

CRCP on the N49 – How the repair of the longitudinal joint leads to a better view on the crack formation of CRCP

Anne Beeldens, PhD, CE ¹; Lucie Pertry, CE ²; Pieter De Winne, CE ³; Marijn Lybaert, CE ⁴; Pascal Buys⁵

Abstract

The N49/E34 is a regional road reconstructed in 2005 by a classic design with 230 mm of CRC on an asphalt interlayer and a lean concrete base. The portion of the road near Assenede was constructed in two stages, with tie bars in the longitudinal construction joint. During construction a problem occurred with the automatic insertion of the tie bars and the contractor had to switch from inserting the tie bars into the fresh concrete to drilling and gluing the tie bars into the hardened concrete.

After some years, widening of the longitudinal joint – locally up to 40 mm – was measurable, owing to the malfunctioning of the anchorage in the joint. The closure of a stretch of the road for repairs provided an opportunity for detailed measurements of crack distribution. A difference between the fast lane, which was still bonded to the asphalt interlayer, and the slow lane, which was completely debonded, was clearly visible in the crack pattern.

This paper first analyses the problem at the longitudinal joint. In order to determine the cause of the joint widening, ultrasonic tomography was used for a non-destructive analysis, and a visual inspection was made on core samples drilled along the longitudinal joint. A second part will deal with the repair method for the longitudinal joint. A short description of the different feasible methods will be given; cross-stitching was eventually chosen. Details of the rehabilitation work in practice will be shown. Finally, the difference in crack pattern and crack distribution between the slow and the fast lane will be discussed.

The failure of the longitudinal joint on a portion of the N49/E34 provided an opportunity to investigate the behaviour of the CRCP more thoroughly, which resulted in a nice test section for measurements of the crack pattern.

Introduction

Continuously reinforced concrete pavement (CRCP) is commonly used on highways in Belgium. CRCP is known as a durable paving technique with limited maintenance needs. A classic design consists of 220 to 250 mm of CRC on an asphalt interlayer of 50 mm and a lean concrete base of 200 to 250 mm, depending on traffic volume.

In 2005, the E34/N49 regional road from Antwerp to Knokke in the western part of Belgium was partly reconstructed in CRC. The structure is shown in figure 1. The E34 is

¹ Senior Researcher at the Belgian Road Research Centre (BRRC), Belgium, Visiting professor at the Department of Civil Engineering, KULeuven, Belgium, a.beeldens@brrc.be

² Flemish Agency for Roads and Traffic, Belgium

³ Head of the Road Engineering Division at the Flemish Agency for Roads and Traffic, Belgium; Visiting professor, Department of Civil Engineering, Ghent University

⁴ Researcher at the Belgian Road Research Centre (BRRC), Belgium

⁵ CEO of ROBUCO, Belgium

a dual two-lane carriageway road with emergency lanes. Each carriageway was concreted in two stages: first the emergency and right-hand (“slow”) lanes and then the left-hand (“fast”) lane. The two stages were connected by a keyed joint system, with hooked transverse reinforcing bars driven automatically into the concrete during the first stage. After the concrete had hardened, these bars were straightened. In the direction of Knokke, between kilometre points 43,017 and 48,457, a problem occurred with the insertion system and therefore the bars were drilled and glued into the hardened concrete of the first stage.

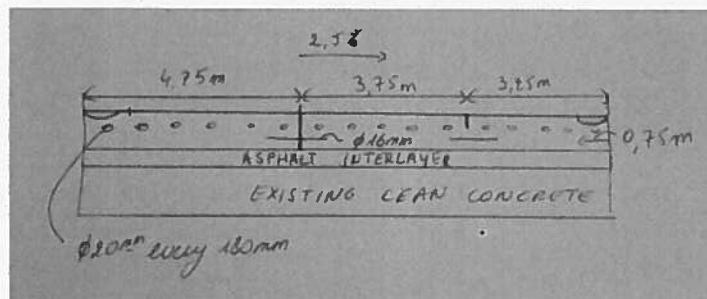


Figure1. Structure of the CRCP on the N49 at Assenede

After some years, the longitudinal joints opened widely – locally up to 40 mm –, owing to the failure of the anchorage. This paper first describes the distresses observed in the N49. After that, the repair method is discussed and, finally, the crack pattern in the N49 is analysed. The results revealed the influence of the CRCP being bonded to, or debonded from, the base by the asphalt interlayer on the final crack pattern.

