Geosynthetics: Applications, Design and Construction, De Groot, Den Hoedt & Termaat (eds)
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Case study of a flexible pavement structure with the EPS geofoam sub-base

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ABSTRACT: Asphalt strain measurements and surface deflection measurements are described that were performed on the Matlingeweg in Rotterdam. The considered pavement structure was of interest for investigation because of its sub-base, which consists of a 1.0 m thick EPS layer, combined with a heavy traffic loading. The measurements were carried out by means of the Falling Weight Deflectometer (FWD) and four strain transducers built-in at the bottom of the asphalt layer. Overlaying of the pavement structure has taken place much earlier than it was originally planned because severe cracking occurred at the pavement's surface within a few weeks after reconstruction. The temperature dependent behavior of the asphalt layer disables a direct comparison between the measured strain values. It implicates that those values have to be translated to a reference temperature before comparison. The back-calculated E-values in the pavement structure layers were used to translate the measured asphalt strain values to a reference temperature and to present the trend of the translated strain values as a function of the pavement structure age. The conclusions and recommendations regard the pavement condition in general (after 3 years in service) and the elasticity moduli of the pavement layers in particular.

1. INTRODUCTION

In this article surface deflection measurements and asphalt strain measurements are described that were performed on the Matlingeweg in Rotterdam, The Netherlands, which sub-base consists of a 1.0 m thick EPS layer. This is a connecting road between two heavily trafficked motorways and it crosses an industrial estate under construction. Accordingly, a large amount of heavy trucks uses the Matlingeweg.

The settlement observations pointed to a varying subsoil bearing capacity along the road alignment. The EPS sub-base was built-in on two sections of this road where extreme local settlements were observed before the road reconstruction.

The measurements were carried out by means of the Falling Weight Deflectometer (FWD) and strain transducers respectively. Four transducers were built-in at the bottom of the asphalt layer for this purpose.

The FWD measurements were performed to enable the back-calculation of the modulus of elasticity of the individual pavement layers. The layer moduli give an impression about the actual bearing capacity of the pavement structure.

The value of the horizontal tensile strain at the bottom of the asphalt layer is a dominant criterion in pavement design. The value of the asphalt strain increases in time due to the damaging effect of the traffic loading. Accordingly, information about the development of the horizontal asphalt strain in a pavement structure permits the analysis of its structural behaviour.

2. MEASURING PROGRAM

In order to get an insight into the structural behaviour of flexible pavement structures with an EPS sub-base the asphalt strains under FWD loading have been measured by the means of the built-in transducers about three times a year. Simultaneously, FWD tests have been performed. The first FWD measurements on the Matlingeweg have been done in October 1990, a few days after reconstruction. These measurements could therefore be considered to be representative for the zero state. The second measurements were carried out before overlaying the pavement structure in December 1990. The measurements from January 1991 till June 1993 were performed on the pavement structure with an extra 80 mm asphalt layer at the top. In figures, this means 2 series of measurements on the pavement structure without overlay and 8 series of

measurements on the structure with an asphalt overlay.

Loading of the pavement was exerted on a 300 mm diameter plate by a falling weight, part of the FWD equipment. The deflections and strains were due to a 50 kN force corresponding to a 100 kN standard axle load. The deflection measurements were always done at the same places, above the built-in strain transducers, marked with the numbers 837 ... 840.

The analyzed pavement structure was not equipped with built-in thermo-couples in the asphalt. The temperatures assumed for the pavement were measured at the bottom of a borehole in the asphalt, at a depth of about 50 mm. The hole depth was less than the bottom of the asphalt layer where the strain transducers were laid. The total asphalt thickness was about 130 mm before and about 210 mm after overlaying. Therefore, the measured temperature values were corrected on the basis of the temperature distribution in pavement structures with an EPS sub-base, described in the literature [1].

3. DESCRIPTION OF THE MEASURING SITE

3.1 *Traffic characteristics*

The Matlingeweg is an alternative route between two heavily trafficked motorways which is a reason for the large number of vehicles using this road. Another reason is its central location in the industrial estate under construction. Because of the very intensive construction activities in this area a larger than average number of very heavy trucks contributes to the extra heavy traffic loading on the Matlingeweg.

The asphalt strain transducers are situated in the outer wheel track in the outer lane because this segment of the road cross section is used by all trucks and other heavy vehicles.

