STRENGTHENING A BRIDGE DECK WITH HIGH PERFORMANCE CONCRETE

F.B.P. de Jong* and M.H. Kolstein**

*Civil Engineer, Delft University of Technology, P.O.Box 5048, 2600 GA, Delft, The Netherlands; Phone +31-15 278 57 16; f.b.p.dejong@citg.tudelft.nl and Ministry of Transport, P.O.Box 59, 2700 AB, Zoetermeer, The Netherlands; Phone +31 – 79 329 25 19 f.b.p.dejong@bwd.rws.minvenw.nl

**Senior Research Engineer, Delft University of Technology, P.O.Box 5048, 2600 GA, Delft, The Netherlands; Phone +31-15 278 40 05; m.h.kolstein@citg.tudelft.nl

Abstract

Renovation techniques have been developed both for movable and fixed bridges. A very effective solution for fixed bridges is the replacement of the mastic asphalt wearing course, which is 50 mm thick on the majority of the bridges in the Netherlands, by a layer of 50 mm reinforced high performance concrete. The paper describes the development of this renovation technique. Attention will be paid to the FEM calculations and the laboratory static and fatigue tests.

After completion of the development stage this renovation has been applied on a part the Caland bridge in the harbour area of Rotterdam. The paper describes this innovative renovation project. In five days in may 2003 the old asphaltic surfacing has been removed, the fatigue cracks in the steel deck plate have been repaired and the new surfacing of high performance concrete has been applied.

Stress spectra measurements have been performed on the Caland bridge, both before and after the application of the concrete surfacing in order to check if the prediction models based on the FEM simulations and the laboratory test are correct. In total 18 strain gauges have been applied at different locations at the bridge deck. The location of the strain gauges is explained in the paper.

The results of these stress spectra measurements are reported in this paper. The reduction of the local bending stresses in the deck plate due to the replacement of asphalt with concrete is approximately 70-80 %, which is in accordance with the expectations.

This stress reduction leads to a significant lifetime enhancement of the deck plate structure. The papers ends with an interpretation of the measured stress reductions in relation to the lifetime enhancement of the bridge deck, for two types of fatigue cracks.
RHPC as renovation technique

Fatigue phenomena are one of the greatest threats for orthotropic steel bridge decks. Especially on bridges in the Netherlands a lot of cracks have been detected the last decade. Fatigue cracks that possibly threaten the traffic safety, are cracks in the deck plate. See figure 1 for this crack type. Research has shown that these cracks arise under the enormous amount of lorries nowadays on the highways and its heavy axle loads. (De Jong, 2003, 2004-a)

The heavy axle loads of trucks generate high bending stresses in the deck plate in transversal direction. These stresses cause fatigue crack growth. For a detailed description of various crack types in orthotropic bridge decks see (De Jong, 2004-b). There are basically two options to stop or at least to lower the crack growth rate in existing bridge decks. The first is to lower the axle loads by legislation. The other is to lower the stress cycles. As lowering axle loads as impossible to achieve, the stress cycles have to be reduced.

To reduce the stress cycles in the deck plate the stiffness of the deck plate structure must be enlarged. A developed renovation technique for fixed bridges is a surfacing of high performance concrete. Fixed bridges in the Netherlands have a wearing course of approximately 50 mm mastic asphalt, with a low stiffness. It is possible to replace this with a wearing course with a higher stiffness. Reinforced High Performance Concrete (RHPC) is a material with this higher stiffness. A wearing course of reinforced high performance concrete with the same thickness as the mastic asphalt layer is a good solution to lower the stress cycles. If also a good intermediate layer between steel and concrete is possible, composite action between steel and concrete is possible. In that case the total stiffness of the composite deck plate structure might be enlarged with factors. Then the stress cycles in the steel deck plate are strongly reduced and subsequently the fatigue life is far better.

The development of the RHPC wearing course started after the fatigue problems with the Van Brienenoord bridge (Kolstein, 1998). The development is a cooperation between Contec ApS and the civil engineering division of the Dutch ministry of Transport, Public Works and Water Management. Also several other partners were involved. The development of this overlay is described extensively by (Buitelaar, 2004). In figure 2 this surfacing system is visible.

