the fit between the calculated values and the measured values is optimized. Dependent on how good the fit is, the number of Prony parameters is determined. Figure 69 and figure 70 give an example of the curve fitting procedure. For this purpose use was made of MATLAB. For the eight cases studied in this research, the number of Prony parameters varied between 17 and 25.
Finally if all the Prony parameters are obtained, the time dependent stiffness can be calculated using formula 9, paragraph 4.5.
Table 26 gives an example of Prony parameters determined for the A15_2006 while the calculated stiffness curves for two different sections are illustrated in figure 71 and 72.

Table 26: Prony parameters A15_2006

<table>
<thead>
<tr>
<th>i</th>
<th>E0 [MPa]</th>
<th>a1 [-]</th>
<th>t1 [-]</th>
<th>ti [-]</th>
<th>ti [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8132</td>
<td>0.173345</td>
<td>7.78E-05</td>
<td>2.89E-06</td>
<td>1.37E-07</td>
</tr>
<tr>
<td>2</td>
<td>8132</td>
<td>0.007913</td>
<td>0.000225</td>
<td>8.38E-06</td>
<td>3.96E-07</td>
</tr>
<tr>
<td>3</td>
<td>8132</td>
<td>0.022902</td>
<td>0.000651</td>
<td>2.42E-05</td>
<td>1.15E-06</td>
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<tr>
<td>4</td>
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<td>0.066286</td>
<td>0.001885</td>
<td>7.02E-05</td>
<td>3.32E-06</td>
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<tr>
<td>5</td>
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<td>0.191849</td>
<td>0.005456</td>
<td>0.000203</td>
<td>9.61E-06</td>
</tr>
<tr>
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<td>0.000588</td>
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<td>0.004924</td>
<td>0.000233</td>
</tr>
<tr>
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<td>0.382851</td>
<td>0.014251</td>
<td>0.000674</td>
</tr>
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<td>0.041246</td>
<td>0.001951</td>
</tr>
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<td>11</td>
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<td>0.005648</td>
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<tr>
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<td>9.282082</td>
<td>0.345511</td>
<td>0.016346</td>
</tr>
<tr>
<td>13</td>
<td>8132</td>
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<td>26.86482</td>
<td>1</td>
<td>0.047309</td>
</tr>
<tr>
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</tr>
<tr>
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<td>0.396294</td>
</tr>
<tr>
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<td>24.24462</td>
<td>1.14698</td>
</tr>
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<td>1885.114</td>
<td>70.17038</td>
<td>3.319666</td>
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<td>203.0918</td>
<td>9.607998</td>
</tr>
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<td>15791.18</td>
<td>587.8016</td>
<td>27.8081</td>
</tr>
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<td>45703.88</td>
<td>1701.254</td>
<td>80.48405</td>
</tr>
<tr>
<td>21</td>
<td>8132</td>
<td>4651309</td>
<td>132279.2</td>
<td>4923.883</td>
<td>232.9423</td>
</tr>
<tr>
<td>22</td>
<td>8132</td>
<td>13462125</td>
<td>382851.2</td>
<td>14251.03</td>
<td>674.1969</td>
</tr>
<tr>
<td>23</td>
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<td>1108073</td>
<td>41246.26</td>
<td>1951.305</td>
</tr>
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<td></td>
</tr>
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</tbody>
</table>
After all the required information about the load, geometry and response was determined, the next step was to calculate the damage in the adhesive zones. Therefore numerous simulations were performed to determine occurring stresses in these zones. These stresses were then used for damage calculation. The results of the damage calculation are discussed in the next chapter.
Chapter 5 Input determination

5.6 Summary

This chapter presented the obtained required information for the LOT simulations. In the research approach it was already mentioned how the simulations were performed. It was also mentioned that for this research only the adhesive zone damage model was of interest. Therefore these two topics are not discussed here.

The determined information presented in this chapter, was used directly in LOT and simulations were performed at 4 different temperatures.

The obtained results are presented in the following chapter.
Chapter 6 Results

6. RESULTS

6.1 General

In this chapter the different general results of this research are presented. The first paragraph discusses the results which were obtained using the Lifetime Optimization Tool i.e. the calculated performance of the different sections. Hereafter the field performance of these sections is discussed and compared with the calculated results. By doing so the objectives of this research are obtained which was aimed at validating the LOT theory. Finally this chapter ends with a discussion of the obtained results. See figure 73.

Figure 73: Outline of chapter 6
6.2 Calculated performance

The previous chapter discussed how the required input for LOT was obtained. After different simulations with LOT the stresses in the adhesive zones were exported to Excel for post processing. By making use of the adhesive zone damage model, the damage development during a 24 h period was calculated.

With the notion damage development, is meant the speed at which damage develops.

To compare the performance of the different sections with each other, first a reference case had to be selected. Use is made of a standard porous asphalt mixture according to the Dutch design method. The response properties of a virgin mortar consisting of pen 70/100 bitumen were assigned to the mixture. These properties were determined during the development of LOT.

Pavement deflections for this reference case were calculated using a representative Dutch highway pavement, see table 27 and 28.

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (mm)</th>
<th>Poisson’s ratio</th>
<th>Stiffness (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>-10 °C</td>
</tr>
<tr>
<td>PA</td>
<td>50</td>
<td>0.35</td>
<td>10475</td>
</tr>
<tr>
<td>STAC</td>
<td>200</td>
<td>0.35</td>
<td>20950</td>
</tr>
<tr>
<td>Unbound base</td>
<td>225</td>
<td>0.4</td>
<td>400</td>
</tr>
<tr>
<td>Sand sub base</td>
<td>1000</td>
<td>0.4</td>
<td>100</td>
</tr>
<tr>
<td>Subgrade</td>
<td>∞</td>
<td>0.4</td>
<td>55</td>
</tr>
</tbody>
</table>

Table 27: Representative structural design of a Dutch motorway pavement

Table 28: Definition of mixture geometry for the reference case

- Equivalent stone radius: 4.8
- Stone density (kg/m³): 2650
- Percentage of stone (%): 80
- Mineral in mortar density (kg/m³): 2650
- Bitumen density (kg/m³): 1030
- Bitumen percentage (%): 4.5
- Void ratio (%): 20

All the eight sections, including the reference case, showed the best performance (lowest accumulated damage) at a temperature of 0°C. However the reference case showed the best performance at this temperature. Therefore this value was used as the reference value.
Finally by dividing all the calculated damage values with this reference value the relative damage during a 24 h period is obtained.

Figure 74 gives an illustration of the calculated stresses of the adhesive zones due to a moving tire and the deflection of the model under this moving load.

**Figure 74: Calculated stresses in the adhesive zones A12_1992 @ -10°C**

After performing different simulations for all the sections, the calculated stresses are exported to Excel for calculation of damage using the adhesive zone damage model.

Hereafter an illustration is given of the obtained results for the A12_1992.

**Figure 75: Calculated stresses in the adhesive zones A12_1992 @ -10°C, magnified**
Figure 76: Damage development due to traffic, pavement deflections and additional temperature stresses ($dt=3^\circ C$), A12_1992, average daily temp = -10°C

Figure 77: Damage development due to traffic, pavement deflections and additional temperature stresses ($dt=6^\circ C$), A12_1992, average daily temp = -10°C
The calculated results of the A12_1992 are presented in table 29. The damage calculation results of the reference case are presented in table 30.

**Table 29: Accumulated total damage for different temperature gradients A12_1992**

<table>
<thead>
<tr>
<th></th>
<th>dt=0</th>
<th>dt=1</th>
<th>dt=2</th>
<th>dt=3</th>
<th>dt=4</th>
<th>dt=5</th>
<th>dt=6</th>
<th>dt=7</th>
<th>dt=10</th>
</tr>
</thead>
<tbody>
<tr>
<td>-10</td>
<td>0.019295</td>
<td>0.052959</td>
<td>0.229234</td>
<td>0.635694</td>
<td>1.360366</td>
<td>2.495977</td>
<td>4.140917</td>
<td>6.399781</td>
<td>18.00491</td>
</tr>
<tr>
<td>0</td>
<td>5.46E-07</td>
<td>8E-07</td>
<td>1.98E-06</td>
<td>5.79E-06</td>
<td>1.64E-05</td>
<td>4.21E-05</td>
<td>9.63E-05</td>
<td>0.0002</td>
<td>0.001163</td>
</tr>
<tr>
<td>10</td>
<td>0.00039</td>
<td>0.000399</td>
<td>0.000421</td>
<td>0.00046</td>
<td>0.000523</td>
<td>0.000617</td>
<td>0.000751</td>
<td>0.000935</td>
<td>0.001894</td>
</tr>
<tr>
<td>20</td>
<td>0.010067</td>
<td>0.010085</td>
<td>0.010104</td>
<td>0.010125</td>
<td>0.010148</td>
<td>0.010174</td>
<td>0.010202</td>
<td>0.010233</td>
<td>0.010344</td>
</tr>
</tbody>
</table>

The reference case showed the best performance at a temperature of 0ºC. According to table 30 the accumulated damage was approximately 6.6E-07. Finally by dividing the values from table 29 by 6.6E-07, the relative daily damage is obtained. See table 31. In this way the insight is obtained how the A12_1992 performs compared to the reference case.

**Table 30: Accumulated total damage for different temperature gradients, reference case**

<table>
<thead>
<tr>
<th></th>
<th>dt=0</th>
<th>dt=1</th>
<th>dt=2</th>
<th>dt=3</th>
<th>dt=4</th>
<th>dt=5</th>
<th>dt=6</th>
<th>dt=7</th>
<th>dt=10</th>
</tr>
</thead>
<tbody>
<tr>
<td>-10</td>
<td>0.005786</td>
<td>0.005868</td>
<td>0.005989</td>
<td>0.006157</td>
<td>0.006378</td>
<td>0.006658</td>
<td>0.007003</td>
<td>0.007418</td>
<td>0.009136</td>
</tr>
<tr>
<td>10</td>
<td>0.000152</td>
<td>0.000152</td>
<td>0.000152</td>
<td>0.000152</td>
<td>0.000152</td>
<td>0.000152</td>
<td>0.000152</td>
<td>0.000152</td>
<td>0.000152</td>
</tr>
<tr>
<td>20</td>
<td>0.000386</td>
<td>0.000386</td>
<td>0.000386</td>
<td>0.000386</td>
<td>0.000386</td>
<td>0.000386</td>
<td>0.000386</td>
<td>0.000386</td>
<td>0.000386</td>
</tr>
</tbody>
</table>

In table 31 it can be seen that at a temperature of -10ºC, an increase of temperature fluctuations causes a rapid increase of damage. As temperatures increase this effect becomes less to almost not significant. This can be seen in the last two rows of table 31.