Failure of the anchorage

In spite of the use of tie bars, the two lanes were found to slip away from each other, leading to the opening of the longitudinal joint. Moreover, local spalling was observed at some places at the edges of the joint, as illustrated in Figure 2.



Figure 2. Widening of the longitudinal joint and cracking over a width of 100 to 150 mm along the joint

Core drilling as well as non-destructive measurements with an ultrasonic tomograph (MIRA^R) revealed two types of distress.

In the direction of Antwerp an elongation of the tie bars took place, as can be seen in Figure 3. In order to be able to straighten the tie bars, a steel quality BE 220S was used. The bars were smooth and had a diameter of 16 mm. A reduction in diameter from 16 mm to 13.13 mm was measured in the middle of the tie bar at the longitudinal joint. In this direction the bond with the concrete was very good, but the steel quality was too low to prevent the joint from opening.



Figure 3. Elongation of a tie bar at the longitudinal joint

A different type of distress was found in the direction of Knokke. In this case the tie bars were not driven automatically into the concrete, but drilled and glued afterwards. The investigation showed that the glue had disappeared in a lot of places, resulting in no bond between the smooth bar and the concrete. Figure 4 shows the positions of three core samples drilled on one line perpendicular to the longitudinal joint. The tie bar was well anchored in the concrete of the second stage (core 1), but totally debonded in the other cases. This was due to the disappearance or lack of glue. The bars in cores 2 and 3 could be removed by hand.

