MONITORING PROGRAM CRCP HIGHWAY PAVEMENT: INTERIM PERFORMANCE EVALUATION

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ABSTRACT

This paper presents the first findings and analysis of data collected in a monitoring program of a newly constructed continuous reinforced concrete highway pavement initiated in 1997. This monitoring program was set up as it was felt that pertinent data were not available to make an objective comparison in the process of deciding between an asphalt and a continuos reinforced concrete pavement structure.

Over a ten-year period a wide range of data will be collected such as construction data, including laboratory results, traffic, temperature (air and pavement), structural condition, detailed visual inspections, surface friction, roughness, noise level, joint movements and other relevant information. All data and reports will be stored in a database to facilitate easy access and quarrying.

The data collected over the first three years and the analysis performed have resulted in a detailed insight into the as-built parameters, development of stresses based on the change in temperature gradients and the traffic load spectrum. At this stage of the monitoring program conclusions can already be drawn based on the information gathered to date as processed in a study performed at Delft University of Technology.

INTRODUCTION

A new highway section has been built in the European route E35 (A12) in the Netherlands, one of the main highways into Germany. Due to the ever-increasing costs associated with investments in infrastructure, it was felt that more detailed information was needed in the decision-making process between an asphalt and a continuos reinforced concrete solution. This was done by looking at the total costs, not only based on the initial construction but also including maintenance costs and other costs such as delay times due to traffic obstructions.

In the design phase a CRCP structure was chosen with a design life of forty years, including a cement-bound sub-base using recycled asphalt, a bituminous bound interlayer and a porous friction course as surfacing. It was however concluded that insufficient data were available to make a realistic comparison between different types of pavement structures.

The construction of the CRCP road section in the southbound direction of the A12 motorway between km 65.18 and 67.44 created an opportunity to set up a monitoring program. The growing traffic queues on this section of the A12 with a fast-growing traffic load which currently stands close to 100,000 motor vehicles per day, led the Dutch Department of Public Works (*Rijkswaterstaat*) to decide to increase the number of lanes by constructing a completely new section parallel to the existing road.

Connecting asphalt sections with standard and special mix designs had to be monitored too. The main objectives are to collect relevant data to enable an objective comparison to be made between CRCP and asphalt structures by checking the design inputs such as percentage of reinforcement of the CRCP and analyzing the long-term behavior of the CRCP structure.

The interim evaluation result (1) of the data collected over the first three years and data analysis directed towards the development of closely spaced transverse cracks (2) are addressed in this paper.

PAVEMENT CONSTRUCTION

The design of the CRCP pavement was based on a method used for non-reinforced concrete slabs (PCC) but incorporating reinforcement in such a way that the most favorable crack-pattern is obtained. The design objectives were to:

- calculate a non-reinforced concrete pavement for a 30-year lifetime;
- incorporate 0.67% reinforcement to obtain class III (0.8-3.0 meter spacing) transverse cracks;
- extend the lifetime to 40 years by neglecting the thickness reduction as a result of the application of reinforcement.

The length of the CRCP section is 2,240 meters, to be constructed in an area with a soft sub-grade. The soft subsoil was replaced by sand before the start of construction of the pavement structure. The foundation of the CRCP consists of cement-bound recycled asphalt granulate (AGRAC) with a thickness of 250 mm. This layer has been pre-cracked to avoid uncontrolled cracking. In between the AGRAC and CRCP a 60 mm thick asphalt inter-layer is constructed to create a leveled and even surface. The CRCP design was based on a thickness of 252 mm with a longitudinal reinforcement of Ø16-120 and a transverse reinforcement at a 60 degree angle of Ø12-700. Dowel bars of Ø12 with a length of 800 mm at 800 mm intervals are used in the longitudinal joints.

Table 1 summarizes the results of the design objectives and the actual parameters of the CRCP pavement. The actual realized parameters are based on back-calculated FWD data.

Table 1. Design objectives and realized parameters

Parameter	**Design Objective**	**Actual Realized**
Thickness sand sub-grade	250 mm	261 mm
E-modulus sand sub-grade	100 MPa	>200 MPa
k-value sub-grade	0.045 N/mm^3	~0.070 N/mm^3
Thickness Lean Concrete base	150 mm	not realized
E-modulus base	9.000 MPa	-
Thickness AGRAC base	250 mm	256 mm
E-modulus AGRAC base	not designed	4.000 MPa
Composed k-value	0.118 N/mm^3	~0.160 N/mm^3
Thickness asphalt inter-layer	60 mm	approx. 60 mm
E-modulus asphalt inter-layer	unknown	7,500 MPa
Thickness concrete	252 mm non-reinforced	250 mm min./261 mm av.
Strength class concrete	C45	C57 average

Due to environmental regulations the CRCP was overlaid as planned, one year after opening for traffic, with a porous friction course to reduce “splash and spray” and noise.

MONITORING PROGRAM

Data such as the type of pavement structure including the sub-grade, traffic (Weight-in-Motion and speed), temperature (air and pavement), structural condition (FWD), visual assessment including crack pattern, surface friction, noise level, roughness, joint movements and laboratory test results were part of this program.