In chapter 5, it was already mentioned that the temperature fluctuations occur at an extreme low frequency. In this range relaxation potential is important. As the material has lost its relaxation potential due to ageing, it can be seen in table 31 what this leads to at low temperatures. This is a clear indication that in the winter, relaxation behavior influences the ravelling performance the most.
In similar way the performance of all the different sections was calculated.
This research focuses on winter damage therefore only the result obtained at -10°C and 0°C are of interest. The results are presented in table 32 and 33.

**Table 32: Obtained results for the different sections @ -10°C**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>% voids</td>
<td>21.3</td>
<td>19.1</td>
<td>20.6</td>
<td>23.1</td>
<td>21.8</td>
<td>21.8</td>
<td>20.8</td>
<td>20</td>
</tr>
<tr>
<td>% bitumen in</td>
<td>3.72</td>
<td>3.6</td>
<td>3.84</td>
<td>4.34</td>
<td>3.6</td>
<td>3.03</td>
<td>3.29</td>
<td>4.51</td>
</tr>
<tr>
<td>eq.stone diameter</td>
<td>7.98</td>
<td>7.78</td>
<td>8.26</td>
<td>8.17</td>
<td>7.49</td>
<td>8.53</td>
<td>7.94</td>
<td>8.24</td>
</tr>
<tr>
<td>G* @ 1E-5 Hz (Pa)</td>
<td>1.11E+08</td>
<td>4.07E+08</td>
<td>1.13E+08</td>
<td>9.09E+07</td>
<td>1.54E+08</td>
<td>1.41E+08</td>
<td>2.32E+08</td>
<td>1.83E+08</td>
</tr>
<tr>
<td>G* @ 1Hz (Pa)</td>
<td>2.08E+09</td>
<td>2.86E+09</td>
<td>1.99E+09</td>
<td>1.77E+09</td>
<td>2.42E+09</td>
<td>2.12E+09</td>
<td>2.06E+09</td>
<td>2.00E+09</td>
</tr>
<tr>
<td>G* @ 40 Hz (Pa)</td>
<td>2.82E+09</td>
<td>3.67E+09</td>
<td>2.84E+09</td>
<td>2.49E+09</td>
<td>3.17E+09</td>
<td>2.77E+09</td>
<td>2.89E+09</td>
<td>2.63E+09</td>
</tr>
<tr>
<td>D0 (mm)</td>
<td>0.183581</td>
<td>0.091765</td>
<td>0.184509</td>
<td>0.217447</td>
<td>0.195402</td>
<td>0.201163</td>
<td>0.224681</td>
<td>0.196552</td>
</tr>
<tr>
<td>D300 (mm)</td>
<td>0.172935</td>
<td>0.083053</td>
<td>0.173117</td>
<td>0.203362</td>
<td>0.184088</td>
<td>0.183032</td>
<td>0.208942</td>
<td>0.185086</td>
</tr>
<tr>
<td>D0-D300 (SCI)</td>
<td>0.010646</td>
<td>0.008712</td>
<td>0.011392</td>
<td>0.014085</td>
<td>0.011314</td>
<td>0.018132</td>
<td>0.015739</td>
<td>0.011466</td>
</tr>
<tr>
<td>Relative daily damage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>dt=3</td>
<td>6.33E+04</td>
<td>9.58E+05</td>
<td>4.85E+04</td>
<td>2.34E+04</td>
<td>1.93E+05</td>
<td>2.44E+05</td>
<td>2.50E+05</td>
<td>2.48E+04</td>
</tr>
<tr>
<td>dt=6</td>
<td>3.16E+05</td>
<td>6.24E+06</td>
<td>2.74E+05</td>
<td>9.79E+04</td>
<td>9.72E+05</td>
<td>9.10E+05</td>
<td>1.43E+06</td>
<td>1.08E+05</td>
</tr>
</tbody>
</table>

**Table 33: Obtained results for the different sections @ 0°C**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>% voids</td>
<td>21.3</td>
<td>19.1</td>
<td>20.6</td>
<td>23.1</td>
<td>21.8</td>
<td>21.8</td>
<td>20.8</td>
<td>20</td>
</tr>
<tr>
<td>% bitumen in</td>
<td>3.72</td>
<td>3.6</td>
<td>3.84</td>
<td>4.34</td>
<td>3.6</td>
<td>3.03</td>
<td>3.29</td>
<td>4.51</td>
</tr>
<tr>
<td>eq.stone diameter</td>
<td>7.98</td>
<td>7.78</td>
<td>8.26</td>
<td>8.17</td>
<td>7.49</td>
<td>8.53</td>
<td>7.94</td>
<td>8.24</td>
</tr>
<tr>
<td>G* @ 1E-5 Hz (Pa)</td>
<td>1.35E+07</td>
<td>1.10E+08</td>
<td>1.74E+07</td>
<td>1.21E+07</td>
<td>1.88E+07</td>
<td>1.80E+07</td>
<td>5.73E+07</td>
<td>3.45E+07</td>
</tr>
<tr>
<td>G* @ 1Hz (Pa)</td>
<td>1.23E+09</td>
<td>1.99E+09</td>
<td>1.16E+09</td>
<td>1.02E+09</td>
<td>1.47E+09</td>
<td>1.31E+09</td>
<td>1.28E+09</td>
<td>1.30E+09</td>
</tr>
<tr>
<td>G* @ 40 Hz (Pa)</td>
<td>2.17E+09</td>
<td>2.98E+09</td>
<td>2.11E+09</td>
<td>1.86E+09</td>
<td>2.49E+09</td>
<td>2.19E+09</td>
<td>2.15E+09</td>
<td>2.09E+09</td>
</tr>
<tr>
<td>D0 (mm)</td>
<td>0.19561</td>
<td>0.09566</td>
<td>0.19047</td>
<td>0.22662</td>
<td>0.20801</td>
<td>0.21445</td>
<td>0.23339</td>
<td>0.20922</td>
</tr>
<tr>
<td>D300 (mm)</td>
<td>0.18282</td>
<td>0.08536</td>
<td>0.17734</td>
<td>0.21199</td>
<td>0.19445</td>
<td>0.19299</td>
<td>0.21511</td>
<td>0.19549</td>
</tr>
<tr>
<td>D0-D300 (SCI)</td>
<td>0.012796</td>
<td>0.010299</td>
<td>0.013131</td>
<td>0.016625</td>
<td>0.013559</td>
<td>0.021462</td>
<td>0.018289</td>
<td>0.013731</td>
</tr>
<tr>
<td>Relative daily damage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>dt=3</td>
<td>0.64</td>
<td>8.73</td>
<td>0.26</td>
<td>0.58</td>
<td>3.26</td>
<td>110.50</td>
<td>2.99</td>
<td>0.30</td>
</tr>
<tr>
<td>dt=6</td>
<td>0.80</td>
<td>145.13</td>
<td>0.35</td>
<td>0.66</td>
<td>4.98</td>
<td>136.67</td>
<td>11.66</td>
<td>0.36</td>
</tr>
</tbody>
</table>
In table 32 and 33 SCI stands for Surface Curvature Index. This parameter gives an indication about the deflection of the porous asphalt under a wheel load. This parameter is calculated by subtracting the deflection calculated 300 mm away, from the deflection calculated right under the wheel load. The results from table 32 and 33 are combined in the following figures.

**Figure 78: Relative daily damage, temperature difference of 3°C**

Figure 78 and 79 show the performance of the different sections.

**Figure 79: Relative daily damage, temperature difference of 6°C**

The ellipse in figure 79 is magnified in figure 80.
When the results from table 32 and 33 are compared with the figures 78, 79 and 80 it is observed that the relative young sections show significant more damage then the relative old sections. According to the hypothesis of this research this would mean that the relative young sections would contain a stiffer mortar and thus a poor relaxation potential.

Previously it was already mentioned that relaxation potential is important in the low frequency range. Figure 81 shows the stiffness values of the different sections in this range.
When the mastercurves are compared with the performance of the sections one can observe immediately that the sequence of the mastercurves from figure 81 coincides with the sequence of the performance in the figures 78, 79 and 80. This is a solid proof that in cold weather conditions, excessive damage of porous asphalt is caused mainly by the relaxation behavior of the mortar i.e. high stiffness values.

The comparison indicates clearly that the stiffest material, thus the material with the lowest relaxation potential, shows the highest damage in the calculations.

The next step in this research was to compare the calculated performance of the different section with the actual field performance. This was the main objective of this study.

The next paragraph discusses how the different sections behaved during the winter of 2009/2010.

6.2 Field performance of the different sections

As mentioned earlier two types of sections were selected for this research:

- relative old sections that showed no significant ravelling damage during the last winter
- relative young sections that showed severe ravelling damage during the last winter

The selection was based on:

- An inventory performed by means of visual inspections by different divisions of the Dutch department of Public Works after the winter of 2009/2010

- Information from Winfrabase: this is a software database of the Dutch department of Public Works, which integrates pavement management with a construction and materials database. Based on this information rehabilitation of resurfacing works are planned

- Information gathered from the MJPV (in Dutch:” Meerjarenplanning Verhardingsonderhoud”): this is a planning that the Dutch department of Public Works uses to plan rehabilitation works on the primary road network
Based on the information summarized above, 8 different sections were selected for this research, see table 34.

**Table 34: The selected sections**

<table>
<thead>
<tr>
<th>Section</th>
<th>Date of construction</th>
<th>Location (km)</th>
<th>Lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>A15</td>
<td>2006</td>
<td>74.6 - 74.9</td>
<td>1HRR,3RR,RS</td>
</tr>
<tr>
<td>N9</td>
<td>2002</td>
<td>80.0 - 81.4</td>
<td>1RL,1RR</td>
</tr>
<tr>
<td>A200</td>
<td>2002</td>
<td>4.8 - 5.2</td>
<td>1HRR,2RR</td>
</tr>
<tr>
<td>N3</td>
<td>2004</td>
<td>1.3 - 1.5 / 3.8 - 4.5</td>
<td>1HRR,2RR,RS</td>
</tr>
<tr>
<td>A4</td>
<td>1997</td>
<td>51.65 - 52.1</td>
<td>HRR 1RR</td>
</tr>
<tr>
<td>A15</td>
<td>1995</td>
<td>117.8 - 118.5</td>
<td>1HRR,2RR,RS</td>
</tr>
<tr>
<td>A12</td>
<td>1992</td>
<td>118.2 - 119.4</td>
<td>HRL-2RL</td>
</tr>
<tr>
<td>A12</td>
<td>1987</td>
<td>31.27 - 30.58</td>
<td>1HR-L,1R-L</td>
</tr>
</tbody>
</table>

The next step was to rank the amount of ravelling. At this point it is important to distinguish between ravelling in general and ravelling because of winter conditions. It is known that loss of stones from the porous asphalt surface usually develops several years after construction. In the last winter this development occurred at a much higher speed which resulted in totally damaged sections, see figure 82.