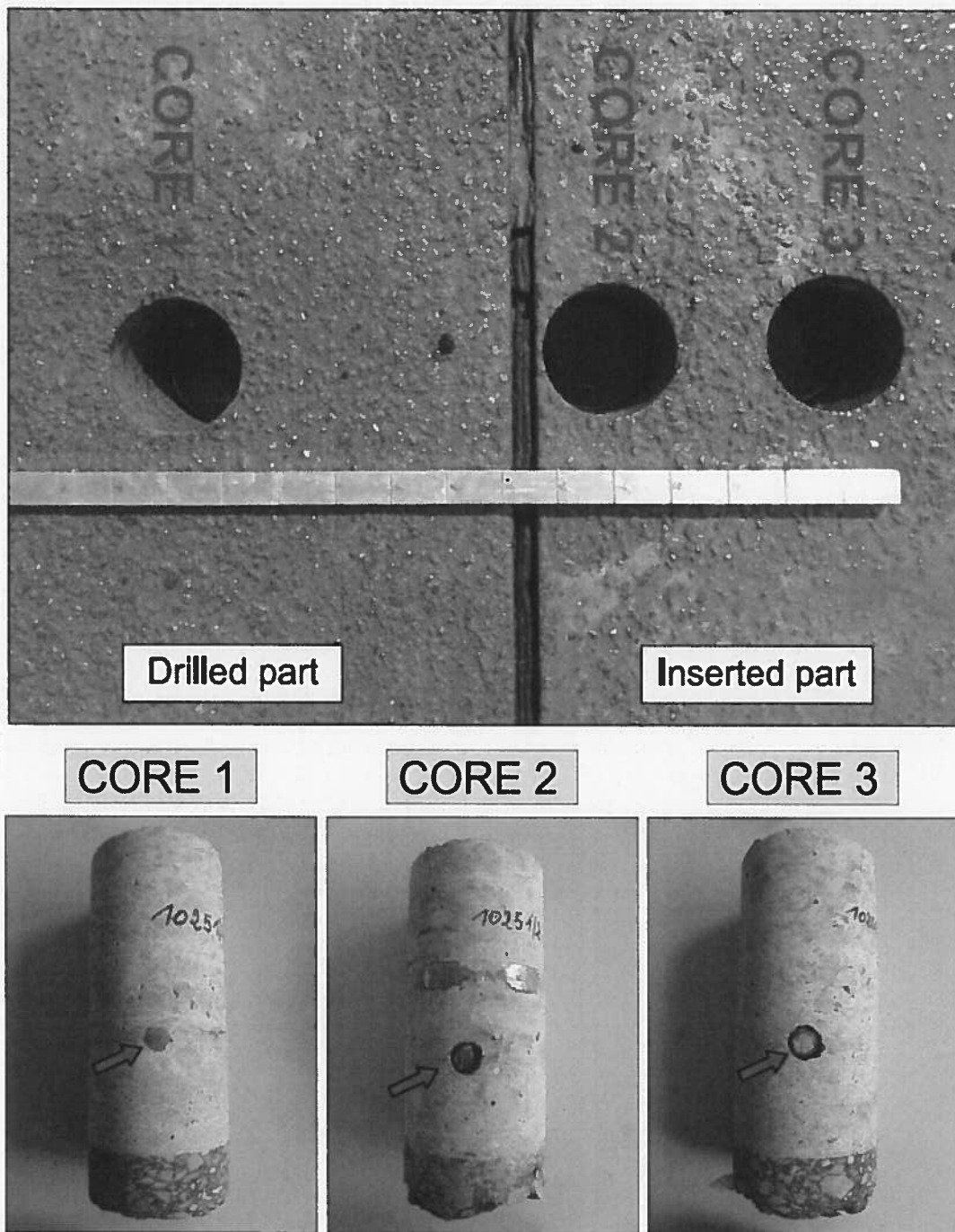


Figure 4. Positions of the core samples taken in the direction of Knokke: good bond in the core taken from concreting stage 2 (core 1), total debonding in the cores taken from concreting stage 1 (cores 2 and 3)

Although the type of anchorage is different for the two lanes, the question remains why the concrete is slipping off. The pavement was placed with a crossfall of 2.5 % towards the shoulders. No extra kerbing was used. One possibility for the slippage could be a defect in the base. Granulated iron slag containing free lime was used in the cement-treated base. This had previously caused a 'cauliflower' effect in the asphalt, owing to the swelling of the free lime. In 2005 it was decided to replace the asphalt with CRC, without changing the base. It may be that this base is still active and that, although no voids are visible beneath the concrete, this swelling is causing delamination between the base and the asphalt binder course and consequent slippage of the fast lane and the emergency lane, among other things under the impact of heavy traffic. This will be checked further in future.

In the fast lane, the bond between the asphalt interlayer and the base was still good. In the slow lane, delamination had occurred at the bottom of the asphalt interlayer. This binder course was damaged more severely along the joint (core 3), owing to the pumping effect of the water in the joint in the presence of heavy traffic.

Some core samples were taken in a place where a longitudinal crack ran alongside the longitudinal joint, as shown in Figure 2. Figure 5 shows the core taken at the end of the tie bar, over the crack. Good adhesion between the bar and the concrete is visible. As this was an exception, the bar had not been able to hold the concrete pavement by itself and a crack had formed right next to the end of the bar.

