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Porous Asphalt Ravelling in Cold Weather Conditions

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Abstract: For environmental reasons, the application of porous asphalt (PA) concrete 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 2008-2009 Dutch winter, temperatures dropped below -10°C nationwide 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. Lifetime Optimisation Tool (LOT) is a meso scale mechanistic mixture design tool for PA discussed elsewhere [1]. 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 this 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.

Key words*: Meso scale modelling; Porous asphalt; Ravelling; Winter damage.*

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 CO_2 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 [2] report that a 5dBA 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 12dBA.

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 and discussed elsewhere [2] and not discussed further in this paper.

During the 2008/2009 Dutch winter, aggressive ravelling developed in a number of short motorway sections, in some cases potholes also 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 motorways. 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 Fig. 1. Also national television broadcasted items with similar contents. A short but intensive national discussion about PA suitability and application followed.

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 [3] 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.

Weather Conditions

Fig. 2 gives an impression of temperatures registered in the municipality of de Bilt (centre of the Netherlands). The plot gives As indicated by Fig. 2, the 2008/2009 Dutch winter was the coldest

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Fig. 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".

Table 1. Summary of Winter Damage Inventory as Per January 21, 2009 [3].

Number of Damaged Sections (-)	Combined Length of Damaged Sections	Minimum Section Service Life (years)	Maximum Section Service Life (years)	Average Section Service Life (years)
55	< 0.2% of total network	5	18	11-12

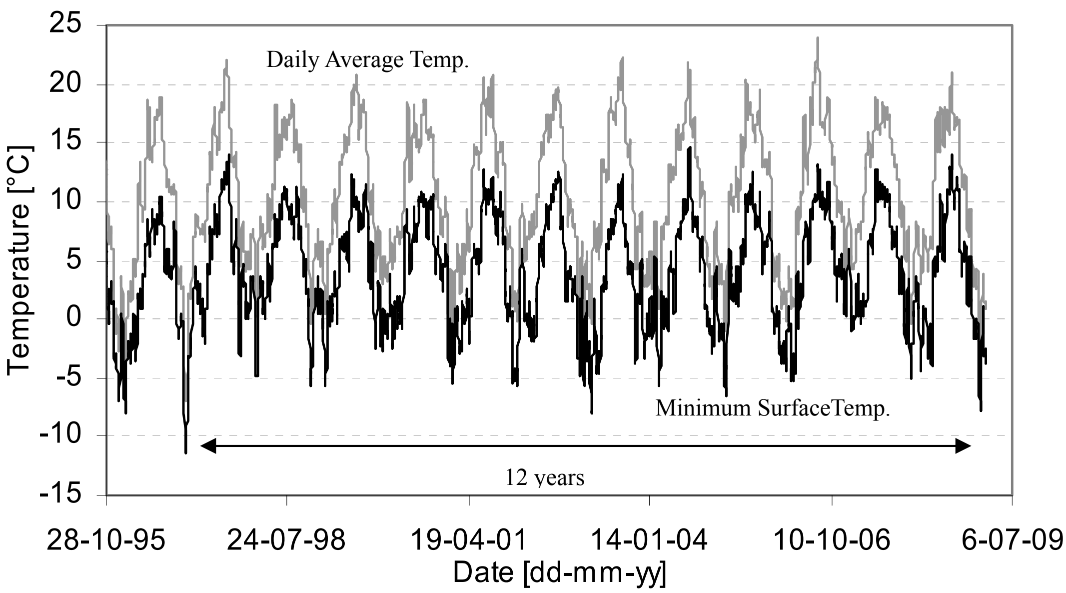


Fig. 2. The 2-Week Moving Average of the Daily Minimum Surface Temperature and the Daily Average Temperature.(According to the Royal Netherlands Metrological Institute (KNMI) [4]).

in a 12-year period. In more detail, temperature data of previous winters are given for two locations in Fig. 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, subgrade, and even the type of PA (grading, type of bitumen, single layer, or double