![Figure 82: Example of excessive ravelling](image)

According to the Dutch method three types of ravelling are distinguished:
- Light, 5-10 % of the representative m²
- Moderate, 10- 20% of the representative m²
- Heavy, 20 % and more of the representative m²
Ravelling in the winter is recognized as heavy. In order to monitor the exact percentage the road has to be closed down. For this purpose many measures have to be taken and unnecessary costs play a role. Nevertheless after the winter the damage is so severe that together with inventarisation reports from before the winter, an experienced visual inspector knows instantly that he/she deals with winter damage. This information is saved in the earlier mentioned plans/databases and based on this, rehabilitation works are planned.

For this research it was decided to divide the performance of the different sections in three classes:
- no damage, 0
- winter damage, 1
- damage but not directly related to the winter, 2

The following considerations were taken into account:

- Information from the data bases
- Visual inspection at the moment of drilling cores
- Discussions with an advisor from the Dutch department of Public Works, responsible for drilling of the cores
- Inspections performed on different sections approximately 7 months after the winter. This inspection was performed on the following sections:
  - A15_1995
  - A15_2006
  - N3_2004
  - N9_2002
  - A4_1997
After interpretation of all the gathered information, the following ranking was given to the different sections:

**Table 35: Ranking of the different sections**

<table>
<thead>
<tr>
<th>Section</th>
<th>Year of construction</th>
<th>Ranking</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>A15_2006</td>
<td>2006</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>N3_2004</td>
<td>2004</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>N9_2002</td>
<td>2002</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>A200_2002</td>
<td>2002</td>
<td>2</td>
<td>poor performance from the beginning</td>
</tr>
<tr>
<td>A4 1997</td>
<td>1997</td>
<td>0</td>
<td>drilled in left traffic lane</td>
</tr>
<tr>
<td>A15_1995</td>
<td>1995</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>A12_1992</td>
<td>1992</td>
<td>1</td>
<td>heavily loaded, extreme increase of damage in the winter</td>
</tr>
<tr>
<td>A12_1987</td>
<td>1987</td>
<td>0</td>
<td>drilled in left traffic lane</td>
</tr>
</tbody>
</table>

The last step in reaching the objectives of this study was comparing the calculated performance with the field performance. This is discussed in the following paragraph.
6.3 Comparison of the performances

Since all the required input information for LOT was selected directly from these different sections, the aim was to compare the calculated performance with the field performance. In this way LOT would be validated on its capability to explain winter damage of porous asphalt.

This validation is illustrated in the following figures.

---

**Figure 85: Calculated performance versus field performance**

**Figure 86: Absolute performance of the different sections**
Chapter 6 Results

From the figures 85 and 86 it is concluded that the field performance of the different sections is in good agreement with the calculated performance. The sections which showed a relative low daily damage in the calculations, showed a good performance in the field while the sections which showed a poor performance in calculations, showed significant ravelling damage during the last winter of 2009/2010.

It should be realized that the calculated performance with LOT, can not be translated directly into lifetime expectancy. Within LOT only the initiation of ravelling is considered.

However the obtained results can be used to compare the performance of different mixtures during the winter.

This means that if contractors perform the same analysis with LOT for a certain porous asphalt mixture, comparison of their results with the result obtained in this study, will provide them with sufficient insight into the performance of their mixture in the winter.

If the relative daily damage stays below a certain value, a good performance can be expected in the winter.

6.4 Discussion

From the presented results in this chapter it is concluded that the Lifetime Optimization Tool is capable to explain the field performance of porous asphalt mixtures in the winter. Secondly it can be concluded that the procedures followed to determine the required input for LOT are accurate enough and sufficient. In case of winter damage, a poor performance in calculations indicates excessive ravelling in the field while a good performance indicates a section with no significant ravelling damage. This is solid for seven of the eight cases studied in this research. Due to lack of information, it was not clear whether the field performance of the A200_2002 was directly related to the winter or not. However this section showed a poor performance in the calculations and also in the field. Nevertheless this section showed that sections with a poor performance in the field also showed a poor performance in the calculations.
7. REGRESSION ANALYSIS

7.1 Introduction

Until now the obtained results indicate that poor relaxation potential of the mortar i.e. high stiffness values, leads to excessive ravelling in the winter. These findings resulted from a study conducted by Huurman and were successfully validated in this study.

However the effect of the remaining factors such as the stone size, the bitumen percentage and the voids content could not be identified during calculations. From the eight sections studied in this research, the 4 sections that showed the worst performance had the highest stiffness values and at the same time the lowest bitumen percentage. See table 36. All the results indicated that the high stiffness values caused an increase of damage while the effect of bitumen percentage was not observed. However questions still remained like e.g. how much did the bitumen percentage contribute to the performance of each mixture? To find answers for questions like these, a regression analysis was performed.

The results of the regression analysis are discussed in the next paragraph.

Table 36: Properties of the different sections

<table>
<thead>
<tr>
<th>Section</th>
<th>$G^*$ (Pa) @ 1E-05 Hz</th>
<th>Bitumen percentage</th>
<th>Void content (%)</th>
<th>Performance</th>
</tr>
</thead>
<tbody>
<tr>
<td>A200_2002</td>
<td>1.41E+08</td>
<td>3.6</td>
<td>21.8</td>
<td>poor</td>
</tr>
<tr>
<td>N9_2002</td>
<td>1.54E+08</td>
<td>3.03</td>
<td>21.8</td>
<td>poor</td>
</tr>
<tr>
<td>N3_2004</td>
<td>2.32E+08</td>
<td>3.29</td>
<td>20.8</td>
<td>poor</td>
</tr>
<tr>
<td>A12_1992</td>
<td>4.07E+08</td>
<td>3.6</td>
<td>19.1</td>
<td>poor</td>
</tr>
<tr>
<td>A12_1987</td>
<td>1.11E+08</td>
<td>3.72</td>
<td>21.3</td>
<td>good</td>
</tr>
<tr>
<td>A15_1995</td>
<td>1.13E+08</td>
<td>3.84</td>
<td>20.6</td>
<td>good</td>
</tr>
<tr>
<td>A4_1997</td>
<td>9.09E+07</td>
<td>4.34</td>
<td>23.1</td>
<td>good</td>
</tr>
<tr>
<td>A15_2006</td>
<td>1.83E+08</td>
<td>4.51</td>
<td>20</td>
<td>good</td>
</tr>
</tbody>
</table>
7.2 Results

For this research a regression analysis is useful because it explains what the effect of multiple variables is on the dependent variable, in this case the damage. A regression analysis does not only depict if a relationship exist between the independent variables and the dependant variable, but it also depicts the height and the direction of the relationship.

In this research the regression analysis was only performed to find out which parameters showed a significant correlation with the damage and not to obtain a regression expression. Determining a regression expression would require a large data set and since in this study only eight sections are studied, it is not sinful to derive such an expression.

For the regression analysis the following parameters were used as independent variables:

- Stone size
- Void content
- Bitumen percentage
- Stiffness @ temp. loading frequency range (1E-5 Hz), stiff1
- Stiffness @ pavement deflection loading frequency range (1Hz), stiff 2
- Stiffness @ traffic loading frequency range (40 Hz), stiff3

The dependent variable in this case is the damage. The effect of the load was assumed to be already included in the damage values. Therefore load was not used in the analysis.

Use was made of SPSS 17 statistical software. A multiple regression was performed. Prior to this a correlation analysis was performed in order to find out if there were independent variables which had a high correlation with each other. This would influence the regression model and according to literature [Huizingh 1996] one of these variables has to be removed out of the analysis. The correlation results are presented in figure 87.
It can be seen that a high correlation exist between stiffness 1, 2 and 3. Therefore stiffness 2 and stiffness 2 were left out the analysis. Hereafter a linear regression analysis was performed for -10°C.

Linear regression analysis assumes that there is a linear relationship between variables. The following formula is used:

\[ \hat{Y} = B_0 + B_1 \cdot X_1 + B_2 \cdot X_2 + \ldots + B_k \cdot X_k \]  

(27)

where:

\[ \hat{Y} = \] the predicted value

\[ B_0 = \] the intercept

\[ B_{1..k} = \] the partial regression coefficient of \( X_{1..k} \).

The parameter \( B_1 \) gives an indication of the influence of parameter \( X_1 \) on the \( Y \) value. In case there is no linear relation between the different independent variables and the dependent variable, the relationship can be made linear by e.g.: transforming the values to log values or square root etc.

Note that there are many options available in SPSS 17 software. For detailed information and procedures, reference is made to Huizingh 1996 and Field 2005.
Chapter 7 Regression analysis

The stepwise linear regression method was chosen. This method assumes an “empty” model and during each step one independent variable is added to the regression analysis. This depends if the variable is significant.

A confidence level of 95% is chosen, which indicates that a variable is significant in case the significance level is smaller than 0.05.

Two regressions analysis were performed:

1) The first analysis considered the following independent variables:
   - Stone size
   - Percentage bitumen
   - Percentage voids
   - Stiffness 1

   The dependent variable was set to be the damage calculated due to temperature a difference of 6 °C. The total number of cases was 8 (eight sections and 1 damage value).

2) The second case was the same as case 1 but, the damage due to 9 different temperature differences was added to the analysis. This led to a total of 72 cases (eight sections and 9 different damage values).

Figure 88 gives an illustration of the regression results obtained for case 1.

<table>
<thead>
<tr>
<th>Model</th>
<th>R</th>
<th>R Square</th>
<th>Adjusted R Square</th>
<th>Std. Error of the Estimate</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>.793&lt;sup&gt;a&lt;/sup&gt;</td>
<td>.630</td>
<td>.568</td>
<td>.40090</td>
</tr>
<tr>
<td>2</td>
<td>.975&lt;sup&gt;b&lt;/sup&gt;</td>
<td>.951</td>
<td>.931</td>
<td>.15982</td>
</tr>
<tr>
<td>3</td>
<td>.992&lt;sup&gt;c&lt;/sup&gt;</td>
<td>.934</td>
<td>.973</td>
<td>.10067</td>
</tr>
</tbody>
</table>

a. Predictors: (Constant), Stiffness1
b. Predictors: (Constant), Stiffness1, Bitumen
c. Predictors: (Constant), Stiffness1, Bitumen, Stonessize
d. Dependent Variable: LOGD

Figure 88: Regression results case 1

It can be seen in figure 88 that a high correlation is obtained between predicted damage values and entered values.
According to the model, three variables were recognized as significant:

- Stiffness
- Bitumen percentage
- Stone size

Figure 89 shows the calculated model parameters.

