MAINTENANCE PHILOSOPHY AND SYSTEMATIC LIFETIME ASSESSMENT FOR DECKS SUFFERING FROM FATIGUE

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Abstract

This paper consists of two parts. The first part describes a new kind of maintenance philosophy. Conventional maintenance strategies are formed by time based inspection intervals and replacement of parts of the structure at the end of their lifetime. This strategy and the insufficiency of this strategy to handle fatigue problems are described. Subsequently the properties of a new risk based maintenance strategy, based on probabilistic calculations models, are described. Successful implementation of this strategy is only possible if the necessary tools in this strategy are available. The paper describes the tools in the strategy. These are lifetime calculation models, local reparation techniques, renovation techniques for complete bridge decks and inspection techniques. This strategy and the developed tools have resulted in practical solutions for the 80 fixed and movable bridges in the Netherlands, which enables the bridge owners to maintain their bridges, minimizing the risks and maintenance costs.

The second part of the paper describes more in detail the lifetime calculation model, which is of vital importance to minimize risks and to schedule inspections and renovations. Reliable lifetime calculations need accurate models of the number and amplitude of the stress cycles and the fatigue behaviour. A formal model to calculate the fatigue crack growth of deck plate cracks is presented. Based on this formal model a computer program has been developed for bridges in the Netherlands. Aspects related to the number and amplitude of the cycles are investigated. These are traffic properties, axle load distributions, the fatigue classification and the calculation of the stresses at the crack location. Special attention has been paid to the effect of the asphalt wearing courses, which are applied at fixed bridges. Asphalt has a reducing effect on the stress in the steel deck plate, but
due to the temperature dependency of the material the modelling is complicated. The paper describes how this effect is taken into account. The formal model is implemented in a computer program and the results of the lifetime calculations are in accordance with several detected fatigue cracks on Dutch steel bridge decks.

**Maintenance Philosophy**

*Introduction.* The conventional maintenance strategy for orthotropic steel bridge decks is formed by time-based inspections of the bridge structure and the replacement of parts of the structure when these parts are at the end of its lifetime. Up to the moment of the observed fatigue problems in the Van Brienenoord Bridge (De Jong, 2003-a, 2004-a) this means that once in five year an inspection of the total structure is performed (asphalt surfacing, deck structure, crossbeams, main girders, structural bearings, coating etc.) The elements that are replaced on regular basis were up to the Van Brienenoord case only the asphalt surfacing, the structural bearings and the coating.

After the problem with the Van Brienenoord Bridge was solved two questions had to be answered in relation to the maintenance philosophy: What are possible reparation techniques for observed fatigue cracks, which need to be repaired very quickly? What are solutions to enhance the lifetime of the total bridge deck structure? The local repair techniques are necessary up to the moment that a total renovation is applied.

*Conventional maintenance.* As stated before practice up to now is conventional maintenance. Conventional maintenance is formed by time-based inspections of the bridge structure and the replacement of parts of the structure when these parts are at the end of its lifetime. The inspections are intended to make decisions about replacement of parts of the structure. The left circle of figure 1 gives a model of this kind of maintenance. This kind of maintenance is appropriate, provided that the time based interval is long and failure of elements within this interval doesn’t cause structural failure.

In case of the fatigue problems on orthotropic steel bridge decks both requirements are not met. Experiences on steel bridges in the Netherlands have shown that monthly visual inspections on the steel bridge deck structure are necessary on the most heavily loaded bridge structures with fatigue cracks. Besides these expensive inspections also expensive emergency reparations of the deck structure are needed in case severe fatigue cracks are observed in the deck plate. Due to the nature of the fatigue phenomenon we can expect that the number of inspections and reparations will always increase. Therefore the conclusion must be that conventional maintenance is no longer appropriate, so the maintenance philosophy has to be changed to a risk based strategy.

*Risk based maintenance.* In changing from conventional maintenance to a risk based maintenance strategy two major changes can be observed. The time based inspection interval has to be replaced by an interval based on a lifetime calculation. Systematic lifetime calculations for fatigue cracks are very important for a risk-based strategy.
On the other hand local (emergency) reparations have to be replaced by renovation of the total bridge deck structure. An example of renovation is the replacement of the asphalt wearing course by a layer of high performance concrete (Buitelaar, 2004, De Jong, 2004-b).