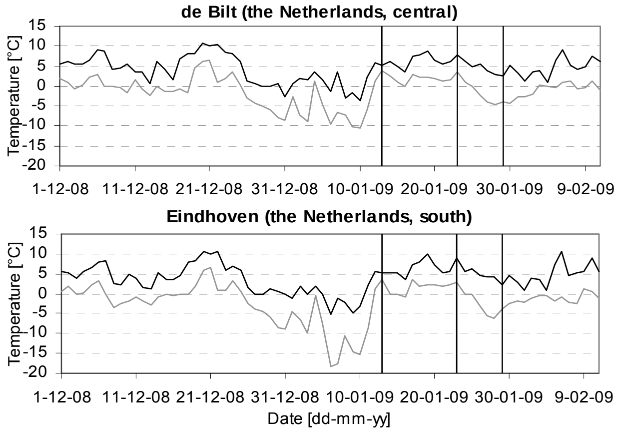


Fig. 3. Daily Maximum and Minimum Temperature in the Municipality of de Bilt and Eindhoven. (Vertical Lines: the Publishing Dates of Articles in Fig. 1).

Table 2. Representative Structural Design of a Dutch Motorway Pavement.

Material	Thickness (*mm*)	Poisson's Ratio (-)	Stiffness (*MPa*) -10°C	0°C	+10°C	20°C
PA	50	0.35	10475	8625	6000	3750
DAC	200	0.35	20950	17250	12000	7500
Unbound Base	225	0.4	400			
Sand Sub Base	1000	0.4	100			
Subgrade	∞	0.4	55			

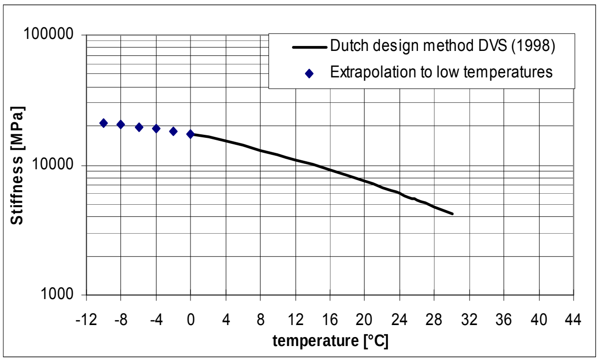


Fig. 4. Stiffness of Dense Asphalt Concrete (Vertical Axis) as a Function of Temperature (Horizontal Axis) as Per Dutch Design Method [6] and Extrapolation Hereof to Obtain the Stiffness at -10 °C.

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 tire [5]. The width of the wheel patch of this commercial tire equals 330*mm*. It was assumed that the length of the wheel patch will equal 170*mm*, leading to an average contact stress of 0.891*MPa*. The tire travels at a speed of 21.25*m/s* or 76.5*km/h*, so that it requires exactly 8ms for the tire to pass over a certain point.

In the simulations, it is assumed that the tire 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 [6]. From this it is concluded that the slow lanes in the Netherlands are subjected to 3,000 to 20,000 equivalent 100*kN* axle loads per day. A value of 10,000 is used in the simulations as a practical and representative number.

From Buiter et al. [7], 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 tires, it is concluded that a 10-mm strip in the center of the wheel path exists that is loaded by 41.9% of passing tires. 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 [6].

The stiffness assigned to dense asphalt concrete (DAC) is obtained from Fig. 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 3 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 subbase are assigned generally accepted stiffness values.

As the size of the LOT structural mixture model [1] 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 center at 5 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, 0, +10, and +20°C. Deflections at 15,000 were considered to be nil and an exponential function was applied to describe deflections further than 2000*mm* away from the load center. The deflections shown in Fig. 5 were obtained.

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 [8]. In the simulations to be discussed use is made of the LOT idealized 2D model. Table 3 lists