![Figure 89: Calculated model coefficients](image)

According to this model, an increase of bitumen percentage causes a decrease of damage while an increase of stiffness will cause an increase of damage.

Further on the model predicts that an increase in stone size will cause an decrease of damage however it can be seen in figure 89 that the B value for stone is very close to not being significant (sig =0.043).

Therefore it is concluded that model 2 of case 1 should be adapted (see # 2, figure 89).

Substitution of the coefficients for model 2 in formula 27 led to the following expression for case 1:

\[
\log D = 1.536 + 3.942 \cdot 10^{-9} \cdot Stiffness1 - 0.718 \cdot Bit. \\
\]

where:

\[
\log D = \log \text{damage} \\
Stiffness1 = \text{stiffness of the mortar @ frequency of } 1 \cdot 10^{-5} \text{Hz} (Pa) \\
Bit = \text{percentage bitumen "on" 100% aggregate (%)}
\]

The results for the second regression analysis are shown in figure 90 and figure 91.
Figure 90: Regression results case 2

It can be seen in figure 90 that the correlation between predicted and entered damage values is very low. Further on it can be seen in figure 91 that the intercept value is not significant.

From both observations it is concluded that the model is very inaccurate. However it can be seen that only the bitumen percentage and stiffness 1 contribute to the correlation value.

After substitution of the coefficients from figure 91, the following expression was obtained for case 2:

\[
\log D = 1.349 + 3.219 \times 10^{-9} \cdot Stiffness_1 - 0.797 \cdot Bit. \\
\]

\textit{where}:

\[
\log D = \log \text{damage} \\
Stiffness_1 = \text{stiffness of the mortar @ frequency of } 1 \times 10^5 \text{Hz} \ (\text{Pa}) \\
Bit = \text{percentage bitumen "on" 100% aggregate (\%)}
\]
When the different results are analyzed, it is observed that for both cases two variables remained significant, the bitumen percentage and stiffness 1. Further it was observed that the direction for these two parameters remained the same while the heights (B-values) varied little. See table 37.

Table 37: B-values of the two cases

<table>
<thead>
<tr>
<th></th>
<th>B-values</th>
<th>Stiffness</th>
<th>Bitumen percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>3.942E-09</td>
<td>0.718</td>
<td></td>
</tr>
<tr>
<td>Case 2</td>
<td>3.219E-09</td>
<td>0.797</td>
<td></td>
</tr>
</tbody>
</table>

In the following paragraph the obtained results are discussed in detail.

7.3 Evaluation of the results

First of all it is stated explicitly that the aim of the regression analysis was merely to obtain insight into which factors contribute to the damage development and not to derive an expression to predict damage values.

The regression results indicated that the bitumen percentage and the stiffness of the mortar in the low frequency range influenced the damage development. The influence of the other variables could not be observed. Several reasons are identified for this:

- A linear regression analysis was performed and according to the conducted literature this is only allowed when the independent variables show a linear relationship with the dependent variable. At first none of the independent variables used in the analysis showed a linear trend with damage. To solve this problem the damage values were transformed into log values. After this only the bitumen percentage and stiffness 1 showed a linear relationship with log D. And as observed in both cases these two variables are significant.

- The number of cases used in the regression analysis was too small. According to literature the number of cases should be at least 15 times the number of independent variables. For case 1 the number of cases was only 8 and the correlation coefficient was 0.98 while for case 2 the number of cases was 72 and the correlation coefficient was only 0.34. Nevertheless the result from the first analysis should be considered as less to almost not accurate.
The main difference between the eight sections studied in this research, was the bitumen percentage and the stiffness values. The voids content and stone size did not differ significantly therefore they were also not identified in the regression analysis as factors which influence the damage significantly.

Apart from the above discussed points, the regression analysis showed that two factors influence the damage development significantly. The LOT calculations performed in this study already proved that high stiffness values especially in the low frequency range, the range were relaxation behavior is important, resulted in a rapid increase of damage. The effect of bitumen percentage was not observed during calculations.

From practical point of view, it is known that a higher bitumen percentage will lead to a relative thicker mortar film and as the mortar film thickness is increased, field ageing will occur at a slower rate. As a result the material will maintain its relaxation potential for a longer period. However the question remains if in the winter, when temperatures drop below freezing point, the adhesive zones will sufficiently benefit from this. To find this out additional simulations were performed for the N9_2002 which contained the lowest bitumen percentage. In two of the simulations the bitumen content was increased while in the last simulation the bitumen content was kept constant and properties of a virgin mortar were assigned to the cohesive zones. The results of these simulations are illustrated in figure 92.

Figure 92: Effect of bitumen content and virgin mortar properties on relative daily damage N9_2002 (dT=6)
From figure 92 the following conclusions are drawn:

- When the bitumen content is increased from 3.03 to 4.5 %, the relative daily damage decreases significantly with approximately 50%.

- If the bitumen content is increased further, the damage decreases at a much slower rate. It is believed that at a certain point, an optimum is reached after which an increase of bitumen content will not result in a decrease of damage. Additional LOT simulations with higher bitumen percentages were not performed to prove this. To the author’s knowledge this was not considered as important since currently porous asphalt mixtures in the Netherlands are designed using 5.5 % bitumen.

- When the bitumen content is kept constant and the properties of a virgin mortar are assigned to the model, it can be seen that the damage decreases rapidly with a factor 6. For winter damage, this indicates that even if the binder content is increased, this will never overrule the effect of relaxation behavior. From this it is concluded that the adhesive benefit more from a flexible material then from a higher bitumen content in the winter.

The conclusions summarized above indicate clearly that in the winter flexibility of the material is dominant. In case the mortar of a specific porous asphalt mixture has a low stiffness, a good performance of the mixture can be expected in the winter. This indicates that solutions have to be found to improve the mortar in such a way that it remains viscous in the winter. Studying this is far beyond this research and is discussed here.

However when stiffness values of different mortars are compared with each other, a good prediction can be done with respect to performance in the winter. To illustrate this, three porous asphalt mixtures were produced using standard laboratory equipment. The three mixtures were produced using different types of bitumen. From each mixture mortar was selected and tested using the DSR test setup to determine the mastercurves.
These properties were first transformed and then used in LOT simulations to calculate the performance of each mixture under winter conditions. The results from this additional performed experiment are discussed in the following chapter.
8. THE EFFECT OF DIFFERENT BITUMEN

8.1 General

The last objective of this research was to show how LOT can be put into practice in the future to prevent winter damage.

Until now all the results obtained in this study have shown that stiffness of the mortar controls the damage development of porous asphalt in the winter.

This means that if the mastercurves of different mortars are compared with each other, this information can be used as indicator about the performance during the winter.

Especially for contractors this is attractive since they can use this information to decide which mixture they should apply to ensure that in the winter, their maintenance costs stay low.

In this chapter results are presented from LOT simulations which were performed for one type of porous asphalt mixture using stiffness properties of three different mortars.

8.2 Mixture information

The composition of the mixture is presented in table 38.

Table 38: Composition of the PA mixture

<table>
<thead>
<tr>
<th>Material</th>
<th>% on</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bestone 4/8</td>
<td>16.6</td>
</tr>
<tr>
<td>Bestone 8/11</td>
<td>48.0</td>
</tr>
<tr>
<td>Bestone 11/16</td>
<td>22.4</td>
</tr>
<tr>
<td>Bestone brekerzand</td>
<td>8.1</td>
</tr>
<tr>
<td>Filler</td>
<td>4.9</td>
</tr>
<tr>
<td>Fibres</td>
<td>0.2</td>
</tr>
<tr>
<td>Bitumen 70/100</td>
<td>5.4</td>
</tr>
</tbody>
</table>

Three types of mixtures were produced using standard laboratory mixing equipment:

- ZOAB 10+ with standard pen 70/100 bitumen
- ZOAB 10+ with standard pen 70/100 bitumen produced at 100°C, LEAB
- ZOAB 10+ with polymer modified bitumen, PMB

After mixing a part of the material from all the three mixtures was aged according to a recently developed laboratory ageing protocol [Sharew 2010].
This led to a total of 6 different mixtures, three fresh materials and three aged materials.

The mortar samples for DSR testing were prepared following the same procedure presented earlier in this report.

The results from the DSR tests are presented in the next paragraph.

**8.3 DSR test results**

Frequency sweep tests to determine the stiffness of the mortar were performed at 7 different temperatures: -10°C, 0°C, 10°C, 20°C, 30°C, 40°C and 50°C. The results are illustrated in the following figures.

*Figure 93: Mastercurves of the three different mortars, virgin*

It can be observed in figure 93 that the LEAB_mortar has the highest stiffness and will therefore behave more elastic.

This is strange since it was expected that for the virgin materials, the stiffness would be approximately the same. Especially for the LEAB and the 70/100 the same properties were expected.

Compared to the 70/100 mortar, the LEAB_mortar has the same composition and also the bitumen type is identical.
In both cases the bitumen is heated up to an temperature of approximately 175 °C prior to mixing with the aggregates.

However in case of LEAB, the hot bitumen is first emulsified using water and then mixed with the aggregates. This allows production at lower temperatures which results in consumption of less energy hence it’s name, Low Energy Asphalt Beton (LEAB).

It is believed that this production process causes an increase of stiffness for the mortar. Further on it can be observed in figure 93 that the polymer modified mortar (PMB) behaves as expected. Polymers are used to improve properties of the mixture at high temperatures especially to prevent rutting of pavements which is a type of distress that mainly develops at relatively high temperatures.

Addition of polymers results in an increase of stiffness and lower phase angles. Both can be observed in figure 93.

However compared to the LEAB material, the PMB mortar has a lower stiffness.

In case of the aged mortars, the difference between the different mortars is not that significant. For this situation the LEAB mortar shows a higher stiffness and lower phase angle. See figure 94.

![Mastercurve @ -10 °C](image)

**Figure 94: Mastercurves of three different mortars, aged**
From both figures it can be concluded that the difference between the three tested mortars lies mainly in the low frequency range. In the previous chapters it was already shown that especially the behavior in this range is important in the winter.

Based on this information it is expected that in damage calculations, the LEAB mixture will show the worst performance. The results of the damage calculations are discussed in the next chapter.

8.4 Results

It is mentioned numerous times in this report that LOT requires information about the geometry, load and material behavior i.e. stiffness.