Application phases

Applying an RHPC wearing course on an existing bridge deck with a mastic asphalt wearing course is divided in several phases. The description of these phases is the result of several tests. The phases are:

1. Removing the existing mastic asphalt wearing course
2. Visual and ultrasonic inspection of the deck plate structure to find all the cracks in the deck plate
3. Repair of detected cracks if they are bigger than the repair criterion, for example by submerged arc welding of the deck plate crack
4. Shot blasting the surface of the steel deck plate to Sa 2.5 to achieve a perfect bonding between epoxy layer and steel deck plate.
5. Applying an two-component epoxy based surfacing sprinkled in with calcinated bauxite aggregate (3-6 mm)
6. After hardening of the intermediate epoxy layer, removing the unbonded bauxite granules, because unbonded granules are weak points in the adhesion between concrete and epoxy.
7. Apply reinforcement in 3 layers $\phi8$ mm spaced at 50 mm. The direction of the reinforcement bars in the bottom layer is transversal, the direction of the middle layer is longitudinal and the direction of the top layer is transversal.
8. Casting the concrete HPC strength class C110, reinforced with steel fibers and acrylic fibers.
9. Compacting, power floating and curing the surface in order to create a very dense surface.
10. Shot blasting the surface in order to create skid resistance for the traffic

Testing and development program

Before application on a bridge deck in a highway, this RHPC wearing course system is subjected to several tests and calculations. The objective of this program was to develop a system appropriate for application on a highway bridge. Only aspects, which are difficult to research by calculations and laboratory research, are left for the first application on a highway bridge.

**Bonding layer between steel and concrete.** Adhesion tests on small test specimens have been performed to optimize the epoxy-bonding layer between concrete and steel. Four types of this interface have been tested. (Buitelaar, 2004):
- Prefab panels glued in-situ on the steel deck using a two-component epoxy paste adhesive
- Casting the mortar on a wet two-component epoxy paste adhesive
- Casting the mortar on a hardened two-component epoxy paste adhesive sprinkled in with granite using both reinforced and no-reinforced samples.
Static and fatigue tests turned out that casting the mortar on a hardened epoxy layer with application of reinforcement bars was the best solution for the interface layer.

**Large application tests on a total area of 80 m2.** After the first laboratory tests the RHPC wearing course has been applied on a part of the old and removed bascule bridge Van Brienenoord. The bascule bridge was renewed in 1998 after detection of cracks in 1997. The old part is kept for research objectives. An important conclusion from these tests is that the vibration screed should add sufficient energy to the concrete in order to ensure a good compaction of the concrete in the interface between the epoxy layer and the concrete. Without a good compaction in this interface composite action is not guaranteed. After hardening of these test areas the concrete layer is shot blasted. Measurements of the skid resistance showed that this is enough to obtain the required skid resistance for traffic.

**Technological and durability tests.** Several test have been performed on technological and durability aspects of the concrete mixture. Aspects which have
been researched are: compressive strength, modulus of elasticity, flexural strength, short term shrinkage and creep, long term shrinkage and creep, effect of curing and power floating, frost/thaw resistance, chloride penetration. A description of these tests is not in the scope of this paper. For further details see (Buitelaar, 2004)

**FEM calculations and tests.** Several FEM calculations have been made in order to calculate the peak stress in the steel deck at the location of the deck plate crack. FE-models have been made without surfacing, with a mastic surfacing and with a RHPC surfacing. Besides these models also a model with glued steel plates and a model with troughs filled with polyurethane have been made. The last two models are for renovations techniques for movable bridges. The used FEM model is given in figure 3. A wheel load is placed on top of the deck plate centered above the trough profile at the location of the crossbeam. The calculated reduction of the peak stress at the location of the crack with a RHPC layer is 4 to 5 compared to a wearing course of mastic asphalt. (Pover, 2002)