3.2 *Settlements*

The part of the area, where the Matlingeweg is located, once was the bed of the branches of the river Meuse. Later, the original sediments were covered by material that was dredged from the nearby harbor. The existence of silt and of dredged material at certain depths in the subsoil implies its poor bearing capacity. Resulting settlements have values varying from a few centimeters up to 0.2 m and more per year on some sections.

3.3 *EPS sub-base design*

The flexible pavement structure of the Matlingeweg is just partly laid on an EPS sub-base. Actually, EPS was built-in on two separate sections of this road (a sand sub-base is applied in the remaining part). Furthermore, the EPS sub-base has two different thicknesses on the sections where it was applied. The EPS layer is 0.5 m thick at the ends of the two above-mentioned sections and 1.0 m thick in the central part of both sections. In this way the quantity of EPS used was minimized for economic reasons.

The settlement values and corresponding pavement structure design are shown in Figure 1 for the longer of the two sections of the Matlingeweg with an EPS sub-base. A 1.0 m thick EPS package of two layers was laid on the terrain where the total settlement reached more than 0.7 m during three years. A 0.5 m thick EPS layer was used to obtain a gradual crossing between the road parts on relatively weaker subsoil and the adjacent road parts on the subsoil with a somewhat higher bearing capacity.

EPS30 was used in the upper EPS layer and EPS25 in the second layer. EPS30 and EPS25 are expanded polystyrene foams with densities of 30 kg/m^3 and 25 kg/m^3 respectively. The most widely used EPS type in road construction is EPS20 which means that, in the case of the Matlingeweg, heavier types than usual were used. The advantage of EPS30 and EPS25 over EPS20 is a somewhat higher elasticity modulus. The strength and E-value of the EPS increase with density [2]. The disadvantage of heavier EPS types is that they are simply more costly. The question is whether a few MPa more in elasticity modulus in the sub-base justify the accompanying extra costs. The larger the quantity of EPS applied, the more important this consideration becomes.

3.4 *Flexible Pavement Structure*

The roadbase in the pavement structure of the Matlingeweg consists of a 0.4 m thick layer of a mixture of crushed masonry and crushed concrete, and a 0.15

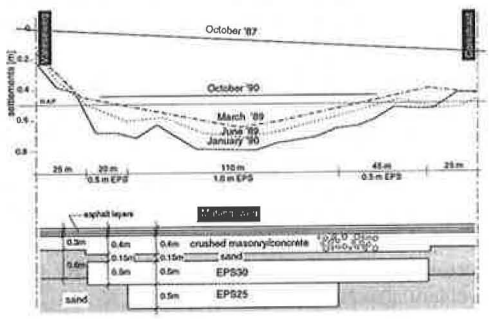

Figure 1. Previously obtained settlements and longitudinal profiles of the Matlingeweg.

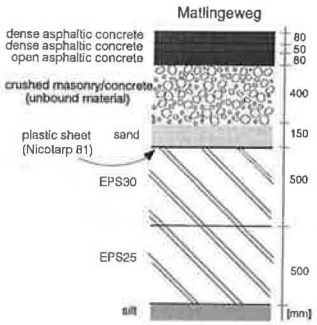

Figure 2. Pavement structure (incl. the asphalt overlay) with the EPS sub-base on the Matlingeweg.

m thick layer of sand above EPS. The applied roadbase material is an unbound material with a grain size distribution between 0 and 40 mm. This material is produced in a recycling process where bricks and concrete pieces that remain after demolition work are crushed to desired grain sizes. On the road sections without an EPS sub-base, the crushed masonry/concrete base is 0.3 m thick.

The drain sand above the EPS layer(s), besides its contribution to draining and loadspreading, protects the plastic sheet on the top of the EPS against mechanical damage. The plasticsheet also has to prevent leaked oil derivatives from reaching the EPS, as this could then be dissolved.

The asphalt package on the top was laid in two phases. Firstly, an open asphaltic concrete layer, type 0/22, with a thickness of about 80 mm, was laid over the roadbase. A top layer of 50 mm thick dense asphaltic concrete was laid during the road construction in October 1990. In December 1990, much earlier than planned, the pavement structure was overlaid by a 80 mm thick layer of dense asphaltic concrete. Figure 2 shows the pavement structure including the overlay on the Matlingeweg.