Due to the large amount of data collected and the size of the database, the accessibility of the data is of great importance. The software package MONIVIEW (see Figure 1) was developed to enable all data stored in the database to be viewed. The database can be explored at any time during this monitoring program for a period of ten years, beginning in 1997.

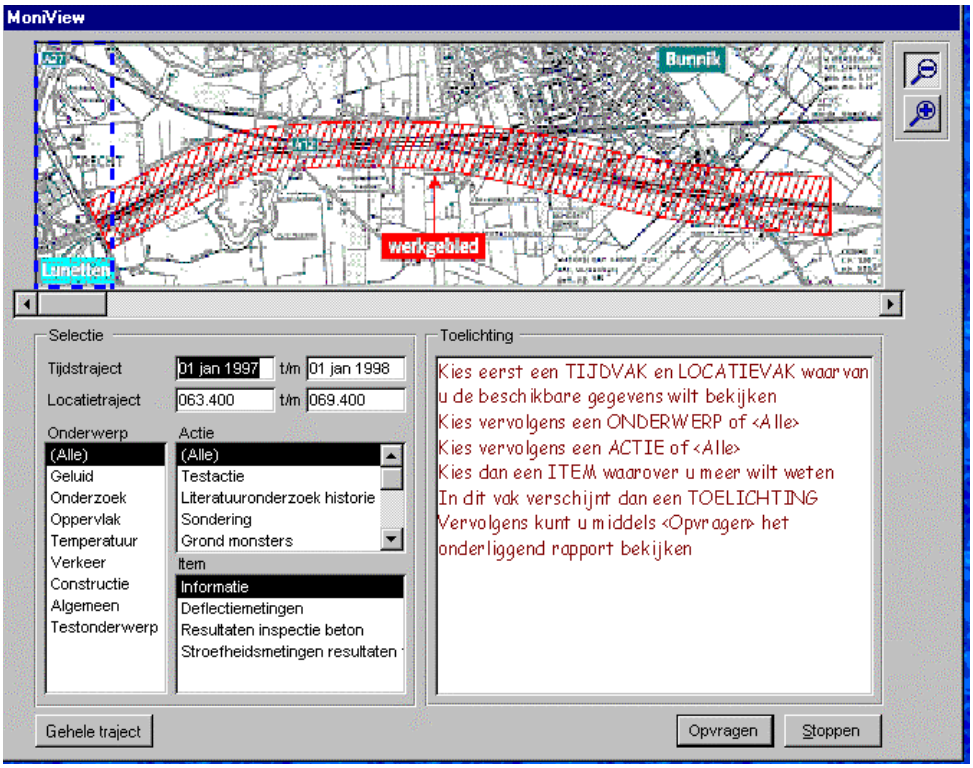

Figure 1. Opening screen shot of MONIVIEW

To quarry the database, the time period and road section first have to be selected. Thereafter a subject has to be chosen, as well as the type of information and the specific item. Based on the item selected the reports containing the requested information can be consulted.

INFLUENCE OF VARIATIONS IN TEMPERATURE

There are many climatic factors influencing the temperature of the CRCP and the related temperature gradient. These include the number of hours of sunshine, the minimum and maximum day temperatures, the wind speed, the amount and type of precipitation and the weather conditions in the preceding days. All these varying influences make the comparison between the weather conditions and the temperature gradient in the concrete extremely complex. During the monitoring temperatures were measured at different depths in the CRCP and weather data were provided by a nearby weather station.

Climatic changes will result in temperature-induced stresses in concrete pavements. These temperature stresses can be divided into three categories:

1. Stresses as result of changes in temperature ΔT being constant over the thickness h;
2. Stresses as result of an irregular change in temperature over the thickness h;
3. Stresses as result of a temperature gradient Δt being constant over the thickness h.

An even increase or decrease in temperature ΔT will result in compressive or tensile stresses in the concrete slab due to friction with the base layer. These stresses can be neglected for non-reinforced slabs due to the limited dimensions. For CRCP structures the ‘slab dimensions’ will be the result of the ‘controlled’ cracking by the longitudinal reinforcement. As cracks will appear

every 1.0 to 2.5 meters (class III cracking) the stresses due to a constant increase in temperature over the concrete thickness will be extremely small.

An irregular change in temperature will result in internal stresses, though these are only relevant in the case of very thick slabs, and are very small for the 250 mm thickness in question. Temperature gradients Δt will result in bending stresses of the same magnitude as traffic-induced bending stresses. For this reason temperature gradients have to be included in the design of concrete pavements. The temperature gradient Δt [^{o}C/mm] can be defined as:

- $\Delta t = (T_{top}-T_{bottom})/h$.

Positive values of the temperature gradient are notably greater (absolute) in comparison to negative values. This can be explained by the influence of sunshine, which together with the air temperature is responsible for the temperatures in a concrete pavement. This could mean that the highest temperature gradients would occur in the summer. Positive gradients will develop during daytime together with the main volume of commercial traffic. Eisenmann (3) has developed a method to calculate the so-called warping stresses in a concrete slab due to a positive temperature gradient introduced under the name 'critical length'. Although this method is not meant for CRCP pavements, once cracks have developed a CRCP construction can be regarded as consisting of 'slabs'. The reinforcement will only keep these slabs together and control the spacing of the cracks, without controlling the stresses due to temperature gradients.