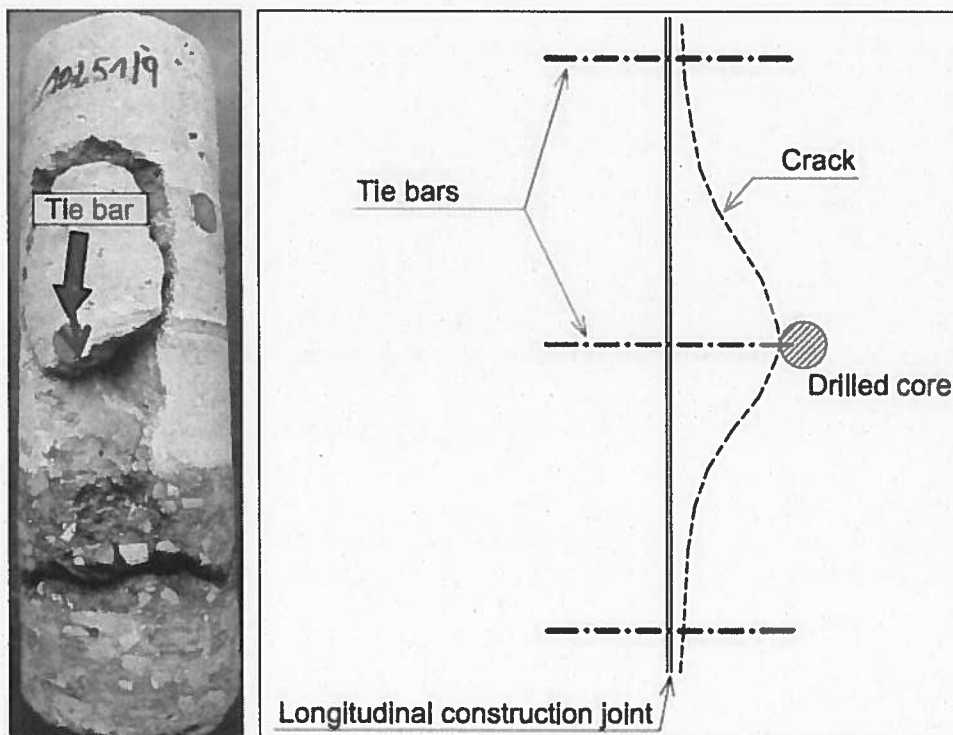


Figure 5. Core sample taken at the end of a tie bar in the presence of a longitudinal crack

Besides core drillings, non-destructive measurements were made with an ultrasonic tomograph. These measurements allowed determining the positions of the tie bars accurately, as can be seen in Figure 6. Not only the tie bar but also the longitudinal reinforcement could be located, as well as the transition between the concrete and the asphalt interlayer at a depth of 220 mm. The tie bars are situated at a depth of 150 mm and the longitudinal reinforcement at a depth of 110 mm.

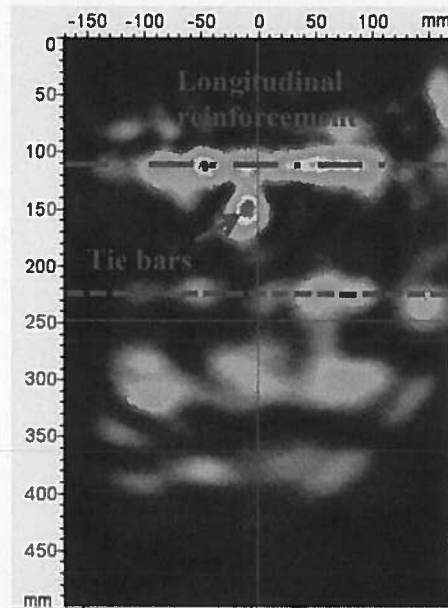


Figure 6. Measurements with an ultrasonic tomograph accurately indicated the positions of the tie bars and the longitudinal reinforcing bars and of the transition between concrete and asphalt

Repair technique

General concept. Three different types of repair had to be made: local repairs alongside the longitudinal joint where cracks had formed, anchoring the two lanes in CRC, and sealing the longitudinal joint.

Local repairs. The local repairs were made with a repair mortar. A rectangular zone was removed from the concrete. In most cases this was within a width of 150 mm, so no longitudinal bars were cut by this operation. The repairs were full-depth.

Cross-stitching. For the anchorage of the two concreting stages, two different options were considered: cross-stitching or slot stitching (M BEB R2) (IGGA, 2010). As the pavement contained longitudinal reinforcement, the slot stitching technique could not be used: the tie bars would interfere with the longitudinal reinforcement and would, therefore, cause a weaker spot rather than reinforce the pavement.

The principle of cross-stitching is to insert the bars at an angle of 35° to 45° into the concrete after filling the boreholes with epoxy, as shown in Figure 7.