Besides the FEM calculations and the lab tests on small specimens also a test on a full-scale test specimen has been performed. One of the sections of the old bascule bridge Van Brienenoord, on which an application test has been performed in an earlier stage, is subjected to static and fatigue tests. The results of the static tests on this panel are in accordance with the results of the FEM calculations. After the static test a fatigue test has been performed. The wheel load (270 * 320 mm, width *length) is again centered above the trough at the location of the crossbeam. Figure 4 gives a photo of the fatigue test. In total 8.4e6 load cycles have been applied. The first 4.2e6 with a load range of 105 kN, then 1.4e6 with a range of 137 kN, then 1.4e6 with a range of 168 kN and finally 1.4e6 cycles at a range of 210 kN. Calculations have shown that this load set up equals 75 years of heavy vehicle traffic on the Moerdijk bridge, which is the most heavy loaded bridge deck in the Netherlands. After the fatigue test an ultrasonic inspection has been performed without detecting any crack in the steel deck plate. The conclusion is that the replacement of the asphalt surfacing by a RHPC surfacing is a promising solution with respect to the fatigue life of orthotropic steel bridge decks.

**Pilot Project Caland bridge**

After the successful full-scale fatigue test the next step in the development of the RHPC surfacing was application in a real highway bridge. The selected bridge for the first application was the Caland bridge, a bridge in the harbor area in Rotterdam. This bridge is one of the bridges with severe fatigue problems and also replacement of the asphalt surfacing was desirable. This bridge has two fixed spans and a movable part, see figure 5. One direction of the shortest fixed span (86m) is renovated with a RHPC surfacing. This direction has two lanes. The area of the RHPC surfacing is approximately 650 m2, 7.6 m wide by 86 m long. This renovation is done from 29 April 2003 to 4 May 2003.

The phases of this renovation project are described in this paper. In figures 6 to 20 a brief outline of this project is given. At first the mastic asphalt layer is removed (figure 6 and 7). After the removal of the asphalt a covering is placed over
the whole working area (figure 8) to ensure that rainfall doesn't cause delay or a worse quality of the concrete surfacing. Subsequently the ultrasonic TOFD inspection is performed (figure 9). This is a very accurate technique, small cracks with a height of approximately 2 or 3 mm are found by this technique. The cracks, which are found, are identified on the top of the steel deck plate (figure 10). Some of these cracks had to be repaired. These cracks are repaired with the submerged arc welding procedure (figure 11 and 12). After the repair the new weld is magnetic tested (figure 13). After these repairs the steel deck plate is shot blasted to Sa 2.5 (figure 14) and immediately the two-component epoxy is applied and sprinkled in with bauxite (figure 15.) After the hardening of the epoxy layer the unbonded bauxite granules are removed and the reinforcement is placed (figure 16 and 17). Then the concrete is casted on the bridge deck and compacted with a vibration screed (figure 18). The thickness of the concrete layer is 60 mm. T-profiles of 60 mm high at the edges of the concrete layer are the supports for the screed. With power floating (figure 19) the surface is made denser in order to prevent for shrinkage cracks in the concrete. The concrete is shot blasted for skid resistance, the covering is removed and then the bridge is opened for traffic. The concrete layer is kept wet for one week after opening for traffic. This was also to minimize the shrinkage (figure 20).

**Stress spectra measurements**

**Objective.** Due to the replacement of the asphalt layer by a layer of RHPC the stresses in the steel deck plate reduce. This reduction is calculated with FEM models and verified with static measurements on full-scale laboratory tests. To verify these results stress spectra measurements have been performed on the Caland bridge. These measurements are extensively described in (Kolstein, 2003). The stress spectra have been measured under a conventional mastic asphalt wearing course and under the RHPC surfacing. Evaluation of the measurements gives a reduction due to the applied RHPC surfacing.

**Instrumentation.** In September 2002 strain gauges have been applied at the underside of the bridge deck structure at the right wheel track of the heavy vehicle lane. The span of the fixed span is 86 m and the strain gauges have been applied at approximately 74 to 76 m in traffic direction. Approximately 10 m before lorries are leaving the bridge. At this location dynamic amplification is limited. A general view of the measured cross sections is shown in figure 21. At 9 locations a pair of 2 strain gauges have been attached. These strain gauges are positioned 15 mm from the weld toe and they are all positioned perpendicular to the longitudinal weld between trough web and deck plate. This means that the gauges all measure the transversal strain, figure 22. As shown in figure 23 the strain gauges near the crossbeam are located 20 mm from the crossbeam web. The numbering of the strain gauges is shown in figure 24, 25 and 26. The location of the strain gauges is chosen so that the stress reduction both for deck plate cracks as well as for cracks in the longitudinal weld between deck plate and trough web is measured. Besides the strain gauges two temperature sensors have been applied. One is connected at the bottom side of the steel deck plate and one is hanging in the free air.
Maximum stress spectra. The induced stress variations at the strain gauge locations have been measured under normal traffic conditions. Two periods of measurements have been carried out.