The material behavior followed from the test results presented in the previous chapter. Table 39 shows the geometrical input that was used to generate the 2D idealized model.

<table>
<thead>
<tr>
<th>Table 39: Used geometrical input for simulations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average grain diameter (mm)</td>
</tr>
<tr>
<td>Mix density kg/m³</td>
</tr>
<tr>
<td>Density mineral in mortar kg/m³</td>
</tr>
<tr>
<td>Density stone kg/m³</td>
</tr>
<tr>
<td>Density bitumen kg/m³</td>
</tr>
<tr>
<td>% stone</td>
</tr>
<tr>
<td>% bitumen on</td>
</tr>
<tr>
<td>% air voids</td>
</tr>
</tbody>
</table>

Again three types of loading were modeled. Pavement deflections were calculated using the pavement structure details given in table 40.

<table>
<thead>
<tr>
<th>Table 40: Structural design used for calculation of pavement deflections</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>PA</td>
</tr>
<tr>
<td>STAC</td>
</tr>
<tr>
<td>Unbound base</td>
</tr>
<tr>
<td>Sand sub base</td>
</tr>
<tr>
<td>Subgrade</td>
</tr>
</tbody>
</table>

Loadings due to temperature fluctuations and traffic were simulated following the same procedures discussed in chapter 4 and 5.
Figure 95 shows the calculation results for the three virgin mortars.

![Figure 95: Performance of the virgin materials, $dt = 6 \degree C$](image)

It can be noticed in figure 95 that the LEAB mixture shows the highest daily damage compared to the other two mixtures. This was expectable since the LEAB mortar showed the highest stiffness.

Considering this it was expected that the PMB mixture would perform worse then the 70/100 mixture since the PMB mortar was slightly stiffer then the 70/100 mixture. The results however proved that this was not the case.

It can be seen in figure 95 that the PMB mixture performs slightly better.

The main reason for this lies in determining the material behavior of the mortar.

It was already mentioned in chapter four that the DSR data i.e. the mastercurves (figure 93, 94) can not be used directly in LOT.

First the measured data during the DSR test is fitted using the Christensen-Anderson model. The fitted data is then used in MATLAB to fit the Prony model. Based on this fit the number of Prony terms is determined.

From this it can be concluded that if fitting the DSR data is not accurate, this will result in a poor fit of the Prony model and might result in non representative model parameters. Figure 96 illustrates the fitting procedure of the PMB_virgin material.
It can be seen in figure 96 that especially for the phase angle, the Christensen-Anderson model is unable to fit the measured data in the low frequency range (high temperatures). This was only observed for the polymer modified mortar. Figure 97 and 98 illustrate the fitting procedure of the virgin PMB mortar.
Figure 98: Fitting of the phase angle of the PMB_virgin mortar (T=10°C)

It can be seen in figure 97 that the fit of the G* is accurate while figure 98 shows that the fit of the phase angle is not smooth over the entire frequency range.

It is strongly believed that this inaccuracy, illustrated in figure 98, caused the difference between the PMB mortar and the 70/100 mortar, observed in figure 95. Nevertheless in figure 95 it can be seen clearly that the material with the highest stiffness, shows the worst performance.

In case of the aged PMB mortar, the difference between the data and the model was not that significant. See figure 99.
Further it was observed in figure 94 that the mastercurves for the aged PMB mortar and the aged 70/100 mortar were almost identical while the aged LEAB mortar showed a higher stiffness. Based on this it was expected that the aged LEAB material would show the worst performance while the other two material would show identical results.

The calculation results for the porous asphalt mixture with the aged mortar properties are shown in figure 100. It can be seen that in case of the aged materials, the performance is completely as expected. The material with the highest stiffness, showed the worst performance while the two other sections which had same stiffness properties, showed identical performances.
8.5 Evaluation

From the obtained results presented in this chapter, the following conclusions are drawn:

- LOT can practically put to use to study the performance of porous asphalt mixtures in the winter.
- On the basis of mortar stiffness values, mastercurves, a good estimation can be done with respect to the performance of porous asphalt in the winter.
- The measured phase angle data of polymer modified materials can not be fitted optimal by the so called S-shaped functions. However this was only observed for one material. Nevertheless one should take care when using these type of functions to fit phase angle data for polymer modified mortars.
9. CONCLUSIONS & RECOMMENDATIONS

In this chapter the general conclusion and recommendations of this research are presented. Before discussing these, first the research objectives are recalled. The objective of this research was to find valid answers for the following questions:

- Is the proposed theory correct? Does poor relaxation behavior cause excessive ravelling?
- If not, why?
- If yes, how can LOT then be put in practice to avoid such failures on the Dutch motorways in the future?

- Firstly from all the presented results it is concluded that excessive ravelling in the winter is caused by a reduced relaxation behavior i.e. high stiffness of the mortar. The sections with the stiffest mortar showed a poor calculated performance. For seven of the eight cases this was in agreement with the observed field behavior. From this it is concluded that LOT can be used to explain winter damage.

- It was shown in this study that LOT can easily put to practice to study the performance of different porous asphalt mixtures. One should realize that within LOT the initiation of ravelling is calculated. Before failure occurs a secondary process takes place which is not studied here. This would require information of how ravelling propagates in time. For this purpose additional research regarding this issue is recommended.

- The result obtained in this research indicated that the amount of bitumen in a porous asphalt mixture, contributes to the performance in the winter. In case the bitumen percentage is low, this causes an increase of damage. However the effect of a high stiffness remains dominant in the winter.
Chapter 9 Conclusions & Recommendations

- The good agreement between the calculated performance and the observed field performance indicate that the procedures followed in this research are sufficient enough to obtain representative results.

- In this research no distinction was made between cohesive failure and adhesive failure. Only the adhesive zone damage model was used in damage calculations since no strength parameters for the cohesive zone damage model at low temperatures were available. It is recommended to obtain strength parameters for the cohesive zone damage model at low temperatures to exclude doubts about at which point failure will occur first.
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APPENDICES

1. Copy of the paper which presents the results of the previous conducted study were LOT was used to explain winter damage

2. Input used for calculating pavement deflections for the different sections

3. Mastercurve fitting of DSR data for the different sections
Appendix 1

Porous Asphalt ravelling in cold weather conditions

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L. Mo, Wuhan University of Technology, P.R. China & Delft University of Technology, The Netherlands
M.F. Woldekidan, Delft University of Technology, The Netherlands

ABSTRACT

For environmental reasons the application of Porous Asphalt concrete, PA, on the primary road network is mandatory in the Netherlands. At the moment approximately 90% of the Dutch primary road network has a PA surfacing. During the last Dutch winter temperatures dropped below -10°C nation wide and locally temperatures close to -20°C occurred. During this cold period rapid and aggressive ravelling damage developed in some motorway sections. A short but intense national discussion about PA application followed and ended without conclusion as temperatures rose.

LOT (Lifetime Optimisation Tool) is a meso scale mechanistic mixture design tool for PA discussed in another ENVIROAD 2009 paper (Huurman et al., 2009). Here LOT is applied to explain the rapid development of ravelling in cold weather conditions. It is shown that LOT pinpoints exactly which phenomena cause winter damage. In the course of the work it was also found that PA is vulnerable to ravelling at hot weather conditions. At hot conditions, however, the increase in ravelling sensitivity is far less aggressive.

From these knowledge suggestions for the improvement of the ravelling performance of PA mixtures are made. It is believed that this knowledge may be beneficial for the successful introduction of silent PA in countries that exhibit a continental climate in which summers are hot and winters are cold.

Introduction

The Netherlands is a densely populated country. As a result environmental issues related to traffic are taken very serious. Two issues are especially addressed in the Netherlands, i.e. air pollution and traffic noise.

With respect to air pollution focus is on fine dust and CO₂ emissions. The Dutch government is encouraging car owners to buy cars with limited emissions by implementation of tax-advantages. Also tax levels on cars with high emissions were increased by a law introduced February 1 2008. With respect to traffic noise hindrance the policy of the government is best reflected in the obligation to apply PA on the Dutch primary road network. At the moment close to 90% of the Dutch primary road network is surfaced with some type of noise reducing PA. Noise reductions of 3 dBA compared to dense asphalt concrete are easily achieved by application of standard types of PA. Sandberg and Esjmont (2002) report that a 5 dBA noise reduction can be achieved by special types of PA, e.g. double layer PA where a fine 6/8 or 4/6 mm PA is placed over a courser PA sub-layer. Sandberg and Esjmont also report that surface layers with a porosity of at least 20% result in even larger noise reductions when made more elastic by application of at least 20% rubber or other elastic products. Such poroelastic surfacings with 40% porosity may result in noise reductions up to 12 dBA.

From the above it is concluded that porosity, apart from other aspects, is an important issue in noise reducing road surfacing materials. For that reason the Delft University of Technology developed a mechanistic design tool for PA. The tool aims to explain the ravelling performance of PA mixtures on the basis of the mixture volumetrics and the properties of raw materials. The tool is called LOT (Lifetime Optimisation Tool) and discussed elsewhere in the ENVIROAD 2009 proceedings (Huurman et al.; 2009) and not discussed further in this paper.

During the last Dutch winter aggressive ravelling developed in a number of short motorway sections, in cases also potholes formed rapidly. Although the combined affected road length remained limited emergency repairs and speed limitations temporary reduced network capacity. As a result the applicability of PA was questioned in a national discussion. Focus was on the cold weather performance of PA.
The Dutch winter of 2008-2009

Ravelling of Porous Asphalt concrete

During the last winter extremely aggressive ravelling developed at some short stretches of Dutch motorway. As a result traffic measures and emergency repairs were necessary. This resulted in a reduction of network capacity and triggered the press to publish negative articles, see Figure 1. Also national television broadcasted items with similar contents. A short but intensive national discussion about PA suitability and application followed.

Figure 1. Autoblog 23-01-2009: “Winter takes its toll: Dutch pavements damaged”, AD 29-01-09: “News: Frost damage runs into millions”, AD 13-01-09: “Hundreds of claims after frost damage”

The Centre for Transport and Navigation of the of the Dutch Ministry of Transport, Public Works and Water Management completed an inventory of the winter damage on January 22 2009 (Voskuilen, 2009), see Table 1. It is stated that only 2 out of the 55 affected sections did not have a PA surfacing. Furthermore it is stated that the predominant type of damage was ravelling, however, other types of damage were also observed.

Table 1. Summary of winter damage inventory as per January 21 2009 (Voskuilen, 2009).