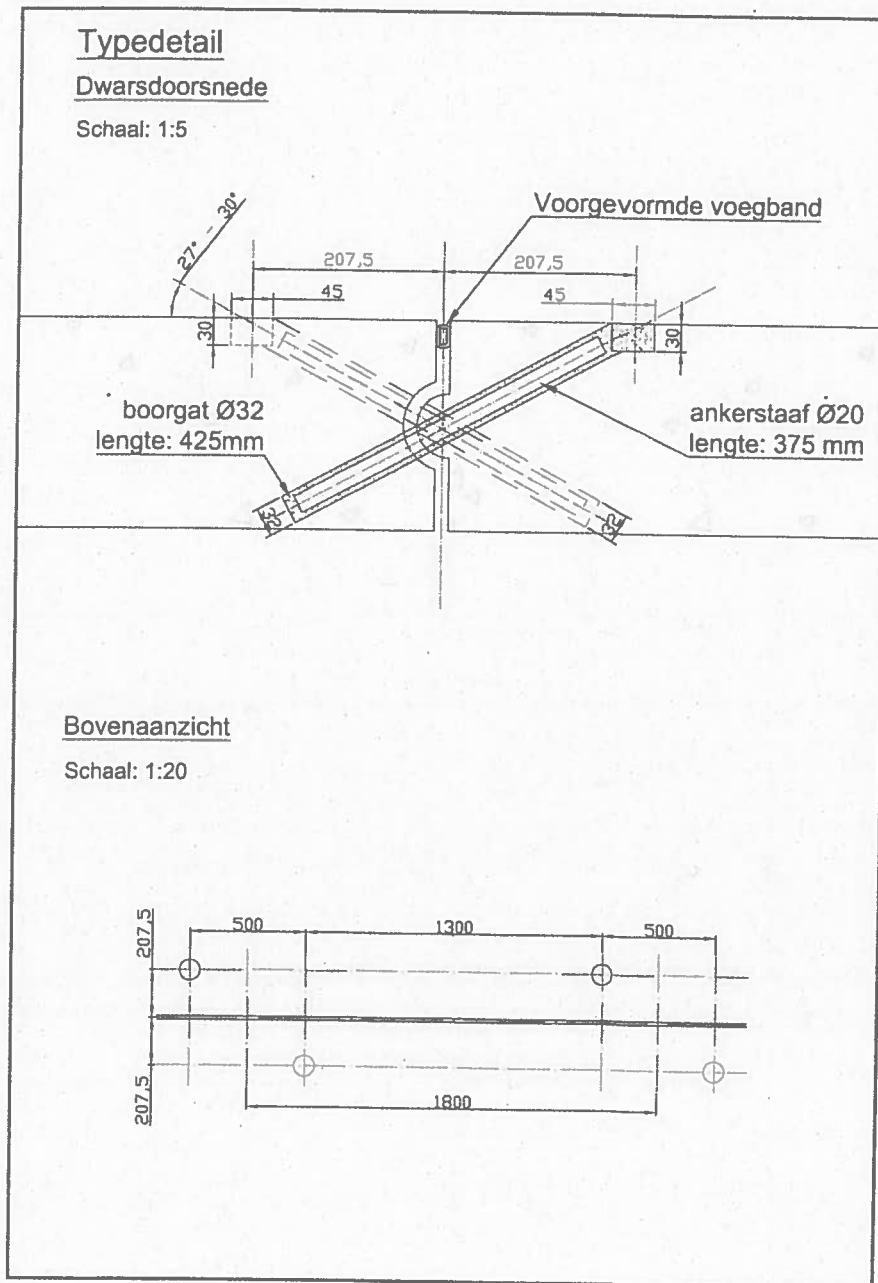


Figure 7. Principle of cross-stitching technique (will be translated in the final version)

In the case of the N49 three additional problems had to be addressed. Because of the widening of the longitudinal joint, the epoxy would tend to disappear in the joint opening. Secondly, owing to the presence of the key-and-keyway transition and of longitudinal reinforcing bars, a precise positioning of the tie bars had to be done in order not to interfere with the existing reinforcement and to provide as much adhesion length as possible. Due to the presence of the keyed joint system the adhesion length changed substantially if the tie bars were placed on one side of the longitudinal joint or the other. Finally, no cutting of existing tie bars was allowed during drilling, as that would cause too high a pressure on the concrete with cracks as a result. The following solutions were applied:

- positioning of the existing tie bars with an ultrasonic tomograph and with ground-penetrating radar. The ultrasonic tomograph was used first. Although the positions of the tie bars could be detected very accurately, the method was time-consuming as there was often a lot of interference due to the presence of the metal lid that was used to obtain the keyed transition. Ground-penetrating radar was used subsequently. Two different antennae were used. Both measurements, one at 2.6 GHz and the other at 400 MHz gave good results. The tie bars were very visible on the image obtained and the measurement could be carried out at walking speed. Figure 8 presents the images obtained with the ultrasonic tomograph and with ground-penetrating radar;