To study the temperature gradient in the CRCP pavement, registration units were installed at 10, 60 and 230 mm depth. Together with information received from a nearby weather station the influence of the number of hours of sunshine was analyzed. Figure 2 shows an example of the measured temperatures on 8 May 1998.

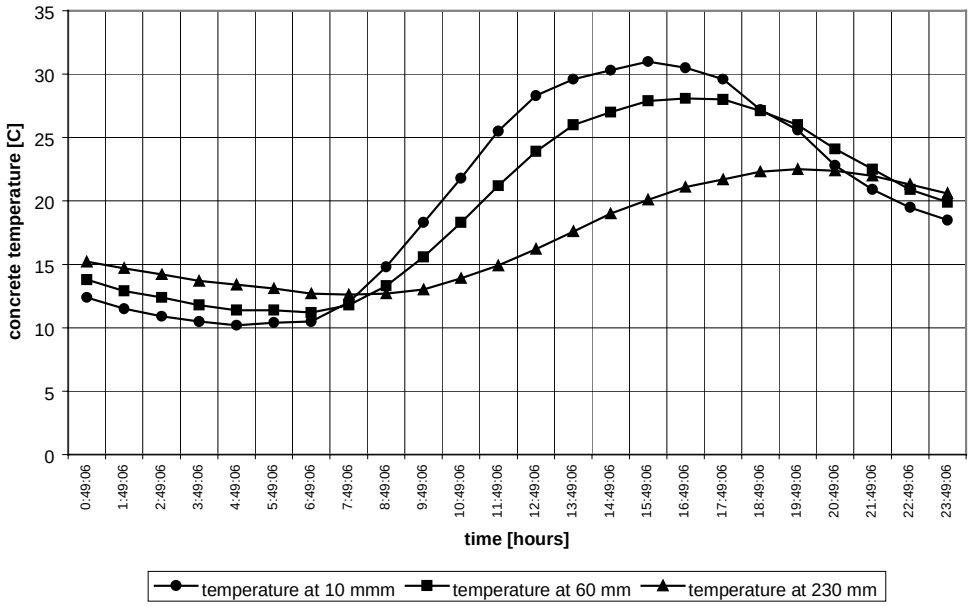

Figure 2. Temperature range on 8 May 1998

Two periods, 7-18 May 1998 and 16-23 November 1998, have been studied in detail. The CRCP was not overlaid yet with the porous friction course. In Figure 3 the number of hours of sunshine is compared with the calculated temperature gradient over the first period.

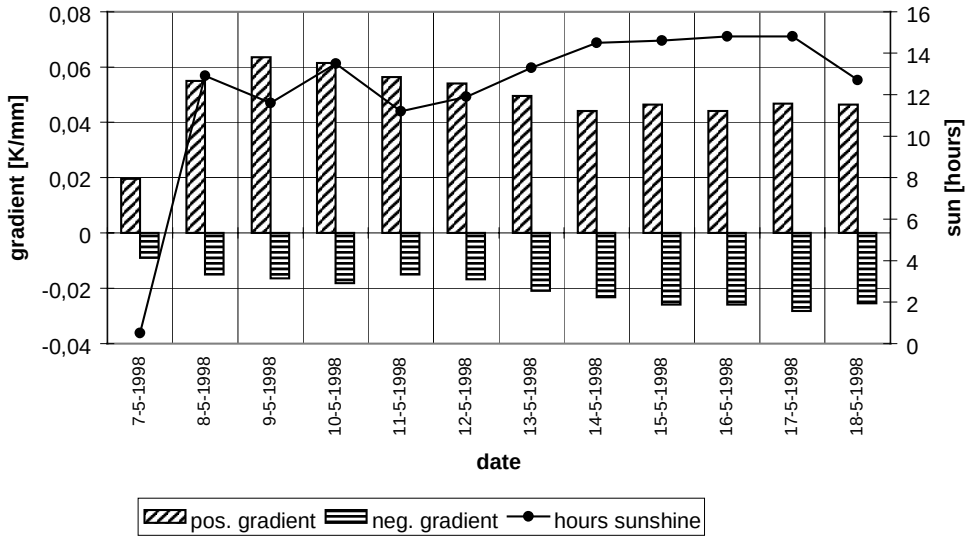

Figure 3. Maximum positive and negative gradients and hours of sun per day

Figure 3 shows that the maximum gradient of 0.0636 [^{o}C/mm] occurs on 9 May when the concrete surface temperature increases 'rapidly', whereas the temperature at the bottom of the concrete has not had time to increase. As the amount of sunshine increases the temperature at the bottom will also increase, resulting in a decrease in the positive gradient. The hours of sunshine per day has a distinct influence on the magnitude of the positive gradient. To confirm this conclusion other periods with fewer hours of sunshine have been analyzed. In the period from 16-23 November 1998 the absolute gradient is smaller in comparison to the first period. It can be questioned whether this is due to the smaller number of hours of sunshine, the lower angle of the sun or the lower air temperatures. Two other periods, in December 1998 and January 1999, were chosen with a minimum amount of sunshine. These two periods do not show any influence due to the number of hours of sunshine. The gradient is clearly influenced by the number of hours of sunshine, and continuation of a sunny period will decrease the gradient again. Before drawing definite conclusions, however, the influence of air temperature and wind speed is also looked at.