<table>
<thead>
<tr>
<th>Number of damaged sections [-]</th>
<th>Combined length of damaged sections</th>
<th>Minimum section service life [years]</th>
<th>Maximum section service life [years]</th>
<th>Average section service life [years]</th>
</tr>
</thead>
<tbody>
<tr>
<td>55</td>
<td>&lt; 0.2% of total</td>
<td>5</td>
<td>18</td>
<td>11-12</td>
</tr>
</tbody>
</table>
Weather conditions

Figure 2. According to literature (KNMI, 2009) the Dutch winter of 2008/2009 was the coldest in 12 years.

Figure 2 gives an impression of temperatures registered in the municipality of de Bilt (centre of the Netherlands). The plot gives the 2 week moving average of the daily minimum surface temperature and the daily average temperature. As indicated by Figure 2 the previous Dutch winter was the coldest in a 12 year period. In more detail temperature data of previous winter is given for two locations in Figure 3.

Definition of a representative case

Introduction

The primary network in the Netherlands is for 90% surfaced with PA. However differences in the pavement structure, sub grade and even the type of PA (grading, type of bitumen, single layer or double layer) differ throughout the network. Similarly
traffic conditions vary. It was decided to obtain insight into the effects of winter by considering a situation that is representative for the Dutch primary network. Hereafter this representative case is further defined.

**Standard load**

The LOT simulations discussed later all consider a 50 kN wheel load applied by a Good Year 425R65 super single tyre. The width of the wheel patch of this commercial tyre equals 330 mm. It was assumed that the length of the wheel patch will equal 170 mm, so leading to an average contact stress of 0.891 MPa. The tyre travels at a speed of 21.25 m/s or 76.5 km/h, so that it requires exactly 8 ms for the tyre to pass over a certain point. In the simulations it is assumed that the tyre is non driven, i.e. free rolling.

**Representative traffic**

In the Netherlands the total traffic load on motorways varies from approximately 30,000 to 200,000 vehicles per day in two directions. Of this traffic approximately 12.5% is commercial; on average each commercial truck introduces 1.6 times the damage introduced by a standard 100 kN axis (DVS, 1998). From this it is concluded that the slow lanes in the Netherlands are subjected to 3,000 to 20,000 equivalent 100kN axle loads per day. A value of 10,000 is used in the simulations as a practical and representative number. From Buiter et al. (1989) it is known that commercial traffic on 3.5 m wide motorway lanes show lateral wander with a 290 mm standard deviation. For 330 mm wide super single tyres it is concluded that a 10 mm strip in the centre of the wheel path exists that is loaded by 41.9% of passing tyres. The simulations consider the situation in that strip.

**Representative pavement and deflections hereof**

The simulations all consider a representative Dutch motorway asphalt pavement. In Table 2 the thickness design of the considered structure is presented. The stiffness’s listed in Table 2 are based on the Dutch design method for Asphalt pavements on motorways, DVS (1998). The stiffness assigned to Dense Asphalt Concrete, DAC, is obtained from Figure 4. PA was assigned half that stiffness. This was done in accordance with the design method that prescribes that only 80% of the thickness of PA should be considered in pavement design. Since flexural stiffness depends on the product of thickness $t^3 \times x$ stiffness this leads to the conclusion that the stiffness of PA equals approximately half the stiffness of DAC according to the design standard. The unbound granular base, often a mixture of crushed concrete and crushed masonry, and the sand sub base are assigned generally accepted stiffness values.
Figure 4. Stiffness of dense asphalt concrete (vertical axis) as a function of temperature (horizontal axis) as per Dutch design method DVS (1998). Please note that the stiffness curve is extrapolated from 0 °C to -10 °C to obtain the stiffness at -10 °C.

Table 2. Representative structural design of a Dutch motorway pavement

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness [mm]</th>
<th>Poisson’s ratio</th>
<th>Stiffness [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>PA</td>
<td>50</td>
<td>0.35</td>
<td>10475 8625 6000 3750</td>
</tr>
<tr>
<td>DAC</td>
<td>200</td>
<td>0.35</td>
<td>20950 17250 12000 7500</td>
</tr>
<tr>
<td>Unbound base</td>
<td>225</td>
<td>0.4</td>
<td>400</td>
</tr>
<tr>
<td>Sand sub base</td>
<td>1000</td>
<td>0.4</td>
<td>100</td>
</tr>
<tr>
<td>Subgrade</td>
<td>∞</td>
<td>0.4</td>
<td>55</td>
</tr>
</tbody>
</table>

As the size of the LOT structural mixture model (Huurman et al., 2009) is limited, deflections of the pavement structure as a whole are fed to the model by prescribed deformations at the model boundaries. For the pavement listed in Table 2 deflections were computed up to 2000 mm away from the load centre at 5 mm and 27.5 mm depth. Use was made of WESLEA, a well known software tool for Linear Elastic Multi Layer Analyses. Computations were made for temperatures of -10°C, 0°C, +10°C and +20°C. Deflections at 15000 were considered to be nil and an exponential function was applied to describe deflections further than 2000 mm away from the load centre. The deflections shown in figure 5 were obtained.

Figure 5. Deflections 5 mm below the pavement surface as a function of asphalt temperature

Representative PA mixture
The most commonly applied type of PA in the Netherlands is a PA 0/16 mm. The mixture recipe of such mixtures was obtained from the Dutch National Standard, RAW (CROW, 2005). In the simulations to be discussed use is made of the LOT idealised 2D model. Table 3 lists the main LOT inputs that determine the structural geometry of that mixture. The true mixture geometry (known only when a mixture is available and can be photographed or scanned) is idealised by consideration of the mixtures recipe and void ratio. On the basis of the mineral grading the equivalent grain size is determined. The mortar film thickness surrounding stone particles and the distance between stones is determined on the basis of volumetric considerations. For a standard Dutch PA 0/16 mm the above translates into inputs listed in Table 3.

**Table 3. Definition of mixture geometry**

<table>
<thead>
<tr>
<th>Equivalent stone radius [mm]</th>
<th>4.8</th>
<th>Mineral in mortar density [kg/m³]</th>
<th>2650</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stone density [kg/m³]</td>
<td>2650</td>
<td>Bitumen density [kg/m³]</td>
<td>1020</td>
</tr>
<tr>
<td>Percentage of stone in mineral (wt %)</td>
<td>80%</td>
<td>Bitumen percentage (wt%)</td>
<td>4.5%</td>
</tr>
<tr>
<td>Void ratio</td>
<td>20%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Material component behaviour**

For LOT simulations insight into the mechanical behaviour of material components is required. In another ENVIROAD paper (Huurman et al., 2009) LOT is discussed in more detail. In this paper it is also indicated that the response and fatigue properties of an SBS modified mortar are determined on the basis of extensive laboratory testing. In the simulations discussed later the properties of aged SBS modified mortar were used. In the simulations the mortar response behaviour is described by the well known Prony Series model, (equation 1).

For aged mortar the response parameters listed in Table 4 were determined on the basis of available data. At the bottom of that table the stiffness parameters of the adhesive zones are given, see Huurman et al. (2009).

\[
E(t) = E_0 \cdot \left(1 - \sum_{i=1}^{n} \alpha_i \left(1 - \frac{t}{\tau_i}\right)^{i-1}\right) 
\]

(1)

Where:

- \(E(t)\) = stiffness as function of time [MPa];
- \(E_0\) = instantaneous stiffness [MPa];
- \(\alpha_i\) = stiffness reduction parameter [-];
- \(\tau_i\) = time constant [s]; \(t\) = time [s].

**Table 4. Prony response parameters and adhesive zone stiffness**

<table>
<thead>
<tr>
<th></th>
<th>-10°C</th>
<th>0°C</th>
<th>+10°C</th>
<th>+20°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>i</td>
<td>E0 [MPa]</td>
<td>αi [-]</td>
<td>τi [-]</td>
<td>τi [-]</td>
</tr>
<tr>
<td>1</td>
<td>0.248513</td>
<td>0.00825</td>
<td>0.00023</td>
<td>7.8E-06</td>
</tr>
<tr>
<td>2</td>
<td>0.041772</td>
<td>0.04063</td>
<td>0.00113</td>
<td>3.9E-05</td>
</tr>
<tr>
<td>3</td>
<td>0.089259</td>
<td>0.20005</td>
<td>0.00558</td>
<td>0.00019</td>
</tr>
<tr>
<td>4</td>
<td>0.182293</td>
<td>0.98504</td>
<td>0.02749</td>
<td>0.00094</td>
</tr>
<tr>
<td>5</td>
<td>0.15219</td>
<td>4.85024</td>
<td>0.13538</td>
<td>0.00461</td>
</tr>
<tr>
<td>6</td>
<td>0.115648</td>
<td>23.882</td>
<td>3.28227</td>
<td>0.11168</td>
</tr>
<tr>
<td>7</td>
<td>0.074913</td>
<td>117.592</td>
<td>3.28227</td>
<td>0.11168</td>
</tr>
</tbody>
</table>
Apart from mortar response also insight into mortar fatigue is required. The fatigue model applied in LOT is given by equation 2.

$$N_f = \left(\frac{W_\phi}{W_{\text{init}}'\text{initial}}\right)^n$$

Where:

- $n =$ material constant [-]
- $W_\phi =$ reference energy [MPa]
- $W_{\text{init}}'\text{initial}$ = dissipated energy per cycle in initial phase [MPa]

Based on available data (Huurman, 2009) the following parameters were determined. As indicated no information is available for mortar fatigue at temperatures of -10°C and +20°C.

**Table 5. Fatigue parameters for LTA mortar**

<table>
<thead>
<tr>
<th>Temperature</th>
<th>$W_\phi$ [MPa]</th>
<th>$n$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0°C</td>
<td>0.981</td>
<td>2.643</td>
</tr>
<tr>
<td>+10°C</td>
<td>0.363</td>
<td>2.986</td>
</tr>
</tbody>
</table>

As discussed elsewhere (Huurman, 2009) the adhesive zone that bonds the stone particles to the mortar may also fail due to damage accumulation. For the development of damage in the adhesive zone the following model is applied in LOT.

$$\dot{D} = \left(\frac{\sigma_{et}}{\sigma_0}\right)^n \text{ for } \sigma_{et} > 0, \quad \dot{D} = 0 \text{ for } \sigma_{et} \leq 0 \quad \text{with} \quad \sigma_{et} = \sigma_n + \tau / \tan \phi$$

Where:

- $\dot{D} =$ rate of damage accumulation [-/s]
- $\sigma_{et} =$ equivalent tensile stress, i.e. tensile stress in the case of zero shear [MPa]
- $\sigma_n =$ adhesive zone normal stress [MPa]
- $\tau =$ adhesive zone shear stress [MPa]
- $\phi =$ friction angle [degr.]
- $n_0 =$ model parameter [-]
- $\sigma_0 =$ reference stress [MPa]

Adhesive zone laboratory tests are done on both Greywacke and Sandstone stone columns. Here interest is in the behaviour of some representative Dutch motorway pavement and for that reason computations are made on the average fatigue behaviour of adhesive zones on Greywacke and Sandstone. Based on available data the following parameters are found.