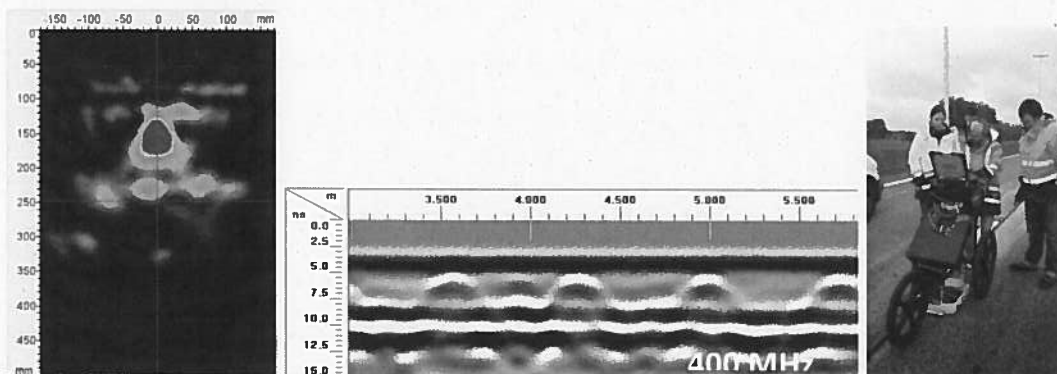


Figure 8. Images of the tie bars obtained with an ultrasonic tomograph and with ground-penetrating radar, respectively

- to avoid filling the joint with epoxy, small holes were drilled on top of the keyway and then injected with a two-component polyurethane foam. However, damage occurred during the drilling of the larger holes for inserting the tie bars, as the small cover between the joint and the first drilling hole delaminated. It was then decided to inject the joint itself with expanding polyurethane foam on the spot where the drilling was to take place, prior to the drilling of the hole;
- in order to be in the cadance of the existing tie bars, which are 800 mm apart, the interdistance was taken 1 600 mm in stead of 1800 mm per pair of stiches. The interdistance between the two tie bars was taken 500 mm and 1 100 mm was taken between the pairs;

- to avoid insufficient anchoring length, it was first decided to place the stitches at 190 mm centre to centre on the side of the key-and-keyway and at 220 mm distance on the other side. However, during the drilling of the holes it became clear that the longitudinal reinforcing bar was much closer to the joint than expected, which caused spalling of the concrete. Therefore it was decided to place the stitches on one side of the joint only in the first stage, which was also beneficial to the anchoring length;
- to avoid water interference, dams were made in the joint, which was then dried;
- the gluing was done with a thixotropic epoxy resin with high epoxy content;
- at the surface the hole was closed with a self-levelling epoxy resin. A cementitious grout was not possible here since no onset was present.

Figure 9 shows the different steps of the stitching operation.