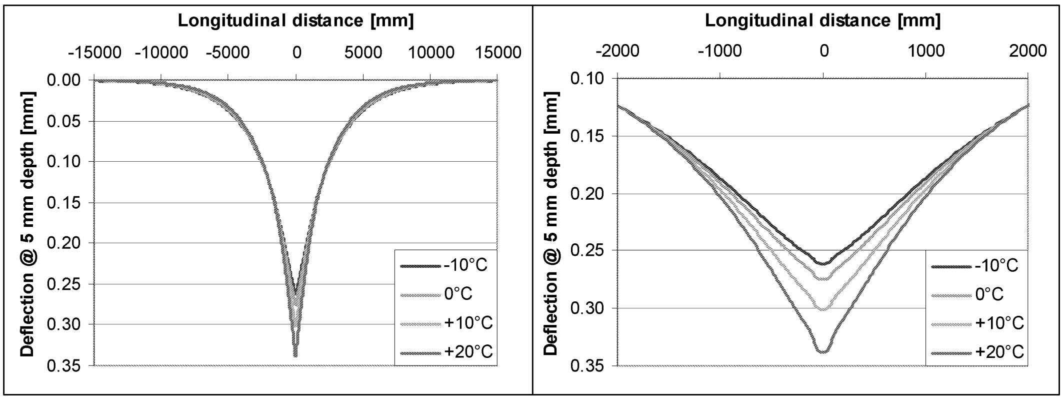


Fig. 5. Deflections 5*mm* Below the Pavement Surface as a Function of Asphalt Temperature.

Table 3. Definition of Mixture Geometry.

Equivalent Stone Radius (*mm*)	4.8	Mineral in Mortar Density (*kg/m³*)	2650
Stone Density (*kg/m³*)	2650	Bitumen Density (*kg/m³*)	1020
Percentage of Stone in Mineral (*wt* %)	80%	Bitumen Percentage (*wt*%)	4.5%
Void Ratio	20%		

Table 4. Prony Response Parameters and Adhesive Zone Stiffness.

		-10°C	0°C	+10°C	+20°C
	E0 [*MPa*]	5284	5284	5284	5284
i	ai (-)	ti (-)	ti (-)	ti (-)	ti (-)
1	0.248513	0.00825	0.00023	7.80E-06	3.40E-07
2	0.041772	0.04063	0.00113	3.90E-05	1.70E-06
3	0.089259	0.20005	0.00558	0.00019	8.10E-06
4	0.182293	0.98504	0.02749	0.00094	4.00E-05
5	0.15219	4.85024	0.13538	0.00461	0.0002
6	0.115648	23.882	0.6666	0.02268	0.00097
7	0.074913	117.592	3.28227	0.11168	0.00479
8	0.049458	579.01	16.1615	0.5499	0.02358
9	0.022485	2850.98	79.5775	2.70763	0.11612
10	0.010191	14037.9	391.83	13.3321	0.57175
11	0.00716	69120.8	1929.33	65.6455	2.81523
12	0.003601	340343	9499.77	323.231	13.8619
13	0.001616	1675809	46775.8	1591.55	68.2542
Adhesive Zone Stiffness	K_s (*MPa/mm*)*	114702	112935	75447	29923
	K_n (*MPa/mm*)*	332636	327512	218796	86778

* K_s and K_n : the stiffness of the adhesive zone where stone meets with mortar.

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 idealized 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.

Material Component Behavior

For LOT simulations, insight into the mechanical behavior of material components is required. Elsewhere [1] LOT is discussed in more detail. In this paper, it is also indicated that the response and fatigue properties of a Styrene-Butadiene-Styrene (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 behavior is described by the well known Prony Series model, Eq. 1.

For long-term 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 [1].

$$E(t) = E_0 \bullet \left(1 - \sum_{i=1}^{n} \alpha_i \left(1 - e^{\frac{-t}{\tau_i}}\right)\right) \tag{1}$$

where,
$E(t)$= stiffness as function of time (*MPa*),
E_0 = instantaneous stiffness (*MPa*],
α_i = stiffness reduction parameter (-),
τ_i = time constant (s), and
t = time (s).

Apart from mortar response, insight into mortar fatigue is also required. The fatigue model applied in LOT is given by Eq. (2).

$$N_f = \left(W_0 / W_{initial}\right)^n \tag{2}$$

where,
n = material constant (-),
W_0 = reference energy (*MPa*), and
$W_{initia\ l}$ = dissipated energy per cycle in initial phase (*MPa)*.

Based on available data [1], the following parameters were determined. As indicated, no information is available for mortar fatigue at temperatures of -10 and +20°C.