The same time periods have been used to analyze the air temperature. For the period in May with high air temperatures no evidence can be found that high air temperatures also result in high gradients. This could be caused by the fact that the temperature differences in this period are too small to detect this correlation or the influence of the hours of sunshine overshadows any other cause. The period of 2-9 January 1999 has only very view hours of sunshine in comparison, with a maximum temperature varying between 5.2 and 13.5 ^{O}C. Figure 4 does indicate that the temperature gradient could be related to the maximum temperature.

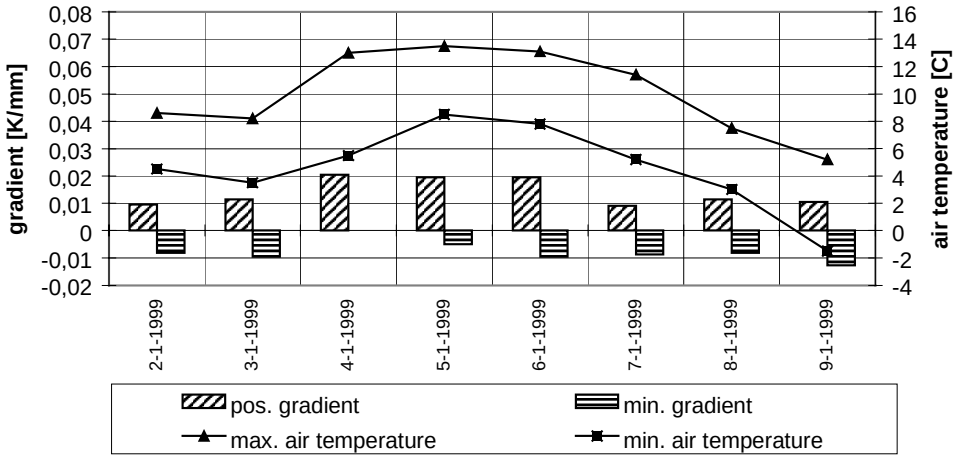

Figure 4. Relation between temperature gradient and air temperature

The gradient in Figure 4 is increasing on 4 January because the air temperature increased by 5 degrees in comparison to the day before. The surface temperature of the concrete rises faster, resulting in an increase in gradient. However, this gradient starts decreasing again from the moment the bottom temperature of the concrete begins to rise.

Looking at the influence of wind speed on the temperature gradient does not result in a clear relationship. The only conclusion that can be drawn is that during colder periods a higher air temperature correlates to higher wind speeds. This is most probably due to the wind direction, as the relatively warmer seawater temperature for winds coming from the northwest warms up the air.

The three most important parameters influencing the magnitude of the positive temperature gradient are the air temperature, the number of hours of sunshine and the wind direction. The highest gradients occur in the summer period when the air temperature rises considerably over a short period, together with the number of hours of sunshine. These gradients decrease again from the moment the bottom temperature of the concrete pavement starts to rise. In the winter periods the highest gradients occur when the air temperature increases sharply.

TEMPERATURE GRADIENTS AND TRAFFIC LOADING

Traffic loads were measured using a Weight-in-Motion (WIM) system. In combination with the distribution of the temperature gradient over a 24-hour period, the design load for the calculation of the bending stresses at the bottom of the CRCP can be determined. The combinations of load and gradient will differ substantially over all seasons. However, in general the temperature gradients during the summer period will be higher than in the winter period. This will result in a design load occurring in the summer period. As there is insufficient data as yet to cover a complete year for the determination of an average temperature gradient, a weighted mean has to be calculated based on a year consisting of four seasons.

For this calculation it is important to know whether the maximum positive gradients coincide with the maximum traffic load. The temperature gradient in the summer period has a sinusoidal shape over the daytime, with a maximum between 12.00 and 16.00 hours. During the winter period the maximum values occur a little later during the afternoon and the shape is far more variable. In Figure 5 the distribution of the traffic in the heaviest loaded lane versus the temperature gradient is shown for 8 May 1998.