3.5 *Damage on pavement structure and overlay*

Alarming early failure was observed one month after completion of the reconstruction and opening of the road to traffic. Cracking occurred in the longitudinal direction in the section with a 0.5 m thick EPS sub-base at the location marked in Figure 3.
Before the overlay was laid two observation pits (●) were made to inspect the overall structure. The

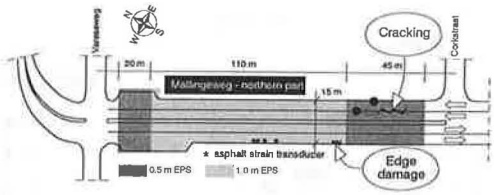

Figure 3. Location of the initial cracking in December 1990 on the Matlingeweg.

condition at the joints in the EPS sub-base pointed out that the EPS blocks were not properly laid. Instead of full contact between the blocks, which would enable the interaction between them, the blocks were laid with joints between them. The width of the transverse joints between the EPS blocks was approximately 20 mm. Furthermore, the unbound roadbase material appeared to contain many fine particles and was not well graded.

The reasons for the early deterioration were quite obvious in this case. Firstly, the presence of open joints results in a total absence of any load transfer between the blocks.

Secondly, it was observed that the longitudinal joint between the blocks was located just in the middle of the two lanes. This is a very critical location since the heavy wheel loads are almost right above the joint. Because heavy trucks almost exclusively use the outer traffic lane the inner wheel track corresponds with the position of the longitudinal joints. Consequently, the heavy trucks loaded the pavement structure at those places where the unbound roadbase material had a strongly reduced support because of the open joints in the layer below and the lack of load transfer in general. This led to a reduced support of the roadbase to the asphalt layers. An additional handicap of the pavement structure at the considered location was the presence of a single EPS layer. The visible result was the longitudinal cracking at the pavement's surface.

Thirdly, high deflections were measured on the structure which indicated that the crushed masonry/concrete base had a low stiffness and that high tensile strains were developing in the asphaltic layer.

From the analysis that was made it was concluded that most of the problems could be attributed to a less than optimal design (joint very close to the heavy loads), to a less than optimal construction (open joints) and to the fact that the structure as designed lacked sufficient stiffness. Therefore the overlay thickness was redesigned and increased from 50 to 80 mm in order to reduce the chance on the occurrence of cracks during the expected pavement design life.

The section where the cracking of the asphalt

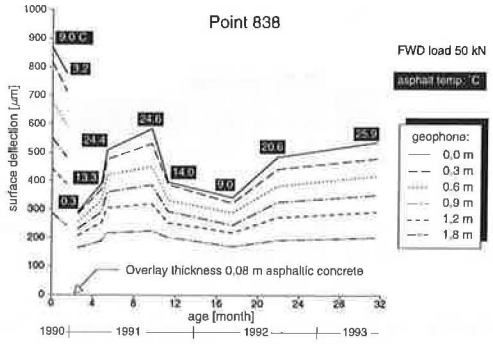

Figure 4. FWD measurement results above the strain transducer 838 on the Matlingeweg.

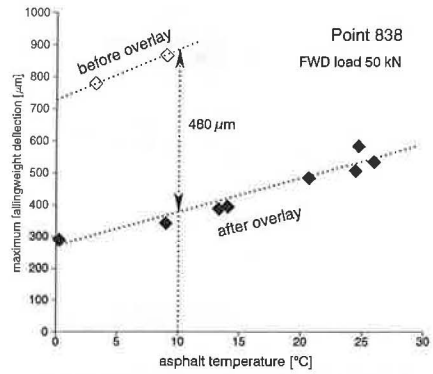

Figure 5. Maximum deflections measured at different temperatures above the strain transducer 838 on the Matlingeweg.

occurred was totally reconstructed. The asphalt layers were removed and the upper 150 mm of the roadbase were stabilized to improve its bearing capacity before overlaying with new asphalt layers with a total thickness of about 210 mm.

The pavement structure was not damaged on the road section where the strain transducers had been built-in. However, the road edge was quite seriously damaged about thirty meters further to the south.