- Week 10-17: Steel deck plate with a conventional mastic asphalt surfacing, before application RHPC surfacing
- Week 19-31: Steel deck plate with a RHPC surfacing

Each axle of a heavy vehicle that passes the bridge generates a strain signal. Only the maximum strain is saved. From the measurements only the weeks 14-17 and 19-23 have been analyzed. In week 18 the Caland bridge is renovated.

The results of the measurements are given for the strain gauges at location B. Because the results at this location mid span between two crossbeams are less influenced by second order effects. Second order effects influence the results of the strain gauges at locations A and C. These effects are due to the extra stiffness of the crossbeam web. This was inevitable because it was impossible to attach the strain gauges at bottom side of the deck plate at the inner side of the trough profile.

The results are given in frequency curves. These curves give the cumulative number of stress cycles with a range larger than the given stress range. The results for the strain gauges at location B are given in figures 27, 28 and 29 for the strain gauges 7, 9 and 11 at the bottom side of the deck plate and in figures 30, 31 and 32 for the strain gauges at the trough web.

Stress reduction. A strong reduction of the stresses is visible from the figures. Only at the tails of the curves, a smaller or even no reduction is visible. This is due to the effect that a passing vehicle not only generates local stresses in deck plate and trough web, but also more global stresses, e.g. due to the crossbeam behavior. As the reduction of the local stresses due to RHPC is the main concern the tails of the frequency curves are neglected in the analysis.

To derive an indication of the stress reduction due the RHPC surfacing the cumulative numbers of cycles for RHPC are multiplied with a factor 0.8, because the curves in the figure 27 to 32 comprise data for 4 weeks of mastic asphalt and 5 weeks of RHPC. Then the stress range from the frequency curve is determined for the cumulative numbers of 100, 1000, 10000 and 100000 cycles. For mastic asphalt these values are given in table 1, for RHPC these values are given in table 2. The stress reduction factor is defined as the stress for mastic asphalt divided by the stress for RHPC at the same number of cumulative cycles. These factors are given in table 3.

From the figures and the tables becomes clear that a significant stress reduction is achieved by using a RHPC surfacing. For the strain gauges 11 and 12 a smaller reduction is achieved. This is due to the fact that these gauges are placed a little bit outside the wheel track, which means that the global behavior becomes more visible. The higher stress reduction factors for a cumulative number of 100000 cycles are also due to this phenomenon. Based on these measured stress spectra, an average reduction factor for the stress in the deck of 0.2 is derived and an average reduction factor of 0.4 for the stress in the trough web is derived.
Table 1. Mastic asphalt stress ranges (MPa)

<table>
<thead>
<tr>
<th>Cycles</th>
<th>Gauge 7</th>
<th>Gauge 8</th>
<th>Gauge 9</th>
<th>Gauge 10</th>
<th>Gauge 11</th>
<th>Gauge 12</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>116</td>
<td>146</td>
<td>105</td>
<td>149</td>
<td>62</td>
<td>92</td>
</tr>
<tr>
<td>1000</td>
<td>95</td>
<td>117</td>
<td>85</td>
<td>113</td>
<td>46</td>
<td>72</td>
</tr>
<tr>
<td>10000</td>
<td>67</td>
<td>75</td>
<td>62</td>
<td>68</td>
<td>29</td>
<td>47</td>
</tr>
<tr>
<td>100000</td>
<td>31</td>
<td>36</td>
<td>28</td>
<td>29</td>
<td>11</td>
<td>20</td>
</tr>
</tbody>
</table>

Table 2. RHPC stress ranges (MPa)