As discussed elsewhere [1], 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:

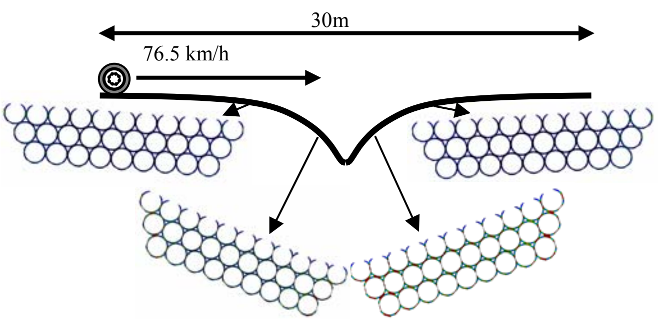


Fig. 6. Deflections of the Pavement as a Whole at -10°C and the Effects Hereof for PA Surfacing.

$$\dot{D} = \left(\frac{\sigma_{et}}{\sigma_0}\right)^{n_0} \text{ for } \sigma_{et} \rangle 0,\ \dot{D} = 0 \text{ for } \sigma_{et} \le 0 \text{ with } \sigma_{et} = \sigma_n + \tau \,{}^{\tau}\!/_{\tan\phi} \quad (3)$$

where,

$\dot{D}$ = rate of damage accumulation (-/s);

σ_{et}= equivalent tensile stress, i.e., tensile stress in the case of zero shear (*MPa)*;

σ_n = adhesive zone normal stress (*MPa*);

τ = adhesive zone shear stress (*MPa*);

ϕ = friction angle (*degr.*);

n_0= model parameter (-); and

σ_0= reference stress (*MPa*).

Adhesive zone laboratory tests are done on both Greywacke and Sandstone stone columns. Here interest is in the behavior of some representative Dutch motorway pavement and for that reason computations are made on the average fatigue behavior of adhesive zones on Greywacke and Sandstone. Based on available data, the parameters listed in Table 5 are found.

LOT Simulations

Introduction

PA at the road surface may experience three main types of mechanical loading:

- Forces introduced to the surface stones by passing tires,
- Deformations that follow from deflection of the pavement as a whole, and
- 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.

Table 5. Damage Accumulation Parameters for Adhesive Zones (Combined Greywacke and Sandstone Data).

Temperature (°C)	σ_0 (*MPa*)	n (-)	ϕ (degrees)
-10	20.63	2.74	70.2
0	12.18	5.14	25.1
+10	10.56	3.56	31.9
+20	8.35	3.17	35.5

Table 6. Linear Thermal Expansion Coefficients for Stone and Mortar.

	Stone (Sandstone/Greywacke)	Mortar
αL	6.6×10^{-6}/°C	2.5×10^{-5}/°C

Wheel Load Forces

The load signals acting on individual surface stones under the passage of a wheel load are described elsewhere [1]. Details of the representative load are found in section "Representative Traffic".

Deflection

The deformations of the pavement as a whole are determined by the linear elastic multi layer analysis as discussed earlier. Deflection bowls were determined at a depth of 5and 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.

Fig. 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 Figs. 2 and 3. As a result of these fluctuations, the temperature of the pavement surface layer

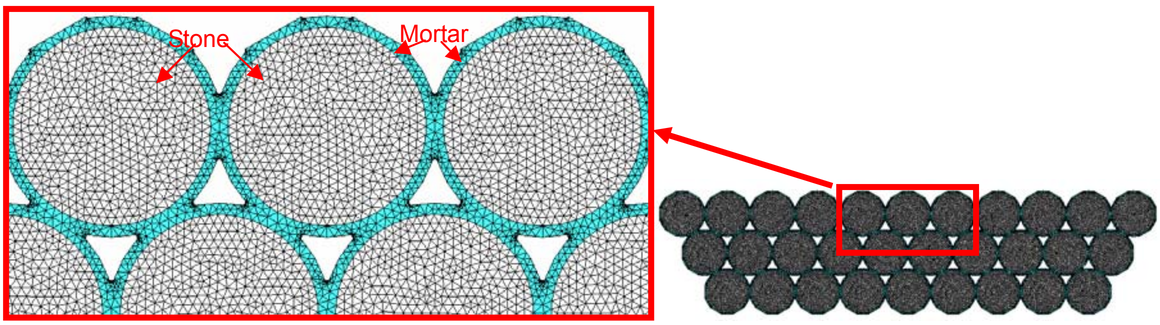


Fig. 7. Model for Determination of In-mixture Temperature Stress.