**Table 6. Damage accumulation parameters for adhesive zones (combined Greywacke and Sandstone data)**

<table>
<thead>
<tr>
<th>Temperature [°C]</th>
<th>$\sigma_n$ [MPa]</th>
<th>$n$ [-]</th>
<th>$\phi$ [degrees]</th>
</tr>
</thead>
<tbody>
<tr>
<td>-10</td>
<td>20.63</td>
<td>2.74</td>
<td>70.2</td>
</tr>
</tbody>
</table>
LOT simulations

Introduction
PA at the road surface may experience three main types of mechanical loading.
- Forces introduced to the surface stones by passing tyres,
- Deformations that follows from deflection of the pavement as a whole,
- Stress that may be introduced as a result of temperature fluctuations.

In the simulations discussed here all of the above load cases are considered. Further discussion follows hereafter.

Wheel load forces
For the load signals that follow from individual wheel loads reference is made to another ENVIROAD paper (Huurman, 2009). Details of the representative load are found in section 3.2.

Deflection
The deformations of the pavement as a whole are determined by Linear Elastic Multi Layer Analysis as discussed in section 3.4. Deflection bowls were determined at a depth of 5 and 27.5 mm. Interpretation of these deflection bowls gives insight into surface layer deformations as a function of wheel the load position. To incorporate the deflection of the structure as a whole these deformations (i.e. translations and rotations) are applied as boundary conditions to the outer edges of the LOT model.

Figure 6 gives a visual impression of the described principle. The figure gives an impression of the deflection bowl at -10°C in combination with four plots of the model. The model plots have been retrieved from the relevant simulation.

Temperature fluctuations
During the day temperatures fluctuate, see Figure 2 and 3. As a result of these fluctuations also the temperature of the pavement surface layer will vary. Fluctuations are especially high close to the road surface. As a result hereof stress may be introduced in pavement structures. An estimate of these stresses in the surfacing

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>12.18</td>
<td>5.14</td>
<td>25.1</td>
</tr>
<tr>
<td>+10</td>
<td>10.56</td>
<td>3.56</td>
<td>31.9</td>
</tr>
<tr>
<td>+20</td>
<td>8.35</td>
<td>3.17</td>
<td>35.5</td>
</tr>
</tbody>
</table>
layers is made by assuming sinusoidal temperature evaluation over a 24 hour period. The following linear thermal expansion coefficients are applied.

**Table 7. Linear thermal expansion coefficients for stone and mortar**

<table>
<thead>
<tr>
<th></th>
<th>Stone (Sandstone/Greywacke)</th>
<th>Mortar</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha_L$</td>
<td>$6.6 \times 10^{-6}/^\circ C$</td>
<td>$2.5 \times 10^{-5}/^\circ C$</td>
</tr>
</tbody>
</table>

For the calculation of thermal stress a model in which the stones are represented as physical bodies, with an E-modulus of 50,000 MPa and a Poisson’s ratio of 0.25, was developed, see Figure 7. Calculations are made using the Visco-Elastic (VE) behaviour (Table 4) at -10°C, 0°C, 10°C and 20°C. Each calculation considered a 24 hour sinusoidal temperature signal with a 5°C amplitude. The average temperature, i.e. the off-set of the sinus, was made equal to the temperature for which the chosen VE properties are valid. Figure 8 gives an impression of obtained results. As indicated especially the horizontal contacts are stressed. Figure 9 gives obtained results in the form of charts. Both charts refer to horizontal contacts.
Figure 9. Obtained results for sinusoidal 10°C temperature change over a 24 hour period at average temperatures of -10, 0, +10 and +20 °C respectively. Left: adhesive zone normal stress in horizontal contacts, Right: Hysteresis loops in horizontal contacts.

Interpretation of LOT simulations

Results

The simulations discussed in section 4 result in stress and strain signals at different locations throughout the mixture. Stress and strain signals as a result of wheel load passages (combined surface loading and deflection) and as a result of temperature fluctuations are available. Application of the damage models discussed in section 3.6 on the combined signals of wheel load passages and temperature fluctuations allows determining the damage that accumulates in a 24 hour period. In combining the two available stress and strain signals it was assumed that 85% of the 10,000 daily axle load repetitions are applied between 7:00 and 19:00 hour. During the night, 19:00 to 7:00 hour, traffic load is 15% of the daily load.

Response simulations have been made for average temperatures of -10, 0, +10 and +20°C. Interpretation of the in-mixture response signals showed that the mixture is most vulnerable to failure of the adhesive zones, i.e. mortar fatigue leads to a longer ravelling life span than adhesive zone damage. This conclusion is drawn for 0°C and 10°C only because at these temperatures ample mortar fatigue data is available.

The simulations showed that the effects of temperature fluctuations over the day are limited for average temperatures of 0, +10 and +20°C (i.e. the material parameters used are valid for these temperatures). The maximum ravelling performance of the mixture is found at 0°C. Figure 10 gives an impression of the damage accumulation in a single 24 hour day as a function of average temperature and the magnitude of temperature fluctuation over that day. The figure plots the relative daily damage as compared to the daily damage introduced at 0°C. The figure clearly indicates that the ravelling performance of the mixture degrades as temperatures increase. However, as temperatures fall the mixture performance degrades much faster and aggressive, especially when temperature fluctuations of some magnitude occur.

Figure 10. Relative daily damage compared to daily damage at 0°C, i.e. maximum mixture performance, as a function of average daily temperature and daily temperature fluctuations.
In Figures 3 an indication of the air temperatures during the period in which aggressive ravelling damage developed is given for two locations in the Netherlands. Table 8 lists the extremes for both locations. Of course there is no direct relation between surfacing temperature and air temperature. However it is fair to say that circumstances at locations in the Netherlands can be represented by an average temperature of -10°C combined with a temperature fluctuation of e.g. 13°C. It is anticipated, but unknown to the authors, that more extreme circumstances developed locally due to micro climate conditions.

Table 8. Extremes in temperature data at two locations in the Netherlands.

<table>
<thead>
<tr>
<th>Location</th>
<th>Date</th>
<th>T_min (°C)</th>
<th>T_max (°C)</th>
<th>T_average (°C)</th>
<th>δT (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eindhoven</td>
<td>06-01-2009</td>
<td>-18.2</td>
<td>-5.3</td>
<td>-11.75</td>
<td>12.9</td>
</tr>
<tr>
<td></td>
<td>07-01-09</td>
<td>-17.8</td>
<td>-1.1</td>
<td>-9.45</td>
<td>16.7</td>
</tr>
<tr>
<td>De Bilt</td>
<td>10-01-2009</td>
<td>-10.5</td>
<td>-3.6</td>
<td>-7.05</td>
<td>6.9</td>
</tr>
<tr>
<td></td>
<td>03-01-2009</td>
<td>-8.9</td>
<td>1.6</td>
<td>-3.65</td>
<td>10.5</td>
</tr>
</tbody>
</table>

From Figure 10 it is concluded that the accumulation of adhesive ravelling damage at $T_{\text{average}}/\delta T = -10^\circ\text{C}/13^\circ\text{C}$ is close to 20,000 times faster than at maximum mixture performance, i.e. 0°C. This indicates that ravelling damage accumulated during the most cold Dutch winter days may easily exceed the damage accumulated in years of less extreme conditions.

On the basis of the above it is believed that LOT explains the very aggressive and extreme ravelling damage that developed at locations during the last Dutch winter. The authors cannot put enough emphasis on this, because implications of this conclusion are that the Centre for Transport and Navigation of the Dutch Ministry of Transport, Public Works and Water Management has the availability of a tool allowing for PA mixture design.

### Causes of winter damage

An explanation of the trends plotted in Figure 10 was found in the observation that a Porous Asphalt surfacing layer is subjected to two types of loadings.

1. **Strain controlled loadings.**
   First temperature fluctuations result in strain controlled loadings. Due to temperature fluctuations the material wants to shrink or expand. The desired strains that follow are countered by opposite strains that result in stresses. These effects are independent of surfacing stiffness and result in a strain controlled type of loading.
   Secondly the pavement deflects under loading. These deflections of course depend on the structural pavement design and the traffic load. However, the contribution of the surfacing layer to structural stiffness is very limited. In other words the surfacing layer cannot limit pavement deflections, even when the surfacing material becomes very stiff. As such, for the surfacing layer, pavement deflections result in a type of loading that is best described as strain controlled.

2. **Force controlled loadings.**
At locations where a passing tyre makes contact with the surfacing layer and equilibrium between applied contact forces and surface reaction forces exists. This type of loading is thus mainly force controlled.

The relaxation behaviour of bituminous mortars degrades as temperatures decrease. For the aged SBS modified bitumen considered in this work this phenomenon resulted in a strong increase of temperature stresses as temperatures drop to -10°C. Adhesive zones poses temperature dependent behaviour. At some temperature the performance of these zones is maximal. With increasing and decreasing temperatures this performance degrades, see Figure 11. For the bitumen considered here and considering the average data obtained for Sandstone and Greywacke the maximum adhesive zone performance is obtained at 0°C.

![Figure 11. Adhesive zone damage rate as a function of temperature and tensile stress.](image)

As shown by Figure 11 the adhesive zone performance at -10°C is about equal to the performance at +10°C. Figure 10, however, indicates that the ravelling performance of the mixture at +10°C is far better than the mixture performance at -10°C in the case that there are no temperature fluctuations. In this case temperature stresses remain absent and observed differences follow from increasing stresses that find their cause in pavement deflection, see #1 in Figure 10. At -10°C the mortar has stiffened so much that an increase in deflection stresses become of importance. Due to the limited relaxation behaviour temperature stresses may develop at an average temperature of -10°C. Due to these stresses the damage development during periods of horizontal compressive temperature stress is reduced. However in periods of horizontal tensile temperature stress the damage accumulation is increased. Depending on the distribution of traffic over the day this results in a decrease of damage at low temperature fluctuations (damage reduction in periods of compression compensates for damage increase during periods of tension). As the temperature fluctuations are of ample magnitude only negative effects can occur. (damage increase in periods of tension is of such magnitude that it cannot be compensated by damage reduction during periods of compression). See #2 in Figure 10. The relaxation behaviour at temperatures above 10°C is such that temperature stresses of magnitude do not develop. Also the mortar stiffness at these temperatures is low, which prevents the development of larger deflection stress. At these circumstances the surface of the PA layer is solemnly subjected to the force controlled loading
introduced by passing tyres. Now the strength of the fatigue material becomes important. As indicated by Figure 11 the strength of adhesive zones degrades at higher temperature so explaining the degradation of mixture ravelling performance at 20°C, see #3 in Figure 10.