<table>
<thead>
<tr>
<th>Cycles</th>
<th>Gauge 7</th>
<th>Gauge 8</th>
<th>Gauge 9</th>
<th>Gauge 10</th>
<th>Gauge 11</th>
<th>Gauge 12</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>15</td>
<td>39</td>
<td>18</td>
<td>42</td>
<td>15</td>
<td>36</td>
</tr>
<tr>
<td>1000</td>
<td>13</td>
<td>33</td>
<td>15</td>
<td>34</td>
<td>13</td>
<td>31</td>
</tr>
<tr>
<td>10000</td>
<td>9</td>
<td>25</td>
<td>11</td>
<td>26</td>
<td>9</td>
<td>25</td>
</tr>
<tr>
<td>100000</td>
<td>4</td>
<td>14</td>
<td>7</td>
<td>17</td>
<td>0</td>
<td>16</td>
</tr>
</tbody>
</table>

Table 3. Stress reduction factor

<table>
<thead>
<tr>
<th>Cycles</th>
<th>Gauge 7</th>
<th>Gauge 8</th>
<th>Gauge 9</th>
<th>Gauge 10</th>
<th>Gauge 11</th>
<th>Gauge 12</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>0.13</td>
<td>0.26</td>
<td>0.17</td>
<td>0.28</td>
<td>0.25</td>
<td>0.40</td>
</tr>
<tr>
<td>1000</td>
<td>0.14</td>
<td>0.28</td>
<td>0.17</td>
<td>0.30</td>
<td>0.27</td>
<td>0.43</td>
</tr>
<tr>
<td>10000</td>
<td>0.14</td>
<td>0.34</td>
<td>0.18</td>
<td>0.39</td>
<td>0.31</td>
<td>0.53</td>
</tr>
<tr>
<td>100000</td>
<td>0.13</td>
<td>0.38</td>
<td>0.26</td>
<td>0.59</td>
<td>0.00</td>
<td>0.79</td>
</tr>
</tbody>
</table>

A simple approach of the lifetime enhancement due to this stress reduction uses the regular slope $m=3$ of the fatigue detail curves. The lifetime enhancement factor in this simple approach is $1/(0.2^3) = 125$ for fatigue cracks of the deck plate and $1/(0.4^3) = 16$. This is a conservative approach because a significant part of the stress cycles is in the region of the slope $m=5$ or even below the fatigue limit. In a more detailed study of the lifetime enhancement this should be researched, and also the effects of the global stresses.

Conclusions

The use of RHPC as bridge deck surfacing is a good alternative for a conventional asphalt surfacing. The pilot project on the Caland Bridge has shown that it is a good and workable solution, which can be applied in a few days. Replacement of the surfacing of mastic asphalt with a RHPC surfacing reduces the stresses in the deck plate with approximately 80% and the stresses in the longitudinal weld between deck plate and trough web with approximately 60%. Therefore it might be concluded that the use of a RHPC surfacing is a very effective solution with respect to the fatigue behavior of the bridge deck structure.

References


Figures

Figure 1. Deck plate crack

Figure 2. RHPC surfacing
Figure 3. FEM model stress calculation

Figure 4. Full-scale fatigue test RHPC surfacing
Figure 5. Caland bridge

Figure 6. Removing asphalt

Figure 7. Removing asphalt

Figure 8. Placing covering

Figure 9. Ultrasonic TOFD inspection

Figure 10. Found crack
Figure 11. Submerged arc welding

Figure 12. Repaired fatigue crack

Figure 13. Magnetic testing

Figure 14. Shot blasting deck plate

Figure 15. Epoxy layer

Figure 16. Reinforcement
Figure 17. Reinforcement

Figure 18. Casting concrete

Figure 19. Power floating

Figure 20. Finished RHPC surfacing
Figure 21. General view of the measured cross sections A, B and C.

Figure 22. Strain gauge positions near the weld at cross section A, B and C.
Figure 23. Strain gauge positions near the crossbeams at cross section A and C

Figure 24. Numbering of the strain gauges at cross section A
Figure 25. Numbering of the strain gauges at cross section B

Figure 26. Numbering of the strain gauges at cross section C
Figure 27. Frequency curves strain gauge 7 at deck plate

Figure 28. Frequency curves strain gauge 9 at deck plate

Figure 29. Frequency curves strain gauge 11 at deck plate
Figure 30. Frequency curves strain gauge 8 at trough web

Figure 31. Frequency curves strain gauge 10 at trough web

Figure 32. Frequency curves strain gauge 12 at trough web