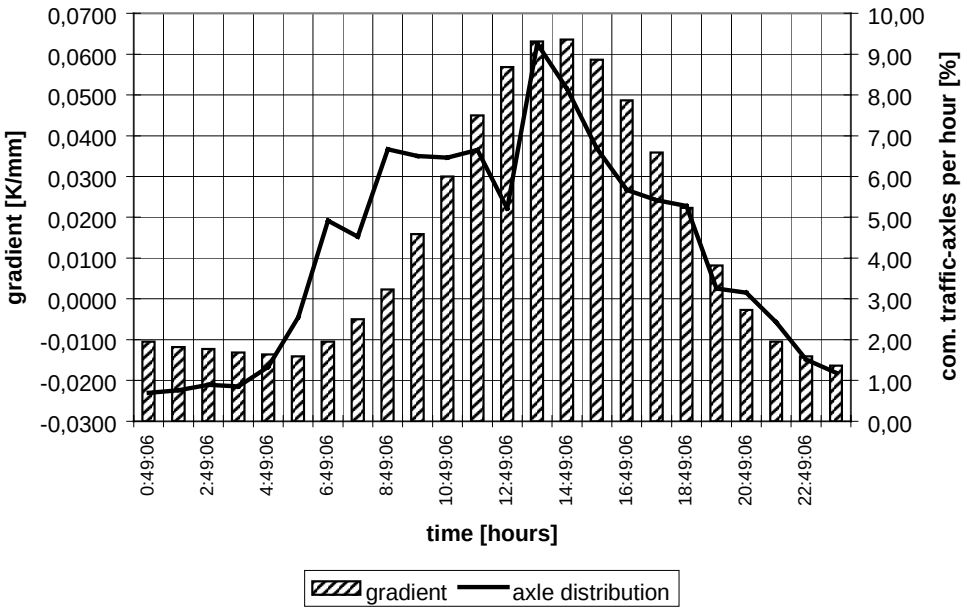

Figure 5. Percentage of axles per hour vs. temperature gradient (08-05-'98)

From Figure 5 it can be concluded that in the summer period the greatest commercial traffic intensity coincide with the highest positive temperature gradients and will be representative for the design of the CRCP. During the winter period the maximum gradients do not always overlap the period with the greatest commercial traffic intensity. Besides this, the gradients - and therefore also the stresses in the CRCP - are often (much) smaller in the winter period.

Using the data gathered a frequency distribution of the percentage of axle loads versus the temperature gradient was built based on a year divided into four seasons. This weighted frequency distribution of the axle loads is based on three winter and one average summer period and is shown in Table 2.

Temperature gradient Δt [°C/mm]	Frequency distribution of axle loads [%]
0.00	59.0
0.01	22.0
0.02	8.0
0.03	5.0
0.04	4.0
0.05	1.5
> 0.06	0.5

Table 2. Weighted frequency distribution vs gradient (3*winter + 1* average summer)

It is evident that the gradient depends on the speed at which the temperature increases or decreases and the number of hours of sunshine per day. As the CRCP was overlaid with a porous asphalt layer one year after opening for traffic, it is expected that the influence of sun and temperature changes will decrease and that the positive temperature gradient will reduce. Temperature measurements performed to date do in fact show that the maximum positive gradient measured before the overlay of 0.0636 ^{o}C/mm has not yet been reached. Better still, the maximum gradient measured so far has never been larger than 0.0364 ^{o}C/mm.

REINFORCEMENT DESIGN

A combination of stresses as a result of temperature and traffic loading will lead to failure of a concrete pavement. The joints in non-reinforced concrete pavements aim to avoid random cracking. In the case of a CRCP pavement the task of the reinforcement is to control the spacing of transverse cracking and keep the cracks closed.

The percentage of longitudinal reinforcement (0.67%) for the A12 is chosen to produce an 'optimal' cracking pattern. This means that sufficient cracks will develop of such a width that load transfer will be guaranteed, ingression of dirt avoided and corrosion of reinforcement reduced to a minimum. This means that most cracks have to fall within a spacing of 0.8-3.0 meters (class III), with a minimum of closely spaced cracks (class I 0.0-0.4 meter and class II 0.4-0.8 meter).

Cracks will develop in hardening concrete due to shrinkage and in hardened concrete due to a decrease of temperature and moisture in the pavement. The shrinkage will be opposed by the reinforcement and the friction with the base layer, resulting in tensile stresses in the concrete. When this tensile stress is higher than the maximum allowable stress, the concrete will crack and the stresses will be taken over by the reinforcement. This explains the objective of the reinforcement: to keep the cracks closed instead of avoiding cracks. The lengthening of the reinforcement due to these stresses should be kept within limits so as to keep the cracks closed, enabling the concrete pavement to continue acting as one element.

The design criteria used for the reinforcement of the A12 show some agreement with the criteria used in the AASHTO method, based on crack spacing, crack width and allowable stress in the reinforcement. However, when the A12 design parameters are used as input in the AASHTO method there is little agreement. In the first place the percentage of reinforcement needed is

much smaller, and secondly there is a contradiction between the maximum percentage of reinforcement based on the minimum crack spacing and the minimum percentage based on the maximum crack width. As the design criteria deviate from the actual achieved values, the AASHTO diagrams were gone through in reverse based on the as-build values. Using the 0.67% reinforcement results in a crack width of 0.018 inch (0.46 mm), however, widths of 0.1-0.2 mm have been measured. In the same way the crack spacing results in 2.25 feet (0.57 m), being class II, whereas 78% of the cracks can be found in classes III and IV. These differences in result may be explained by the difference in friction between the concrete and the asphalt interlayer, resulting in lower stresses in the reinforcement in combination with smaller crack width. In addition, the stresses caused by the temperature gradient will be smaller due to the friction as well as to shrinkage of the concrete. As the friction with the base layer plays an important role in the development of cracks, an asphalt layer has been chosen as the interlayer to create an evenly distributed friction, resulting in a regular crack pattern with small crack width and sustained interlock.