Outline of the PSR Project

Objectives. To tackle the fatigue problem and find solutions for existing and future bridges the Dutch ministry of Transport, Public Works and Water management started a research project PSR. PSR is the acronym of Problematiek Stalen Rijdekken, which is Dutch for Problems Steel Bridge Decks. The objectives of the PSR project are:

- Develop knowledge of the specific cracking mechanisms to stop the fatigue problem on the existing bridges
- Develop a design philosophy which would prevent these problems on new bridge deck designs
- Knowing the condition on fatigue of our main steel orthotropic highway bridges both by inspection and by fatigue lifetime calculations, based on a new developed probabilistic design philosophy
- Develop knowledge of crack propagation, of repairing discovered cracks and knowledge of renovating bridge decks

For a more extensive description of this project see (Boersma, 2003).

These research tasks were carried out by six task forces that worked together closely to pay full attention to all project aspects and to build one integral overall solution: A system in which we founded the knowledge to deal with the existing fatigue problems and which would prevent future problems for new designs. A short description of these task forces follows:

Inspection techniques. This task force had to research and develop existing or new inspection techniques, which would be suitable for inspecting fatigue cracks in an early stage of growth. Also there was a task to enhance the techniques for inspecting from beneath the bridge deck or even through a thin (app. 8 mm) or thick (app. 5cm) wearing coarse on top of the deck.

Lifetime calculations and asphalt behaviour. This task force had to make fatigue calculations for the 80 bridges.

Reparation techniques. This task force had to research and develop reparation techniques and procedure for the different fatigue cracks. It was a challenge to develop a reparation technique which would be superior to the original construction for fatigue load and which would minimize the heat input during welding activities.
Solutions for enhancing lifetime. This task force had to research and develop smart ideas, which would enhance the lifetime of existing bridge deck constructions.

Design rules and Philosophy. This task force worked closely together with the TNO Building Research Institute, which we requested to assist us with the development of a probabilistic design philosophy.

New designs. This task force tried to develop new concepts for lightweight bridge decks. Some concepts have been developed, but the general conclusion was that the standard orthotropic bridge deck is a good concept, provided that the chosen plate thicknesses are based both at static and fatigue calculations with current traffic properties.

Storage system. It is important to collect all relevant information, to record and to make it accessible for different users. The project results are stored in different books, see table 1. In this way future enhancements would be possible and a central coordination on all the information would be a fact. A distinction is made between the design and the maintenance department. Books 1 and 2 are intended for both departments. Books 3, 4 and 5 are specific design parts.

Table 1. Overview books, project results and tools maintenance strategy

<table>
<thead>
<tr>
<th>Book</th>
<th>Title</th>
<th>Parts</th>
</tr>
</thead>
<tbody>
<tr>
<td>BOOK 1</td>
<td>System description and philosophy</td>
<td>Flow chart for fatigue problem approach</td>
</tr>
<tr>
<td></td>
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<td>Overview of the defined fatigue cracks</td>
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<td></td>
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<td>Flow chart of inspection, maintenance and preservation strategy</td>
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<td></td>
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<td>Priority list of bridges</td>
</tr>
<tr>
<td></td>
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<td>Examples of different cases</td>
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<tr>
<td>BOOK 2</td>
<td>Type of fatigue cracks</td>
<td>Cracking definitions/identification</td>
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<tr>
<td></td>
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<td>Reparation information and procedures</td>
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<tr>
<td></td>
<td></td>
<td>Inspection information and procedures</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Lifetime enhancement solutions</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Examples on various topics</td>
</tr>
<tr>
<td>BOOK 3</td>
<td>Lifetime calculations</td>
<td>Theory and specific information on lifetime calculations</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Methods and computer models to determine lifetime</td>
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<td></td>
<td></td>
<td>Design lifetime regarding safety issues and inspection</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Background studies, Asphalt effects</td>
</tr>
<tr>
<td>BOOK 4</td>
<td>Solutions for lifetime enhancement</td>
<td>Theory and information to select the appropriate method</td>
</tr>
<tr>
<td></td>
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<td>Information for cost-effective decision making</td>
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<td>FE Analysis of the chosen solution</td>
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<tr>
<td></td>
<td></td>
<td>Background reports, studies and tests</td>
</tr>
<tr>
<td>BOOK 5</td>
<td>Design</td>
<td>Design philosophy for steel bridge decks</td>
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</table>
The decision for the optimal cost-effective preservation method of a bridge-deck is a critical process and needs a close cooperation between the disciplines design and maintenance from which the practical information like inspection results is collected. Future use, loads and distributions, the extra required lifetime, the technical state of the construction are parameters that influence the possibilities for lifetime enhancement. Also, the costs of the intermediate (safety) inspections in relation to the required reliability have to be considered.