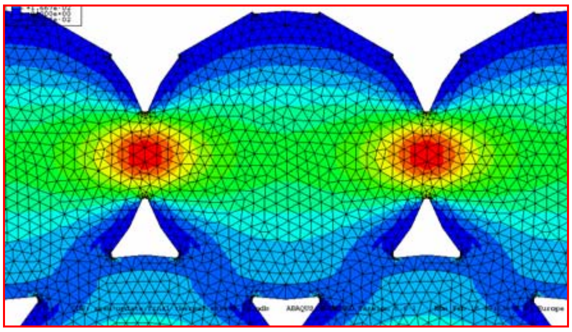


Fig. 8. Indication of Temperature Stress.

will also 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 layers is made by assuming a sinusoidal temperature evaluation over a 24-hour period. The linear thermal expansion coefficients listed in Table 6 are applied.

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 Poison's ratio of 0.25, was developed, see Fig. 7. Note that stones and mortar are modelled physically, each having their own linear thermal expansion coefficient. Calculations are made using the Visco-Elastic (VE) behavior (Table 4) at -10, 0, 10, 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. Fig. 8 gives an impression of obtained results. As indicated, the horizontal contacts are especially stressed. Fig. 9 gives obtained results in the form of charts. Both charts refer to horizontal contacts.

Interpretation of LOT Simulations

Results

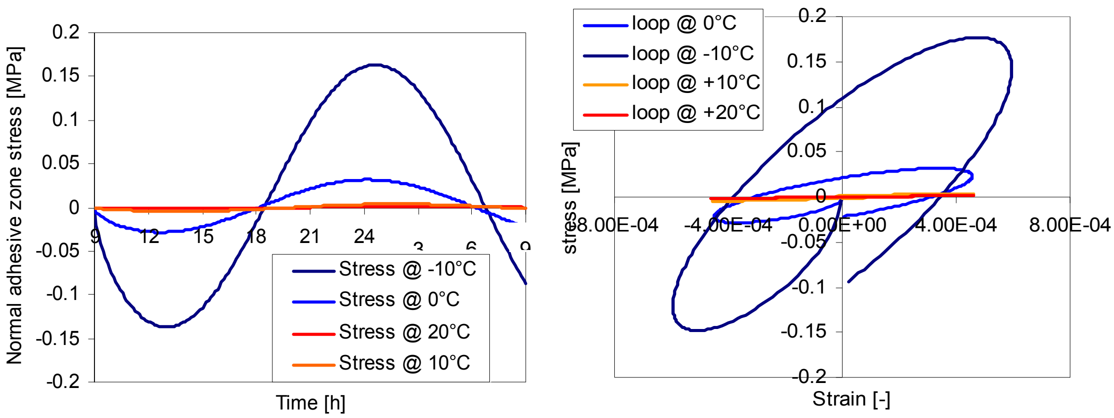
The simulations discussed earlier in "LOT simulations" result in stress and strain signals at different locations throughout the mixture. Stress and strain signals as a result of wheel load passages(combined surface loading and deflection) and temperature fluctuations are available. Application of the damage models discussed earlier 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 are 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. Fig. 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, i.e., the daily damage relative to the damage introduced in a single day 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.

Fig. 10 is explained later in section "Causes of Winter Damage". In explaining Fig. 10, distinction is made between the deflection effect (arrow 1), the temperature fluctuation effect (arrow 2), and the strength reduction effect (arrow 3).

In Fig. 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 7 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



Left: Adhesive Zone Normal Stress in Horizontal Contacts, Right: Hysteresis Loops in Horizontal Contacts.

Fig. 9. The Effects of Sinusoidal, 5°C Amplitude/24-Hour Period, Temperature Changes at Average Temperatures of -10, 0, +10, and +20°C, Respectively.

Table 7. Extremes in Temperature Data at Two Locations in the Netherlands.