4. FWD MEASUREMENT RESULTS AND PAVEMENT ANALYSIS

4.1 *FWD measurement results*

The temperature dependent behaviour of the asphalt layers has a big influence on the behaviour of the whole pavement structure. With respect to the FWD measurements the influence of temperature on pavement behaviour was expressed in lower measured deflections at lower temperatures on the Matlingeweg. A clear correlation between the measured deflection and temperature values can be seen in Figure 4 which shows the measured deflections as a function of time for the measuring point 838 on the Matlingeweg.

The FWD measurement results are very different for the first two surveys and the remaining eight because of the 80 mm asphalt overlay that was applied at the end of 1990. The measure in which this extra asphalt layer contributed to the decrease of deflections can better be observed in Figure 5. This figure shows that the maximum deflection (in the load center) as measured using 50 kN FWD load was linearly dependent on the temperature. At an asphalt temperature of 10° C the reduction of the maximum deflection due to the overlay equals 450 µm to 500 µm, a reduction of more than 50%.

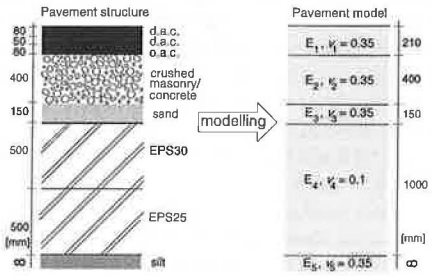

Figure 6. Flexible pavement structure model used in back-calculations of layer E moduli.

4.2 *Back-calculation analysis*

The analyzed pavement structure is modelled by a linear elastic model as it is illustrated in Figure 6. In other words, the materials have been considered as linear elastic, homogeneous and isotropic.

In the back calculation analyses the maximum number of layers that can be taken into account is equal to the number of geophones minus one. Consequently, one 1.0 m thick EPS layer was introduced instead of two layers of 0.5 m EPS30 and 0.5 m EPS25 respectively.

The deflection bowls have been back-calculated using the computer program BISAR. The differences between the measured deflections, corrected to a 50 kN load, and the deflections computed by BISAR were almost always within 2%.

4.3 *Back-calculated layer E-moduli*

The above described back-calculation analysis was carried out using the FWD results obtained on two points, the point 838 and the point 837, as input data. These points were the first (838) and the last measurement point (837) during each of the 10 performed FWD surveys. In this way those measurements were chosen for which the largest temperature difference could be expected in a single survey. The largest difference between asphalt temperature at the beginning and at the end of the FWD survey amounted to about 6.5° C (during the summer day in August 1991).

The back-calculated E-values of the layers in the pavement structure of the Matlingeweg on the measurement point 838 are listed in Table 1.

Table 1. Back-calculated E moduli of the layer materials in the flexible pavement structure of the Matlingeweg, based on FWD results on the measurement point 838.

	Asphalt temp. [° C]	Matlingeweg - point 838 asphalt	base	sand	EPS	subsoil
		back-calculated E modulus [MPa]				
Oct'90*	9.0	17500	85	40	10.4	60
Dec'90*	3.5	20000	80	50	12.6	67
Jan'91	0.3	25000	600	150	19.7	69
Mar'91	13.3	14000	420	120	16	65
Apr'91	24.4	9000	290	70	13	66
Aug'91	24.6	7000	170	80	14	65
Oct'91	14.0	13000	450	150	15	63
Apr'92	9.0	17000	490	150	19.2	68
Aug'92	20.6	10000	190	80	17	69
Jun'93	25.9	7500	180	80	16.5	66

* before overlaying

4.4 *Concluding remarks on FWD analysis results*

Two different cases could be distinguished, before and after overlaying.

The back-calculated E-values for the roadbase layers under the asphalt layer of 130 mm (before overlaying) were critically low (80 to 85 MPa for the crushed masonry/concrete base and 40 to 65 MPa for the sand layer). These values were just a fraction of the E-value usually assumed for these materials in the scope of pavement design. Such a low effective stiffness of the granular materials points out the very restricted support from the roadbase to the asphalt layer. The granular materials have a stress-dependent behaviour [3]. The stiffening of these materials is achieved if confining stresses develop in the roadbase. Low stiffness of the EPS in the sub-base means a handicap for above laid sand and crushed masonry/concrete to develop confining stress under loading. The inability of the EPS to provide a proper support to the roadbase material and the resulting low effective stiffness of the sand and the crushed masonry/concrete in combination with a relatively thin asphalt layer contributed to a large extent to the rapid pavement collapse under the heavy traffic loading.