Results PSR Project

Lifetime calculation and asphalt effects. First of all we started to focus on the project scope. Which bridges would be part of the project and what specific fatigue cracks would be considered? About 80 steel bridges, both fixed and movable with orthotropic steel decks, built from 1925 to 1998 were adopted. We decided to focus on about 10 main fatigue cracks. High priority was given to the deck plate crack.

At this moment we understand the fatigue mechanisms for the various bridge deck details and we can also calculate the fatigue lifetime in respect to traffic load and distribution. A special computer program was written for specific fatigue cracks. Depending on location (the type of traffic and the specific numbers) and the construction details of a bridge deck the lifetime can be calculated. Traffic loads and numbers have been measured at various locations. The present program calculates lifetime predictions and inspection moments for different safety levels. The results of lifetime predictions are in accordance with detected fatigue cracks. We developed a philosophy and system to maintain our 80 bridges and prevent deck-plate cracking. This system is based mainly on inspection, monitoring of existing cracks and renovation techniques.

Results on reparation techniques. The task force reparation techniques developed several solutions; from emergency to qualitative reparations which suit our design philosophy. It is important that the repair procedure is described detailed. In the emergency cases a steel plate is simply welded upon the cracked section on the steel deck, see figure 2. This is only for a short period of time in which a definitive reparation can be planned and organised. The deck-plate solution is an example of a definite solution and it is also used to replace a temporary solution. In this solution the existing steel deck plate is replaced by a thicker one on the cracked location, see figures 3 and 4.

Results on inspection techniques. Because the deck plate fatigue crack initiates at the root of the longitudinal weld between trough and deck plate, which is at the inner side of the trough stiffener it was necessary to develop alternative inspection methods. Two Pulse Eddy Current inspection techniques have been developed that detect through the deck plate grown cracks through a wearing coarse up to 5 cm. Critical in
the performance so far is that only cracks can be found that have fully propagated through the deck plate. This may be useful for safety inspections but it is not adequate for all applications of lifetime enhancing techniques. Also an ultrasonic based inspection technique has been developed, which is applicable from the underside of the bridge deck. Without interrupting traffic it is possible to inspect or monitor on specific locations. However, the accessibility of the structure is important and can be expensive, especially for monitoring purposes. The very reliable, but only on blank steel decks usable TOFD (Time Of Flight Diffraction) based inspection technique has been optimised and recorded in a specific inspection procedure.

**Results on lifetime enhancement.** To enhance the lifetime of the existing bridges rehabilitation techniques have been developed and tested for the bridges. FEM-analysis, laboratory research and full-scale static and fatigue tests were parts of the research program.

A very simple and useful technique for fixed and movable bridges is shifting the road lanes about 60 cm. It is by far the most cost effective method but at the same time you only gain a factor two at a time. If there’s enough space on the bridge deck you can repeat this method without taking any traffic measurements.

An effective technique for movable bridges is filling two troughs under the heavy loaded wheel tracks in the heavy vehicle lane with a special developed epoxy/cork mixture. The stresses in the deck plate are decreased by factor two or three and the failure mechanism will be blocked. Another effective technique for movable bridges is based on gluing a 5 mm thin steel plate onto the existing deck plate. The stresses in the deck plate are decreased by a factor two or three.