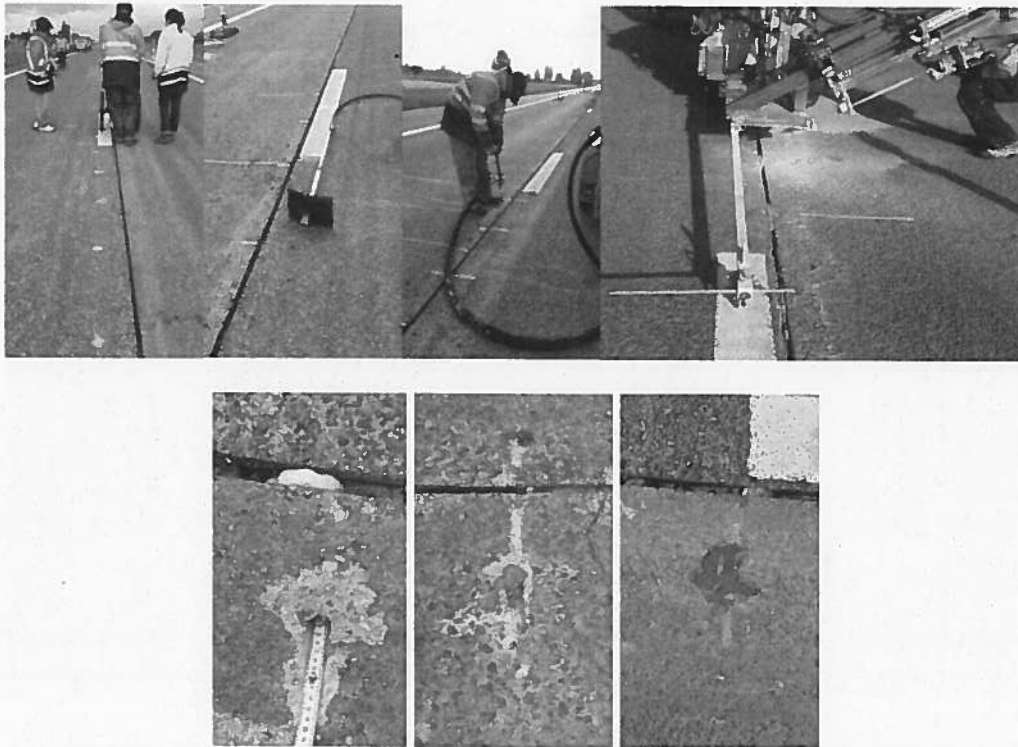


Figure 9. Different steps of stitching: positioning of the existing tie bars; cleaning of the joint (and damming); drying of the joint; filling up the joint with polyurethane foam (no picture); drilling the holes (two at the time); insertion of the tie bars in the epoxy glue; filling up the surface with acrylic resin.

In order to test the stitching system, different tension tests were carried out on tie bars placed in the concrete as can be seen in figure 10. Each time, a tensile force up to 10 ton was applied, which was the double of the required tensile force of 5 ton.



Figure 10. Tensile test on a elongated tie bar on site

Joint sealing. No traditional joint sealing material could be used, since the width of the joint varied between 10.2 mm and 44.4 mm. A flexible EPDM rubber sealant was used. Different sizes were applied, to avoid widening the joint too much. The connection between the different sizes was made by gluing the ends of the sealants together.

Crack spacing

The opportunity of closure for repairs was seized to measure the crack pattern in the slow as well as in the fast lane, in different places along the 5-km work site. In total 800 m were measured, spread over both lanes. Measurements were also made at the end locks marking the transition between the CRCP and the adjacent asphalt pavement, where typically six end locks were placed.

The results are shown in Figure 11. A similar crack spacing spectrum was taken as from the measurements on the E17 by Feys (Feys, 2010). The effect of adhesion between the CRCP and the asphalt binder course and the base is clearly visible. This is also visible in the average crack spacing, which is 0.89 m for the slow and 0.62 m for the fast lane. At the end locks, this average crack spacing increases to 1.28 and 0.97 m, respectively. A double effect is involved here. On the one hand, the higher rate of reinforcing steel (doubled at the transverse ribs) will result in more distribution of stresses and in fewer cracks appearing at the surface. On the other, the effect of the slow lane slipping is visible as well, since the poorer bond between the various layers will result in stresses building up less high.

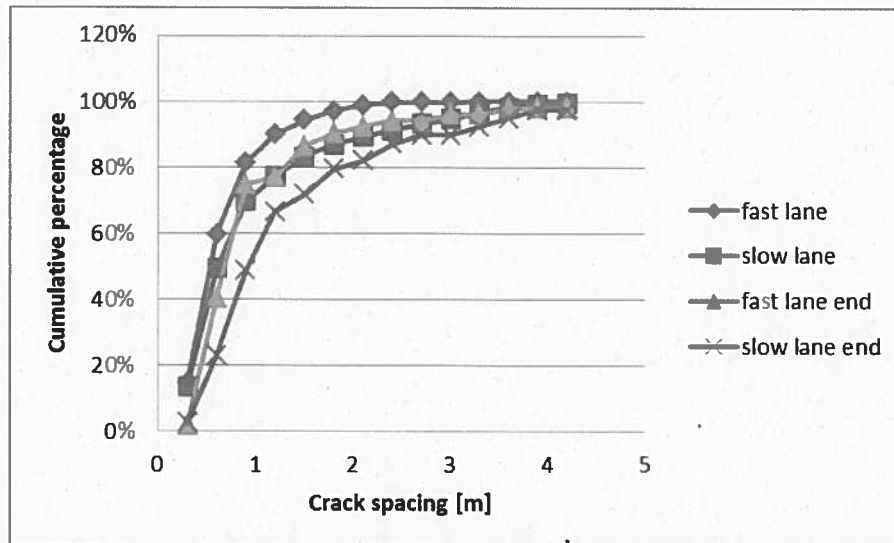


Figure 11. Crack spacing spectrum for the slow and the fast lane, with separate measurements at the end locks