Location	Date	T_{min} (°C)	T_{max} (°C)	$T_{average}$ (°C)	δT (°C)
Eindhoven	06-01-2009	-18.2	-5.3	-11.75	12.9
	07-01-09	-17.8	-1.1	-9.45	16.7
De Bilt	10-01-2009	-10.5	-3.6	-7.05	6.9
	03-01-2009	-8.9	1.6	-3.65	10.5

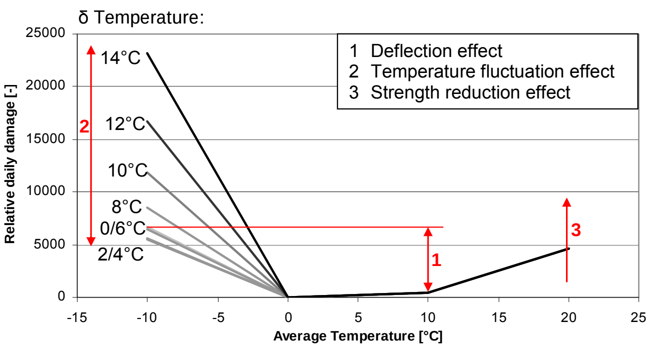


Fig. 10. Relative Daily Damage Compared to Daily Damage at 0°C, i.e., Maximum Mixture Performance as a Function of Average Daily Temperature and Daily Temperature Fluctuations.

temperature of -10°C combined with a temperature fluctuation of 13°C. It is anticipated, but unknown to the authors, that more extreme circumstances developed locally due to micro climate conditions.

From Fig. 10 it is concluded that the accumulation of adhesive ravelling damage at Taverage/δT = -10°C/13°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 Fig. 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 counter acted 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 tire 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 behavior 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 pose temperature dependent behavior. At some temperatures, the performance of these zones is maximal. With increasing and decreasing temperatures, this performance degrades, see Fig. 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.

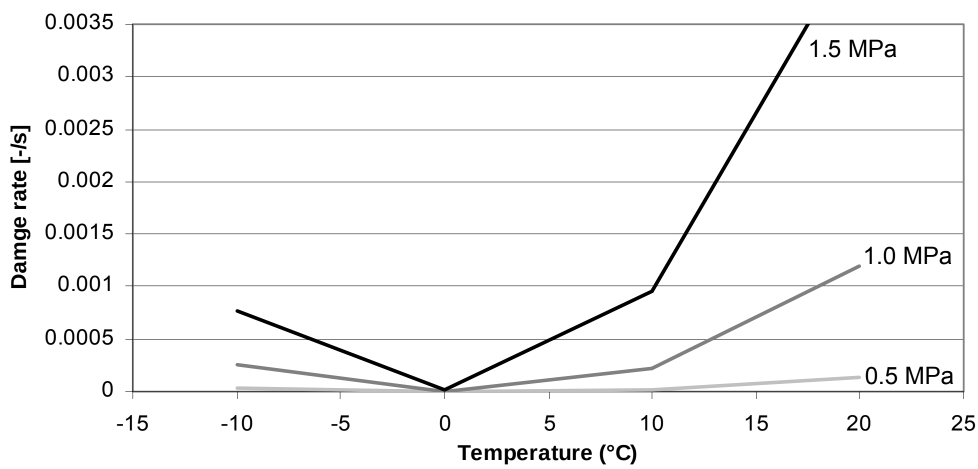


Fig. 11. Adhesive Zone Damage Rate as a Function of Temperature and Tensile Stress.

about equal to the performance at +10°C. Fig. 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 Fig. 10. At -10°C the mortar has stiffened so much that an increase in deflection stresses become of importance.

Due to the limited relaxation behavior 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 Fig. 10.

The relaxation behavior 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 tires. Now the strength of the fatigue material becomes important. As indicated by Fig. 11, the strength of adhesive zones degrades at higher temperatures, explaining the degradation of mixture ravelling performance at 20°C, see #3 in Fig. 10.

Discussion, Conclusions, and Recommendations

Discussion

Comments can be made with respect to the work discussed in this paper. For instance, in reality the mechanical behavior of mortar will vary during temperature fluctuation. Here the behavior 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 [1]; 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 polymer modification [e.g., Acrylonitrile-Butadiene-Styrene (ABS)] or the application of pulverized rubber particles [9]. Increasing the adhesive zone strength may be achieved by the selection of highly potential combinations of stone mineral composition, bitumen and fillers.

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

This work showed the importance of the response behavior 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 behavior over a wide range of frequencies and temperatures are required for future meso scale mechanistic mixture designs.

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