A very effective solution for fixed bridges is the replacement of the mastic asphalt wearing coarse by a reinforced high performance concrete (RHPC) B110 overlay, which is bonded to the bridge deck. Due to the composite interaction between the RHPC overlay and the steel deck plate it is a very effective solution. FEM analysis, laboratory tests and field tests showed that the stress on critical places could be reduced by a factor four to five. Research continued with a fatigue test on a full-scale specimen. In this test the Moerdijk traffic load spectrum was simulated for 75 years, without causing any damage.

**Lifetime calculations**

**Formal model.** Reliable lifetime calculations are important. Accurate information of the fatigue behaviour of details and accurate information of the loading history is necessary. This part of the paper gives a formal model to calculate the fatigue damage for the deck plate crack at the location of the crossbeam. The given numerical values are valid for highway bridges in the Netherlands. For other countries these values may differ. The total fatigue damage is given by:

\[
D_{\text{total}} = \sum_{i=A,B,C} \int \int \int D(t,x,Q) \, dt \, dx \, dQ
\]

with:
D fatigue damage
i wheel type; A = single, B = double, C = super single
t time [hours]
x position in transversal direction
Q wheel load [kN]

The fatigue damage $D$ is given by:

$$D(t, x, Q) = \frac{n_i}{N_i}$$

with:

$N_i$ the number of axle loads at $D = 1$

$n_i$ the number of axle loads given by:

$$n_i = f(Q) \cdot f(x) \cdot f(t) \cdot n_{\text{axle/lorry}}$$

with:

$f(Q)$ the lognormal distribution function of the wheel loads
$f(x)$ the normal distribution of the location of the wheel in transversal direction, with $\sigma = 150$ mm. This distribution is based on measurements described in (Vrouwenvelder, 1998)
$f(t)$ the distribution function for the traffic over the time

$n_{\text{axle/lorry}}$ the average number of axles per lorry, for highway bridges in the Netherlands $n_{\text{axle/lorry}} = 4.05$. This value is based on fatigue load model 4, traffic type long distance (from Eurocode 1991:2, 2003)

The lognormal distribution of the wheel loads is given by:

$$f(Q_i) = \frac{1}{\sqrt{2\pi} \cdot \sigma_i} \frac{1}{Q_i} \exp \left( -\frac{(\ln(Q_i) - \mu_i)^2}{2\sigma_i^2} \right)$$

with:

Different values for $\mu_i$ and $\sigma_i$ for the three different wheel types given by:

$\mu_A = 3.22$ and $\sigma_A = 0.240$ for wheel type A (single)
$\mu_B = 3.40$ and $\sigma_B = 0.475$ for wheel type B (double)
$\mu_C = 3.22$ and $\sigma_C = 0.375$ for wheel type C (super single)

The distribution type and the parameters have been measured on several Dutch highway bridges (Vrouwenvelder, 2000-a and 2000-b)

The distribution function for the traffic over the time is given by:

$$f(t) = f_{\text{day}}(t) \cdot n_{\text{lorry/day}} \cdot (1 + g)^{(t-t_0)} \cdot p_i \cdot \gamma_n$$
with:

- \( f_{\text{day}} \) distribution function for the traffic flow over 24 hours
- \( n_{\text{lorry/day}} \) the number of lorries in 24 hours at the reference time \( t_0 \)
- \( \gamma_n \) growth of the traffic volume, on Dutch bridges 3-5 % per year
- \( p_i \) the part of the traffic for the three wheel types. On Dutch bridges \( p_A \approx 0.3 \), \( p_B \approx 0.4 \) and \( p_C \approx 0.3 \)
- \( \gamma_n \) safety factor for the number of stress cycles. \( \gamma_n = 1.0 \) for a reliability level of \( \beta = 0.0 \), \( \gamma_n = 1.03 \) for \( \beta = 2.0 \) and \( \gamma_n = 1.06 \) for \( \beta = 3.6 \). The values for \( \gamma_n \) are very low, because the number of cycles is the less dominating parameter and the variance is very low, due to a very dense network of traffic flow measurement points.