The second case was the pavement structure after overlaying, with the 210 mm thick asphalt layer. The back-calculated E-values for the roadbase layers in this case had values from 140 to 600 MPa (crushed masonry/concrete base) and from 70 to 150 MPa (sand). The values found in the roadbase after overlaying are somewhat lower than could be expected from the considered unbound material. The fact that the base contained a higher amount of crushed masonry than crushed concrete, could explain its lower resilient modulus. In the case of sand, the resulting modulus has values which can be expected for this material.

In both cases the back-calculated E-value of the EPS sub-base ranges between 10 and 20 MPa which is somewhat higher than the elasticity modulus found in compression tests for the considered EPS types EPS25 and EPS30.

The back-calculated E-value of the asphaltic concrete top layer was obviously strongly dependent on the asphalt temperature during the FWD measurements. The back-calculated E-value of the asphalt top layer varied between 25,000 (asphalt temperature 0.2° C to 0.6° C) and 5,000 MPa (T_{asph}=31.0° C).

The E-values in the roadbase materials follow the trend of the modulus calculated for the asphalt layer. The moduli of elasticity of the roadbase layers decrease in periods with higher temperatures in accordance with a simultaneous decrease of E_{asph}-values. An opposite trend was expected because of the stress-dependent behaviour of unbound materials in the roadbase. In the cases when the E-value of the asphalt decreases (at higher temperatures) the stresses in the roadbase increase. Consequently, its (resilient) modulus should increase because of stress dependency. However, just the opposite trend can be noticed from Table 1. The explanation for this seems to be that EPS did not provide sufficient support to the unbound material in the case of larger stress values. Therefore no high all-around confining stresses, which lead to stiffening of the unbound roadbase materials, could develop in the unbound materials. This hypothesis is supported by the fact that there is a rather good relation between the maximum deflection and the base modulus. At high maximum deflection, indicating a significant amount of movement in the structure, low base moduli were obtained, which seems to be a logical behaviour of unbound materials.

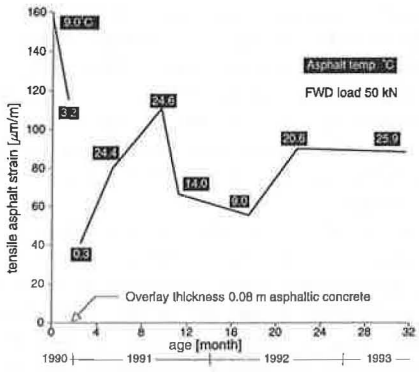

Figure 7. Horizontal tensile strains measured by means of the transducer 838 at the bottom of the asphalt layer of the Matlingeweg.

5. ASPHALT STRAINS

5.1 *Asphalt strain measurement results*

The tensile strains measured at the bottom of the asphalt layer due to the FWD loading are dependent on the asphalt temperature. Figure 7 shows the strains measured by the means of the transducer 838 built-in in the Matlingeweg.

The minimum asphalt temperature during the strain measurements was equal to 0.2° C (January 1991) while the maximum temperature was 31.0° C (August 1991). Figure 8 shows the same strain values as in Figure 7 but this time the strains are plotted against the corresponding asphalt temperatures.

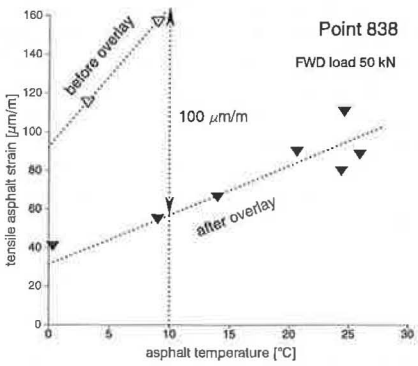

Figure 8. Horizontal tensile strains measured by means of the transducer 838 as a function of the asphalt temperature.

It is obvious that this relation shows more scatter than the deflection vs temperature plot but a strong dependence of the asphalt strain on temperature can be observed. From this figure it can also be observed that the overlay produced a reduction in strain level of 80 to 100 µm/m at a temperature of 10° C. If this relation is used to estimate the tensile strain at an asphalt temperature of 20° C (normally this temperature is used for design purposes) then a measured asphalt strain of about 210 µm/m is predicted for the pavement structure before overlaying and about 80 µm/m after overlaying. This high asphalt strain estimated for the original structure would result into a rather short pavement life expressed in the number of repetitions of a 50 kN load. The asphalt strain reduction due to the overlay can be roughly estimated at about 110 to 150 µm/m at the reference asphalt temperature of 20° C.