The objective of the reinforcement in a CRCP pavement is to create evenly spaced cracks within well-defined limits. If closely spaced cracks develop a CRCP will no longer act as one slab, but as a series of individual ‘beams’. This will result in a reference stress acting perpendicular to the driving direction of the traffic, causing longitudinal cracking. Locally these transverse and longitudinal cracks could form ‘tiles’, which are loosened by passing traffic. Visual inspections have been performed regularly to monitor the development of the transverse cracking. During the first round of inspections the anticipated crack pattern was not yet complete, and about 90% of the cracks were in class III (0.8-3.0 m) or class IV (>3.0 m). During the second round of inspections the crack pattern had developed completely and a shift had taken place towards class I (<0.4 m) and II (0.4-0.8 m). Local closely spaced cracks can be observed. Figure 6 shows an example of such a detailed visual inspection. As the CRCP is overlaid now with a porous friction course a further cracking development is not visible anymore.

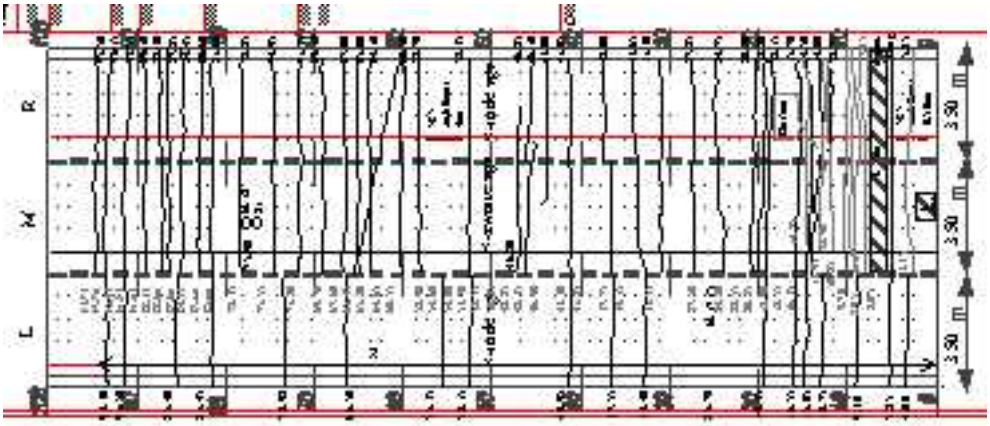

Figure 6. Detailed visual inspection

The local occurrence of these closely spaced cracks was studied in detail. In the first place the possibility was investigated of whether the concrete quality, CRCP ends, reinforcement bar overlaps or day-joints could have been the cause. Based on the location of these closely spaced cracks, no correlation could be proved. However, one problem was traced, as in some places a

crack was found that showed an angle in comparison to all other transverse cracks. These types of cracks had a repetition rate of about 100 meters, indicating that they were related to a pipe used to drain water from one side of the road to the other during construction. These pipes, with a 40 mm diameter, were located at the bottom of the CRCP, resulting in a reduction in thickness, and automatically induced a crack.

The next step was to find a correlation between the closely spaced cracks and the stiffness of the CRCP. Based on the data provided by non-destructive deflection testing (FWD), the stiffness of all layers were back-calculated. This was done assuming full friction between all layers. This resulted in erratic results for the calculated stiffness of the CRCP layer. However, this did not correlate with the visual inspection results assuming a reduction in stiffness due to reduced thickness. The only possible explanation for the disagreement between the measured deflections in the homogeneous sections identified and the crack pattern was found to be a varying degree of friction between the CRCP and the asphalt interlayer. A lack of bonding will however not result in more closely spaced cracks. However, due the local lack of bonding the CRCP pavement will curl and warp more easily and this, in combination with the traffic load, could cause more closely spaced cracks locally.

STRESSES DUE TO CLOSELY SPACED CRACKS

The locally present closely spaced cracks in the CRCP form small concrete beams in transverse direction. However, no longitudinal cracks appear in those concrete beams as is usually the case, but therefore the CRCP is still too young. Based on the deformations caused by the imposed loads the stresses can be calculated. Two extreme deformation cases can be recognized: the beam remains bonded with the asphalt interlayer or the beam warps due to a positive temperature gradient and the lack of bonding (4). In the first case it is a continuous elastic supported beam and in the second case it is a beam only supported at its ends (see Figure 7). For the calculations, a beam has dimensions of 100 mm width, 250 mm height and 3600 mm length.