The distribution of the traffic flow over 24 hours is based on several measurements on the Dutch highways and given by:

\[
f_{\text{day}}(t) = \frac{1}{\text{hrs}_{\text{day}}} + 0.03 \cdot \sin \left( \frac{t \cdot 2\pi}{\text{hrs}_{\text{day}}} - \frac{\pi}{2} \right) - 0.005 \cdot \sin \left( \frac{t \cdot 6\pi}{\text{hrs}_{\text{day}}} - \frac{\pi}{2} \right)
\]

The number of axle loads at \( D = 1 \) is given by:

\[
N_i = N_c \cdot \left( \frac{\Delta\sigma_c}{\Delta\sigma_i} \right)^m
\]

with:

- \( N_c \) \( = 2 \times 10^6 \) conform Eurocode 3
- \( \Delta\sigma_c \) detail classification
- \( \Delta\sigma_i \) actual stress
- \( m \) slope of the fatigue detail curve

The detail classification \( \Delta\sigma_c \) for the deck plate crack at the location of the crossbeam is derived by crack growth calculations based on fracture mechanics and on fatigue tests. The crack growth calculations have resulted in a classification of 115/156 MPa for design/mean value (Dijkstra, 1998, De Jong, 2004-c). The fatigue tests have resulted in a classification of 147/197 MPa for the same crack dimension (De Jong, 2004-c). At this dimension the crack depth just equals the deck plate thickness and the length at the top side of the deck plate is 0 mm.
\[ m = \begin{cases} 
3 & \text{for } \Delta \sigma_i > \Delta \sigma_c \\
5 & \text{for } 0.55 \cdot \Delta \sigma_c < \Delta \sigma_i < \Delta \sigma_c \\
\infty & \text{for } \Delta \sigma_i < 0.55 \cdot \Delta \sigma_c 
\end{cases} \]

The actual stress \( \Delta \sigma_i \) is given by:

\[
\Delta \sigma_i = \frac{M_L(x)}{W_{steel}} \cdot f_{dyn} \cdot f_{stat} \cdot r(t) \cdot \gamma_{\Delta \sigma 1} \cdot \gamma_{\Delta \sigma 2}
\]

with:
- \( M_L(x) \) the moment at the crack location
- \( W_{steel} \) resistance moment steel deck plate for a unit thickness of 1 mm
- \( f_{dyn} \) dynamic factor with value 1.1 (Vrouwenvelder, 2000-a, 2000-b)
- \( f_{stat} \) a factor to fit the results of a simple mechanical beam model to the calculated stresses with 3D FE-models. For this crack type \( f_{stat} = 1.1 \)
- \( r(t) \) reduction factor on the stress for the stress reduction in the steel deck plate due to asphaltic surfacings
- \( \gamma_{\Delta \sigma 1} \) safety factor for the weight of the wheel loads. \( \gamma_{\Delta \sigma 1} = 1.0 \) for a reliability level of \( \beta = 0.0 \), \( \gamma_{\Delta \sigma 1} = 1.15 \) for \( \beta = 2.0 \) and \( \gamma_{\Delta \sigma 1} = 1.29 \) for \( \beta = 3.6 \).
- \( \gamma_{\Delta \sigma 2} \) safety factor for stress calculation. \( \gamma_{\Delta \sigma 2} = 1.0 \) for a reliability level of \( \beta = 0.0 \), \( \gamma_{\Delta \sigma 2} = 1.0576 \) for \( \beta = 2.0 \) and \( \gamma_{\Delta \sigma 2} = 1.1061 \) for \( \beta = 3.6 \).