5.2 *Asphalt strains at the reference temperature*

The previously described analysis allowed a rough comparison of the strains at different asphalt temperatures, although the development of the asphalt strains in time due to the traffic loading could not be determined in this way. Therefore this analysis was made by studying the development of the asphalt strain in time at a given reference temperature of 20° C. Transformation of the measured strain values to the corresponding values at the reference temperature of 20° C was carried out by the means of an established relationship between the back-calculated E-values of the asphalt layer (Table 1) and the corresponding asphalt temperatures (Figure 9).

One should keep in mind the manner in which the asphalt temperature was determined. No built-in thermo-couples were available so that the temperature was measured in a narrow borehole with a depth of approx. 50 mm. The measured temperatures have been corrected and the corrected temperature values, shown in Figure 7, have been considered as relevant for the asphaltic concrete layer of the Matlingeweg. In this way the value of S_{mix}=10,500 MPa was found for T_{ref} equal to 20° C.

As an illustration the chart of a standard Dutch asphalt mix is also shown in Figure 9. It is the gravel asphaltic concrete used in road pavement structures exposed to traffic class 3 and 4 [4].

The next step consisted of BISAR calculations of the tensile strains at the bottom of the asphalt layer using the back-calculated E-values of Table 1. The resulting strain values are marked as ϵ_{calT} (cal=calculated, T=temperature). Correlation between the calculated asphalt strains ϵ_{calT} and the corresponding measured strains ϵ_{mT} is shown in Figure 10.

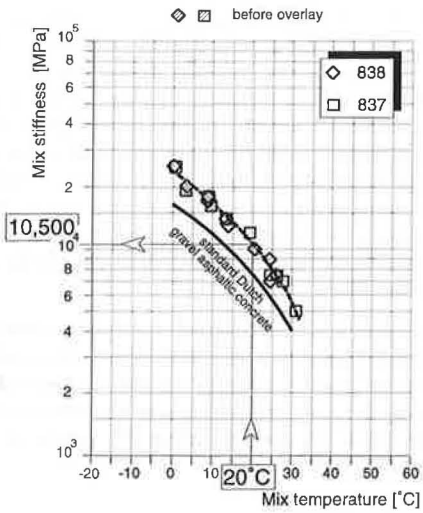

Figure 9. Relation between asphalt mix stiffness and asphalt mix temperature.

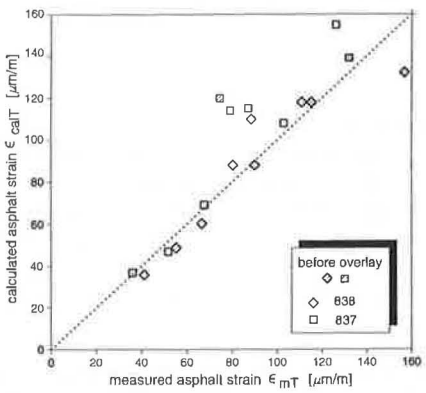

Figure 10. Correlation between the measured strains at the bottom of the asphalt layer and the corresponding strain values calculated by means of the BISAR program.

BISAR calculations were performed again with the value S_{mix}=10,500 MPa as the input E-value for the asphalt layer. The corresponding E-values from Table 1 were used for the remaining pavement layers. Consequently, the stress-dependent behaviour of base materials was neglected but no alternative method exists. Resulting strains had the mark ϵ_{cal20}.

In Figure 11 the calculated asphalt strain for the asphalt temperature at 20° C is shown as a function of time. Before placement of the overlay, the ϵ_{cal20}

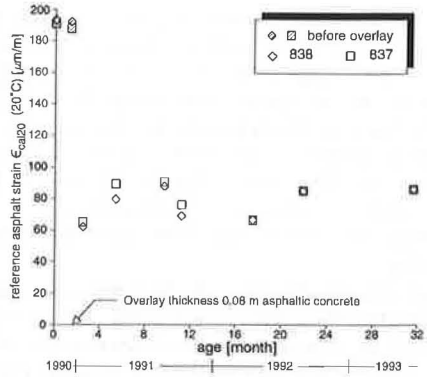

Figure 11. Calculated asphalt-strain values in time for the reference asphalt temperature of 20°C.

asphalt strains were about 192 μm/m, a very high value. This high asphalt strain resulted in a short pavement life of the original structure. After overlaying the asphalt strain was equal to about 85 μm/m. Furthermore, no increase in the asphalt strain in time after the overlay points to a good condition of the pavement structure.