Discussion, conclusions, recommendations

Discussion

Comments can be made with respect to the work discussed in this paper. For instance in reality the behaviour of mortar will vary during temperature fluctuation. Here the behaviour of mortar was related to the average temperature and did not further vary during temperature fluctuations. Also the assumed distribution of traffic over the day may be argued. Or the fact that the paper discusses a generalised case that is representative for the Netherlands whereas discussion of a particular pinpointed case may be preferred.

Despite these possible discussions the authors want to challenge the reader to see the broader scope. We are not discussing a particular case, but our aim is to further optimise and develop a mechanistic mixture performance design tool. Explaining the development of winter damage on network level is considered an important step in achieving that goal.

Conclusions

* It was shown that LOT is able to explain the aggressive and extreme ravelling damage as developed during last winter at locations in the Netherlands.
* The previous indicates that LOT is a tool capable of explaining low temperature mixture performance, i.e. -10°C, combined with validation tests at 10°C (Huurman et al., 2009) this is a first indication of the validity of LOT over a range of temperatures.
* It was shown that cause of winter damage is found mainly in a strong reduction of the relaxation potential of the aged mortar at low temperatures. The reduction of adhesive zone performance at low temperatures is enhancing these effects.
* It was shown that the ravelling performance of the mixture at high temperatures degrades as a result of adhesive zone strength reduction.
* The successful introduction of PA in areas with a continental climate will depend on the availability of a mortar that remains viscous at low temperatures with good (adhesion) strength at high temperatures.

Recommendations

The work discussed in this paper indicates that the performance of PA can be improved significantly when a mortar is applied that remains flexible at low temperatures, even after aging. Application of such a mortar will especially address low temperature performance.

Further improvements of ravelling performance may be obtained when the adhesive zone strength at high temperatures is addressed. It is expected that the benefits hereof remain limited compared to the benefits that follow from the above recommendation. This is especially so since the effects of healing, which is especially strong at high temperatures, was not considered here.

Indications are that mortars may be made more flexible at low temperatures by focussed polymer modification (e.g. ABS) or the application of pulverized rubber
particles (Garcia-Morales et. Al 2006). Increasing the adhesive zone strength may be achieved by selection of highly potential combinations of stone mineral composition, bitumen and fillers.

Given the results indications are that significant improvements in mixture ravelling performance can be achieved by implementation of the above recommendations.

This work showed the importance of the response behaviour of mortar in explaining the performance of asphalt concrete mixtures. It is strongly recommended to strive for better constitutive models for mortar. It is believed that the models available today (including the Prony series model used here) can be improved. Models that are stress dependent and able to accurately describe the response behaviour over a wide range of frequencies and temperatures are required for future meso scale mechanistic mixture designs.

References

CROW (2005); Standard Provisions RAW, manual for specifications in soil, hydraulic and pavement engineering (in Dutch: Standaard RAW Bepalingen, handboek voor bestekken in de grond-, water- en wegenbouw), ede, the Netherlands


Huurman M., Mol L., Woldekidan M.F. (2009); Mechanistic design of silent asphalt mixtures, Enviroad, Warsaw, Poland

KNMI, Royal Netherlands Metrological Institute (2009); temperature data publicly accessible via website: knmi.nl, de Bilt, the Netherlands.


Input used for calculations of pavement deflections for the different sections

**Appendix 2**

**Table 1: Input used for WESLEA, A15_2006**

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (mm)</th>
<th>Poisson’s ratio</th>
<th>Stiffness (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>-10 °C</td>
</tr>
<tr>
<td>PA</td>
<td>41</td>
<td>0.35</td>
<td>10475</td>
</tr>
<tr>
<td>STAC</td>
<td>333</td>
<td>0.35</td>
<td>20950</td>
</tr>
<tr>
<td>Sand sub base</td>
<td>1000</td>
<td>0.4</td>
<td>20950</td>
</tr>
<tr>
<td>Subgrade (clay)</td>
<td>∞</td>
<td>0.4</td>
<td>100</td>
</tr>
</tbody>
</table>

**Table 2: Input used for WESLEA, N3_2004**

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (mm)</th>
<th>Poisson’s ratio</th>
<th>Stiffness (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>-10 °C</td>
</tr>
<tr>
<td>PA</td>
<td>45</td>
<td>0.35</td>
<td>10475</td>
</tr>
<tr>
<td>STAC</td>
<td>211</td>
<td>0.35</td>
<td>20950</td>
</tr>
<tr>
<td>Blast furnace slag</td>
<td>350</td>
<td>0.4</td>
<td>100</td>
</tr>
<tr>
<td>Sand</td>
<td>700</td>
<td>0.4</td>
<td>100</td>
</tr>
<tr>
<td>Subgrade (clay)</td>
<td>∞</td>
<td>0.4</td>
<td>55</td>
</tr>
</tbody>
</table>

**Table 3: Input used for WESLEA, A200_2002**

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (mm)</th>
<th>Poisson’s ratio</th>
<th>Stiffness (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>-10 °C</td>
</tr>
<tr>
<td>PA</td>
<td>38</td>
<td>0.35</td>
<td>10475</td>
</tr>
<tr>
<td>STAC</td>
<td>337</td>
<td>0.35</td>
<td>20950</td>
</tr>
<tr>
<td>Sand sub base</td>
<td>1000</td>
<td>0.4</td>
<td>100</td>
</tr>
<tr>
<td>Subgrade (clay)</td>
<td>∞</td>
<td>0.4</td>
<td>55</td>
</tr>
</tbody>
</table>

**Table 9: Input used for WESLEA, N9_2002**

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (mm)</th>
<th>Poisson’s ratio</th>
<th>Stiffness (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>-10 °C</td>
</tr>
<tr>
<td>PA</td>
<td>50</td>
<td>0.35</td>
<td>10475</td>
</tr>
<tr>
<td>STAC</td>
<td>210</td>
<td>0.35</td>
<td>20950</td>
</tr>
<tr>
<td>Lava stone</td>
<td>250</td>
<td>0.4</td>
<td>150</td>
</tr>
<tr>
<td>Lava stone</td>
<td>250</td>
<td>0.4</td>
<td>100</td>
</tr>
<tr>
<td>Subgrade (sand)</td>
<td>∞</td>
<td></td>
<td>100</td>
</tr>
</tbody>
</table>
Input used for calculations of pavement deflections for the different sections

**Table 10: Input used for WESLEA, A4_1997**

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (mm)</th>
<th>Poisson’s ratio</th>
<th>Stiffness (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>-10 °C</td>
</tr>
<tr>
<td>PA</td>
<td>49</td>
<td>0.35</td>
<td>10475</td>
</tr>
<tr>
<td>STAC</td>
<td>254</td>
<td>0.35</td>
<td>20950</td>
</tr>
<tr>
<td>Concrete granulate</td>
<td>250</td>
<td>0.4</td>
<td>600</td>
</tr>
<tr>
<td>Sand</td>
<td>1000</td>
<td>0.4</td>
<td>100</td>
</tr>
<tr>
<td>Subgrade (clay)</td>
<td>∞</td>
<td>0.4</td>
<td>55</td>
</tr>
</tbody>
</table>

**Table 11: Input used for WESLEA, A15_1995**

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (mm)</th>
<th>Poisson’s ratio</th>
<th>Stiffness (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>-10 °C</td>
</tr>
<tr>
<td>PA</td>
<td>41</td>
<td>0.35</td>
<td>10475</td>
</tr>
<tr>
<td>STAC</td>
<td>213</td>
<td>0.35</td>
<td>20950</td>
</tr>
<tr>
<td>Sand cement</td>
<td>225</td>
<td>0.2</td>
<td>8000</td>
</tr>
<tr>
<td>Sand</td>
<td>1000</td>
<td>0.4</td>
<td>100</td>
</tr>
<tr>
<td>Subgrade (clay)</td>
<td>∞</td>
<td>0.4</td>
<td>55</td>
</tr>
</tbody>
</table>

**Table 12: Input used for WESLEA, A12_1992**

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (mm)</th>
<th>Poisson’s ratio</th>
<th>Stiffness (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>-10 °C</td>
</tr>
<tr>
<td>PA</td>
<td>49</td>
<td>0.35</td>
<td>10475</td>
</tr>
<tr>
<td>STAC</td>
<td>270</td>
<td>0.35</td>
<td>20950</td>
</tr>
<tr>
<td>Concrete</td>
<td>230</td>
<td>0.15</td>
<td>15000</td>
</tr>
<tr>
<td>Sand cement</td>
<td>150</td>
<td>0.2</td>
<td>8000</td>
</tr>
<tr>
<td>Subgrade (sand)</td>
<td>∞</td>
<td>0.4</td>
<td>100</td>
</tr>
</tbody>
</table>

**Table 13: Input used for WESLEA, A12_1987**

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (mm)</th>
<th>Poisson’s ratio</th>
<th>Stiffness (MPa)</th>
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</thead>
<tbody>
<tr>
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<td></td>
<td></td>
<td>-10 °C</td>
</tr>
<tr>
<td>PA</td>
<td>40</td>
<td>0.35</td>
<td>10475</td>
</tr>
<tr>
<td>STAC</td>
<td>363</td>
<td>0.35</td>
<td>20950</td>
</tr>
<tr>
<td>Sand</td>
<td>1000</td>
<td>0.4</td>
<td>100</td>
</tr>
<tr>
<td>Subgrade (clay)</td>
<td>∞</td>
<td>0.4</td>
<td>55</td>
</tr>
</tbody>
</table>
Mastercurve fitting of DSR data for the different sections

Appendix 3

A12_1987, Treference = 10ºC

A12_1992, Treference = 10ºC
Mastercurve fitting of DSR data for the different sections

A15_1995, $T_{\text{reference}} = 10^\circ\text{C}$

A4_1997, $T_{\text{reference}} = 10^\circ\text{C}$
Mastercurve fitting of DSR data for the different sections

A200_2002, $T_{\text{reference}} = 10^\circ C$

![Graph of A200_2002](image)

N9_2002 $T_{\text{reference}} = 10^\circ C$

![Graph of N9_2002](image)
Mastercurve fitting of DSR data for the different sections

N3_2004 T reference = 10°C

A15_2006 T reference = 10°C