Figure 5 gives the mechanical model for the stress calculation. The moment at the crack location, location \( L \), is described with:

\[
M_L(x) = \frac{qc}{24l} \left( 24 \frac{d^3}{l} - 6 \frac{bc^2}{l} + 3 \frac{c^3}{l} + 4c^2 - 24d^2 \right)
\]

with:

\[
a = \begin{cases} 
0 & \text{for } x \leq w \\
x - w & \text{for } w < x < l + w \\
l & \text{for } l < x 
\end{cases} \]

\[
b = \begin{cases} 
x & \text{for } x < l \\
l & \text{for } x \geq l 
\end{cases} \]

\[
c = \begin{cases} 
b & \text{for } x \leq w \\
w & \text{for } w < x \leq l \\
l - a & \text{for } l < x 
\end{cases} \]
\[ d = l - \frac{a}{2} - \frac{b}{2} \]

\[ q = \frac{Q}{w_t \cdot l_t} \]

with:
- \( q \) the uniform distributed wheel load
- \( w_t \) the width of the tyre
- \( l_t \) the length of the tyre

With these formulas the moment at the crack location can be calculated for each x-coordinate. In this way it is possible to calculate influence lines for all possible tyre types. This has been done for the three current wheel types.

**Stress reduction due to asphaltic surfacings.** In the Netherlands the movable bridges have a thin epoxy resin of approximately 8 mm. Fixed bridges have a thick wearing course of approximately 50 mm mastic asphalt. This asphalt layer on fixed bridges reduces the stress cycles in the deck plate. Modelling this effect is necessary to obtain accurate lifetime calculations. The mechanical modeling is difficult because the behaviour of the bituminous asphalt layer is strongly temperature dependent.

Because the asphalt models are necessary for lifetime calculations of the steel deck plate our interest is in the first place the reducing effect on the stress in the steel deck plate. Therefore strains, stresses and displacements of the asphalt surfacing are less important. The modelling of the stress reducing effect of the asphalt layer is split up into three parts.

- What are the temperatures of the asphalt layer on steel bridge decks as function of the time, sun and air temperature?
- What is the stiffness (E-modulus) of the asphalt material as function of the asphalt temperatures and loading frequencies?
- What are the stress reducing factors, based on calculations of the clamped beam model, with and without asphalt surfacing?

The assumption is made that the temperature of the asphalt is a function of the air temperature, the time and the sunshine. From measurements of the KNMI, the royal Dutch meteorological institute, the average daily minimum, maximum and average air temperature and also the average hours of sun and daylight are known for every month (www.knmi.nl), see table 2.

<table>
<thead>
<tr>
<th>Month</th>
<th>Min T (°C)</th>
<th>Max T (°C)</th>
<th>Avg T (°C)</th>
<th>Avg sun (hrs)</th>
<th>Avg day (hrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>0.1</td>
<td>5.1</td>
<td>2.8</td>
<td>1.44</td>
<td>8</td>
</tr>
<tr>
<td>February</td>
<td>-0.1</td>
<td>5.8</td>
<td>2.9</td>
<td>2.47</td>
<td>9.5</td>
</tr>
</tbody>
</table>

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Based on this measured temperatures functions have been derived which describe the air temperature.

\[ T_{air}(t) = A(t) + B(t) \cdot C(t) \]

Part A is a sinus function with a period of one year and describes the average temperature over the year. Part C is a sinus function with a period of 24 hours and describes the temperature signal over one day. The amplitude of part C is the difference between the average daily minimum and average daily maximum. This amplitude is described by part B.

\[
A(t) = 9.68 + 7.32 \cdot \sin \left( t \cdot \left( \frac{2\pi}{\text{hrs year}} \right) - 0.62\pi \right)
\]

\[
B(t) = 7.55 + 2.2 \cdot \sin \left( t \cdot \left( \frac{2\pi}{\text{hrs year}} \right) - 0.48\pi \right) - 0.58 \cdot \sin \left( t \cdot \left( \frac{5\pi}{\text{hrs year}} \right) \right)
\]

\[
C(t) = \sin \left( t \cdot \left( \frac{2\pi}{\text{hrs day}} \right) - \frac{2\pi}{3} \right)
\]

From measurements on the Moerdijk Bridge both the air and the asphalt temperatures as functions of time are known. The measurements have been performed from July to September. The measurements show that there is a difference between asphalt temperature and air temperature, which is due to the sun. It is assumed that the asphalt temperature is the air temperature with an additional component. It is assumed that this component of the temperature is a function of the average hours of sun per day and the average hours daylight per day. This can be described with the formula:

\[ T_{asphalt}(t) = T_{air}(t) + g(t) \cdot H_{sun}(t) \cdot H_{day}(t) \]
with:
g(t) is a function which fits calculated asphalt temperatures to the measured asphalt temperatures.