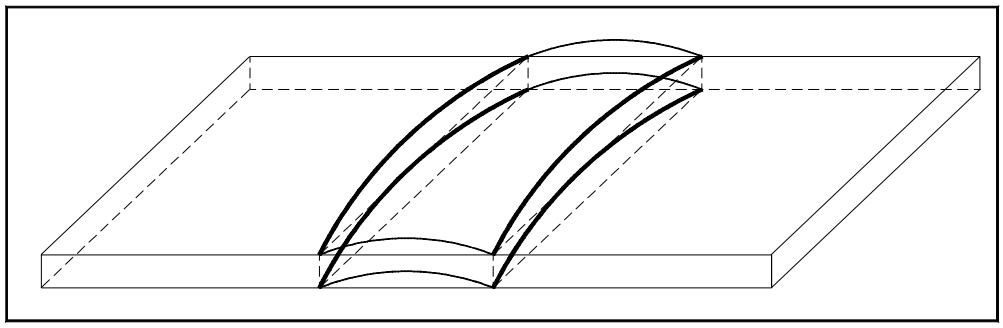

Figure 7. ‘CRCP beam’

In the first case it is assumed that the beam is continuously supported by bonding (and interlock). The temperature difference between the top and bottom of the beam will result in flexural stresses. In the case of a maximum positive gradient of 0.0636 ^{o}C/mm the temperature difference will be 16 ^{o}C. The difference between the resulting stresses at the top and bottom will be equal to:

- $\Delta\sigma_{t;b} = E*\Delta\varepsilon$ and $\Delta\varepsilon = \alpha*\Delta T_{t;b}$

  where:

E	= 42,500	[N/mm^2]
α	= 1*10^{-5}	[^{o}C^{-1}]
$\Delta T_{t;b}$	= 16	[^{o}C]

  $\Delta\sigma_{t;b}$ = 42,500*1*10^{-5}*16 = 6.8 N/mm^2

This means that at the top of the concrete pavement there is a flexural compressive stress of 6.8/2 = 3.4 N/mm^2 and at the bottom of the concrete pavement there is a flexural tensile stress of 6.8/2 = 3.4 N/mm^2.
These stresses are present in the middle of the span of the concrete beam. To obtain the stresses in the wheel track, the mid-span stresses should be multiplied by the reduction factor R which has a parabolic shape over the width of the beam.

- $R = 4y(B'-y)/(B'^2)$

  where:

B'	= span over the width of the beam [mm]
y	= distance from point on transverse edge to longitudinal edge minus 1/3 C [mm]
C	= length of support [mm]

  with

C	= 652 mm
y	= 533 mm
B'	= 3165 mm

  Resulting in R = 0.56

The flexural tensile stress due to the temperature differential at the bottom of the beam underneath the wheel load is 0.56*3.4 = 1.90 N/mm^2.

The concrete's own weight has been ignored, since the continuous support does not cause any bending stresses in the concrete.

The traffic load acting on the beam is an axle load having a maximum of 220 kN under the most unfavorable conditions, i.e. two wheel loads of 110 kN with a spacing of about 2.10 meters. With a beam width of 100 mm it will only take about ¼ of the load (27.5 kN) based on its area. These loads are shown schematically in Figure 8.

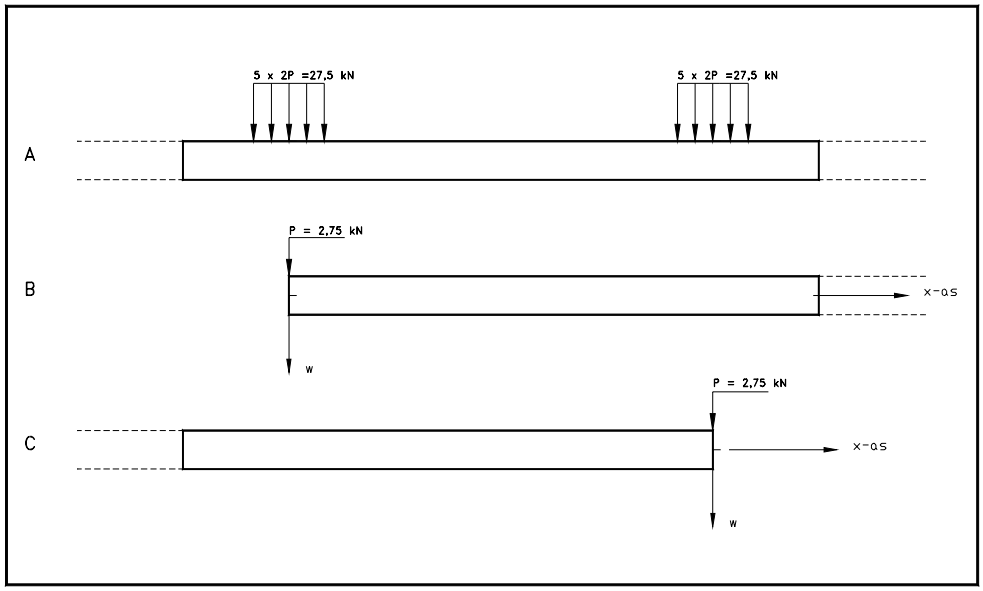

Figure 8. Schematic loads on a continuous long elastic supported beam

Load condition A can be divided into the two cases B and C due to symmetry. For the loads acting at the ends the bending moment can be calculated based on bending due to a point load on an infinitely long beam. Based on the bending moment the stress at the bottom of the beam can be calculated.
For the vertical displacement W of the beam for the cases B and C applies:

- $W = e^{\beta x}*(C1\cos\beta x + C2\sin\beta x) + e^{-\beta x}*(C3\cos\beta x + C4\sin\beta x)$

If B C1 and C2 are zero when the bending due to a point load for $x \rightarrow \infty$ will be zero. For case C applies that the bending for $x \rightarrow -\infty$ will be zero, while C3 and C4 will also be zero. With some transformations the following equations are found:

- $W(B) = (P\beta/k)*e^{-\beta x}*(\cos\beta x + \sin\beta x)$ and
- $W(C) = (P\beta/k)*e^{\beta x}*(-\cos\beta x + \sin\beta x)$.

The bending moment due to a load P = 2.75 kN can be calculated as follows:

- $M = -EI\{(2P\beta^3/k)*e^{-\beta x}*(\cos\beta x + \sin\beta x)\}$

where:

EI	$= 5.35*10^6$ N/m^2
P	$= 2.75*10^3$ N
β	$= 5.39$ m^{-1}
k	$= 18.75*10^9$ N/m^2

resulting in:

$M_{x=0}$	= 254 Nm
$M_{x=0.075}$	= 193.8 Nm
$M_{x=0.150}$	= 146.75 Nm
$M_{x=0.225}$	= 111.7 Nm
$M_{x=0.300}$	= 84.7 Nm.

The largest bending moment of 816 Nm occurs in the middle under a wheel, resulting in a flexural tensile stress of $\sigma_t = M/W = 816\ Nm/1.04*10^{-3}\ m^3 = 0.78\ N/mm^2$.

Together with the stresses due to the temperature gradient this will result in a total stress of 1.90 + 0.78 = 2.68 N/mm^2 due to an occasional combination of maximum gradient (0.0636 $^oC/mm$) and a maximum wheel load of 110 kN. The critical flexural tensile stress for a C45 strength class is however much higher (about 5.3 N/mm^2). No longitudinal cracks will be caused by these conditions.

In the second case the beam is only supported at its ends, without any bonding. This condition was also calculated, resulting in a maximum flexural tensile stress due to a 110 kN wheel load of 3.48 N/mm^2. As the stress due to the temperature difference is 1.90 N/mm^2, the total tensile stress at the bottom of the beam is 5.38 N/mm^2. This figure is higher than the maximum allowable stress based on a C45 strength class. Based on these calculations longitudinal cracks should have appeared, but were not present based on the visual inspection.

The actual flexural tensile stress caused by the temperature load at the bottom of the CRCP is probably much smaller due to friction with the asphalt interlayer, which will only increase when loaded. A good bonding will limit undesirable tensile stresses together with the load-spreading capability of the CRCP. These load-spreading capabilities are achieved by limiting the width of the transverse cracks, thus maintaining interlock and minimizing rotation of adjacent beams.

The crack pattern in a CRCP pavement is affected by the amount of longitudinal reinforcement. This makes it impossible to create a fixed spacing of the transverse cracks caused by a decrease in the concrete temperature. The cracking should not be influenced by the amount of reinforcement but by introducing crack inducers to be fixed to the asphalt interlayer. The spacing of these crack inducers should be determined in such a way that the tensile stresses at the bottom of the CRCP caused by a maximum decrease in temperature will never exceed the maximum allowable flexural tensile stress of the strength class of concrete used.

CONCLUSIONS

Begun in 1997, a monitoring program is in place on a section of the European route E35 (A12 motorway) to gather relevant data during and after construction of a CRCP pavement. This program was initiated because not enough pertinent data were available to make an objective comparison in the design stage between an asphalt and concrete structure.

The main conclusions that can be drawn based on the information gathered to date are:

- Using FWD data it can be concluded that the layer stiffnesses of all pavement layers are well beyond the modula used in the design;
- During the construction of the porous friction course a relatively high temperature gradient was observed in the CRCP. It is recommended that this observation be investigated in relation to the effect of the high wheel loads of construction traffic on the structural behavior;
- The main purpose of the reinforcement of the CRCP is to control the transverse crack spacing and crack width. Although the crack spacing is on average in line with expectations, undesirable clustering of transverse cracks (spacing of less than 0.8 m) has been observed;

- Discontinuities incorporated into the design initiate undesirable cracking;
- The bonding of the bituminous interlayer with the CRCP influences the spacing of transverse cracks: less bonding results in more widely spaced cracks; conversely, a good bond has a positive effect on the occurrence of cracks as the concrete will be supported continuously in the event of a positive temperature gradient;
- The use of crack inducers could solve the problem of the irregular transverse cracking and would combine the positive influence of bonding between the CRCP with the bituminous interlayer and controlled transverse crack spacing. The main task of the reinforcement in this case will be to control the width of the transverse cracks.

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The content of this paper reflects the views of the authors, who are responsible for the facts and accuracy of the data presented. The content does not necessarily reflect the official views and policies of the Dutch Department of Public Works.

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