Conclusions

The N49/E34 showed shortly after construction a severe opening of the longitudinal joint, owing to the lack of glue and possibly to defects in the base layer with swelling as a consequence. As the width of the longitudinal joint was at some places more than 40 mm, repair had to be done. Stitching was applied, with a precise positioning taking into account the presence of the existing tie bars, the keyed transition and the longitudinal reinforcement bars.

The crack spacing indicated the influence of the debonding of the slow lane from the base layer. More small crack spacing was found in the case the bonding was still present.

References

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- IGGA. (2010). *Stitching concrete pavement - Cross-stitching and slot-stitching*. IGGA. M BEB R2. (n.d.).

PROCEEDINGS



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FOREWORD

These are the proceedings for the Eleventh International Conference on Concrete Pavements, which was organized by the International Society for Concrete Pavements (ISCP) and held in San Antonio, Texas, USA on August 28-September 1, 2016. This is the eleventh in a series of such conferences to bring together experts from all over the world to discuss the state of the art, innovation, and new technology related to concrete pavement design, construction, evaluation, performance, and rehabilitation.

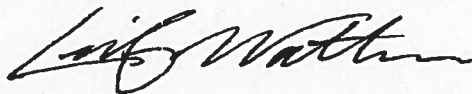
The First International Conference on Concrete Pavement Design was held at Purdue University on February 15-17, 1977. The Second International Conference on Concrete Pavement Design was held in Indianapolis, Indiana on April 14-16, 1981. Both of these conferences were planned and organized by Eldon J. Yoder, Professor of Civil Engineering at Purdue University. The Third International Conference, dedicated to his memory and his many contributions to the engineering profession, was held on April 23-25, 1985 at Purdue University. The Fourth and Fifth conferences were also held on the Purdue Campus in 1989 and 1993, respectively. The Sixth Conference was held in Indianapolis, Indiana in 1997.

After the Sixth Conference, the International Society for Concrete Pavements was formed to continue the excellence in technology transfer that Purdue University brought to this series of conferences. The Seventh Conference was held at the Disney Coronado Springs Resort in Orlando, Florida, on September 9-13, 2001. The Eighth Conference was held in Colorado Springs, Colorado on August 14-18, 2005. The Ninth Conference was held in San Francisco, California on August 17-21, 2008. Finally, the Tenth Conference was held in Québec City, Québec, Canada on July 18-21, 2012.

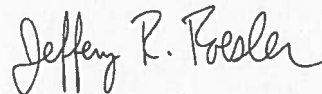
The papers presented in these proceedings were selected based on peer review from manuscripts submitted prior to November 2015. Ninety papers were accepted for this conference on the basis of these peer review comments. The papers published in these proceedings have been organized in the order as was planned for the general program, which is described beginning on page vii.

We would like to take this opportunity to thank the authors and reviewers who volunteered their time to ensure that the papers in these proceedings are of the highest quality. We wish to thank the members of the conference steering committee, the editorial committee, the student competition committee, the workshop committee and all of the workshop chairs and speakers, the entertainment committee, and the technical visit committee for their tireless efforts in developing each of the many components that comprised the overall conference program. We extend special thanks to all of the program sponsors for their financial support, and we gratefully acknowledge the contributions of the conference collaborators for their efforts in promoting the conference. We are especially appreciative of our Gold sponsors, Cemex and the Lehigh Hanson Heidelberg Cement Group.

Finally, we would like to thank: Nadine Murray of PLANit Best; Amy M. Dean for the graphic design and development of the conference program and other materials; and Sushobhan Sen of University of Illinois for editing, formatting and organizing the conference proceedings.



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