\[ g(t) = \frac{0.5 + 0.5 \cdot \sin \left( t \cdot \left( \frac{2\pi}{\text{hrs}_{day}} \right) - \frac{5\pi}{3} \right)}{12} \]

The hours of sun and the average hours daylight can be described with sinus functions.

\[ H_{\text{sun}}(t) = 3.975 + 2.63 \cdot \sin \left( t \cdot \left( \frac{2\pi}{\text{hrs}_{year}} \right) - 0.46\pi \right) \]

\[ H_{\text{day}}(t) = 12 + 4.52 \cdot \sin \left( t \cdot \left( \frac{2\pi}{\text{hrs}_{year}} \right) - 0.45\pi \right) \]

The calculated values are averages, for cloudy days the difference between asphalt and air temperature is smaller and for sunny days the temperature of the asphalt can reach the 50 °C. However that big differences from the averages values are realistic, this modelling turns out to be appropriate with respect to the fatigue life calculation of the steel deck plate (De Jong, 2003-a).

The stiffness of asphalt materials is dependent on temperature and loading frequencies. To examine the material properties for the asphalt mixes used on orthotropic steel bridge decks 4 point bending tests have been performed (Verburg, 1996). These tests are done for two materials: mastic asphalt and ZOAB. ZOAB is an open graded asphalt mix, which is used on a few steel bridges in the Netherlands. For a wide variety of temperatures and loading frequencies the stiffness is determined. Figure 6 shows the results of these tests. The loading frequencies are depicted as velocity of the heavy vehicles. The velocity of 25 km/h is equivalent to a loading frequency of 6 Hz and a velocity of 125 km/h a frequency of 30 Hz.

Based on the 4 point bending tests a few conclusions can be drawn. Mastic asphalt is much stiffer as ZOAB and the influence of the temperature is very significant and the influence of the loading frequency is limited. Extrapolation of the results to approximately 30-40 °C shows that the asphalt looses its stiffness practically, which is in accordance with visual observations.

Based on these tests formulas for the asphalt stiffness as function of the temperature have been derived. These functions have been derived for a velocity of 85 km/h assuming that the majority of the lorries are driving with this speed. The functions do not describe the stiffness at higher temperatures. In that case a minimum value of 50 MPa for the asphalt stiffness is appropriate.
The mechanical model depicted in figure 5 is used to calculate the stress in the steel deck plate. To calculate the stress in the steel deck plate surfaced with an asphalt layer, the same mechanical model is used. The assumption is made that there is no effective composite action between asphalt and steel, due to the very low stiffness of the intermediate bituminous layer of 2 mm. The calculation of stresses is therefore based on a model of two beams, one steel and one asphalt, with the same deflection. For each stiffness of the asphalt layer the reduction factor \( r(t) \) can be calculated. This reduction factor is defined as the ratio of the stress in the steel deck plate with asphalt surfacing and the stress in the steel deck plate without asphalt surfacing. This is given by the formula:

\[
r(t) = \frac{EI_{\text{steel}}}{EI_{\text{asphalt}}} \left(1 + \frac{EI_{\text{steel}}}{EI_{\text{asphalt}}}\right)
\]

The presented formal model and the modelling of the asphalt effects are appropriate for the fatigue calculation of deck plate cracks. The formal model has been implemented in a computer program. With this program the fatigue life of the deck plate crack for several bridges in the Netherlands has been calculated. The results both for movable bridges and fixed bridges with an asphalt layer are in accordance with the results of inspections on the bridges (De Jong, 2003-a).

References


Figures

![Diagram showing maintenance philosophy]

Figure 1. Maintenance philosophy, conventional maintenance (left red circle), risk-based maintenance (right green circle)

![Diagram showing emergency reparation]

Figure 2. Emergency reparation for deck plate cracks
Figure 3. Deck plate solution

Figure 4. Details deck plate solution

Figure 5. Mechanical model stress calculation deck plate
Figure 6. Stiffness modules asphalt materials