From Figure 11 it can be observed that the overlay produced a reduction in strain level of approximately 105 μm/m, which is somewhat higher than the values estimated on the basis of Figure 8. Nevertheless, the predicted strain reduction is in fairly good agreement with the measured values.

5.3 *Concluding remarks on asphalt strains*

The strain level at the bottom of the asphalt layer was equal to approximately 192 μm/m (at the reference temperature of 20° C) before the overlay was placed. The design life of the original pavement structure, roughly estimated on the basis of the fatigue curves for the gravel asphaltic concrete in [4], amounts to only few thousands of 100 kN standard axle load repetitions. Notwithstanding the inability to determine the pavement design life in a more accurate way (no fatigue test results were available of the asphaltic concrete actually applied) it was obvious that the pavement structure on the Matlingeweg had a very short design life before the overlay. The main reason for the improper pavement design was the use of the overestimated E-values for the unbound materials in the roadbase.

The asphalt strain at the reference temperature (20° C) remained more or less constant in the 3 year period after overlaying. The constant value of the strain

is a sign of a good condition of the pavement structure. Furthermore, no trend could be observed at the end of the observation period, which would indicate that an increase in the asphalt strain should be expected in the next period. The horizontal tensile asphalt strain amounts to about 85 μm/m at the reference temperature of 20°C. This means that the lifetime of the pavement after overlaying is approximately 25 to 30 times longer than the lifetime of the pavement before overlaying.

Back-calculated E-values for the roadbase material of the Matlingeweg point to a decrease with the temperature of the effective stiffness of this material built-in above the EPS because of the lack of support from the sub-base. In order to design an appropriate road base thickness the minimum E-values from literature should be used as input data in calculations of the design life of the pavement structures with the EPS sub-base.

6. CONCLUSIONS AND RECOMMENDATIONS

- The back-calculated very low E-values for the sand (from 40 to 65 MPa) and the crushed masonry/concrete (from 80 to 85 MPa) before overlaying highlight the inability of the EPS to provide a proper support to the roadbase in the considered pavement structure with a 130 mm thick asphalt layer. Correspondingly insufficient support of the roadbase to the asphalt layer resulted in a critically high asphalt strain of about 192 μm/m (T=20°C). Use of overestimated E-values for the roadbase materials was probably the main reason for the inappropriate pavement design.

- Open joints between the EPS blocks in a sub-base can have very serious consequences for the design life of pavement structures, and thus have to be avoided by all means. The joints between the blocks in various layers should not coincide with each other. The open joints are especially risky in the case of an EPS sub-base which consists of only one EPS layer. The longitudinal joints between the EPS blocks should not be close to a wheel track. An adequate (lateral) support of the blocks is necessary to prevent any movement of the blocks.

- The back-calculated E-value of the EPS sub-base ranges from 10.4 to 19.7 MPa, which is somewhat higher than the elasticity moduli found in literature for the EPS types under consideration (EPS25 and EPS30).

- The back-calculated E-value of the asphaltic concrete layer varied between 5,000 and 25,000 MPa, due to the temperature range of 0.2°C to 31.0°C, during the various FWD measurements.

- The E-value of the crushed masonry/crushed concrete roadbase ranged from 140 to 600 MPa after overlaying. The modulus found for the sand varied between 70 to 150 MPa. In some measurements the values found for the crushed masonry/ concrete were lower than could be expected for this unbound material. In order to design an appropriate pavement thickness on an EPS sub-base the E-values obtained in this study for the unbound base and sand layer are suggested to be used as input data in calculations of the design life.

- The asphalt strain remained more or less constant in the 3-year period after the overlay. The constant value of the strain is a sign of a good condition of the pavement structure. The maximum horizontal tensile asphalt strain amounts to about 85 μm/m at the reference asphalt temperature of 20°C.

ACKNOWLEDGEMENT

Shell Nederland Chemie BV is greatfully acknowledged for sponsoring this research into EPS geofoam.

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