OVERVIEW FATIGUE PHENOMENON IN ORTHOTROPIC BRIDGE DECKS IN THE NETHERLANDS

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Abstract

An orthotropic steel bridge deck is a construction type with advantages. It is a lightweight construction, which is beneficial especially in case of larger spans. However over the last decades several details showed fatigue cracks. Well-known examples of fatigue cracks in the Netherlands are those observed in the bascule bridge Van Brienenoord in Rotterdam in summer 1997. Due to the huge amount of heavy vehicle traffic at the highways and the heavy but still growing axle loads, several fatigue cracks have already been observed in bridge decks and more cracks are expected. A lot of orthotropic steel bridge decks in the Netherlands are build up in the sixties and seventies. Fatigue problems are nowadays a challenging problem for bridge engineers.

This introductory paper describes several fatigue prone details. Especially four types of cracks are discussed in this paper. These are cracks in the deck plate (1), cracks in the longitudinal weld between deck plate and longitudinal trough profile (2), cracks in the trough splice joint (3) and cracks in the connection between the trough profile and the crossbeam (4). For those four crack types the mechanical background is briefly analysed. The description of the cracks is illustrated with visual observations of the cracks in the steel structure itself and also with visual observations of the surfacing layers.

The analysis of these structural details leads to the conclusion that especially fatigue cracks in the deck plate are potentially disturbing the traffic flow. For this reason emphasis on solutions for this type has been made. The analysis also leads to conclusions that are useful for the development of improved bridge deck structures.

History

In the third decade of the 20th century, engineers in Germany and the USA were searching for an alternative to wooden and concrete decks supported by stringers and crossbeams. They considered steel decks as a promising alternative. The objectives were cost savings by a reduction of the steel mass and also achieving a reduction of
the weight which affects the support structures such as piers, abutments etc. (Leendertz, 2003)

**Battle deck.** The first generation was the “Battle deck“ concept were the deck consisted of a steel plate with a thickness of 10 to 20 mm, connected with welds to longitudinal I-beams at centre-to-centre distances of 250-850mm. In these decks the crossbeam distances varied from 4.5 to 7.5 m. On the deck plate an asphaltic wearing course was used. The deck plate has two functions: 1. Supporting the traffic loads and distribution of the loads to the stiffeners. 2. Providing additional area to the top flange of the stiffeners. Figure 1 shows a cross-section of the battle deck construction.

**Beam grid deck.** A further development was a deck structure with T-beam stiffeners upside down in longitudinal and transverse direction, see figure 2. On the deck plate an asphaltic wearing course was applied. The deck plate had three functions: 1. Supporting and distributing the traffic loads to the longitudinal and transverse stiffeners. 2. Providing a top flange for the longitudinal stiffeners. 3. Providing a flange for the transverse stiffeners. The system worked as an orthogonal an isotropic plate, briefly called orthotropic deck. Figure 2 shows a cross-section of the beam grid deck.

**Open stiffeners.** Search for a more economic use of materials and a reduction of labour needed for fabrication and assembly resulted after the 2nd world war to the orthotropic steel decks with open stiffeners and later on the orthotropic steel decks with closed stiffeners.

The commonly used open stiffeners are strips, bulb profiles and angles. They are welded in longitudinal direction to the deck plate with fillet welds. Usually these stiffeners are continuous and pass the crossbeams through cut outs, often with cope holes. The open stiffeners are applied for maximum spans of approximately 2 m. Figure 3 shows a cross-section of a deck with three types of open stiffeners.

**Closed stiffeners.** The commonly used closed stiffeners can have a V-shape, U-shape or a trapezoidal cross-section. Sometimes in the past the V-shape stiffeners have an extension, so that the stiffener cross-section takes the form of a wineglass. The stiffeners are welded to the deck plate in longitudinal direction. The closed stiffeners can be fitted between the crossbeams or can be continuous and pass over supports, that is the so-called floating deck structure, or through cut outs in the crossbeam. In many cases the cut outs have been enlarged with cope holes as a solution for fitting problems. The closed stiffeners are used for spans from approximately 3.5 – 5.0 m. Figure 4 shows continuous trapezoidal stiffeners through three types of cut outs.

**Steel bridges in the Netherlands**

In the past many orthotropic steel bridges have been build. In the Netherlands the vast majority of these bridges is build up between 1960 en 1980. The deck plate of the majority of these bridges is stiffened with a trapezoidal trough profile. Figure 5 shows the construction of this type of bridge. In the network of highways in the Netherlands
there are approximately 80 steel bridges, divided in movable and fixed bridges. There are a lot more steel bridges in the secondary road network. Fatigue however isn’t a problem for the bridges in secondary roads. Only the traffic flow of lorries on the highways is able to induce noticeable fatigue phenomena. (De Jong, 2003)

**Traffic properties.** The heavy vehicles induce the fatigue phenomena. Therefore information about the traffic properties is relevant. In general the number and size of the loads is essential. Loads can be axle loads or vehicle loads, as they are in principle responsible for fatigue phenomenon at deck plate level respectively in the main girder system. The network of highways in the Netherlands is equipped with instrumentation to measure the amount of lorries and passenger cars. The traffic flow is measured 24 hours a day for almost every highway bridge. With an additional measurement of the number and type of the axles at lorries the number of axles can be calculated. Figure 6 gives an indication of the network of measuring locations in the environment of Rotterdam.

For example: the number of heavy vehicles on the Moerdijk Bridge, in the highway A16 (Rotterdam-Antwerp) is approximately 2.5e6 per year per lane. This is already more than the indicative number of heavy vehicles per year per slow lane, which Eurocode 1 part 2, Traffic loads on bridges gives (Eurocode 1-2, 2003). Moreover this number increases each year with a percentage of approximately 4 or 5 %. This makes clear that estimation of the number of vehicles should be performed with great accuracy, especially for the fatigue verification of new designs. From measurements became clear that the average number of axles on a lorry is approximately 4. Construction details at the top of the bridge deck construction for which every single axle generates one single stress cycle suffer from approximately 1e7 stress cycles per year.

The loads of the axles have been measured the last decade at several bridges several times (Vrouwenvelder, 1998). The objectives of these measurements are: 1. Obtain an accurate statistical model for the distribution of the axle loads for fatigue calculations. For fatigue calculations with Miners rule a histogram for the axle loads is necessary. 2. Development of a statistical model for the design load of a bridge, in order to enable a probabilistic calculation of the ultimate limit states. The relevant results for the fatigue calculations are given in figure 7. For three axle types, single, double and super, the load spectra are given.

**Fatigue phenomenon in steel bridge decks**

The last decades several fatigue cracks have been detected in the deck structure of bridges with trapezoidal stiffeners. These cracks have been defined. They are divided in four categories.

1. Cracks in the deck plate
2. Cracks in the longitudinal weld between deck plate and trough web
3. Cracks in the trough splice joint
4. Cracks in the connection between trough profile and crossbeam
Cracks in the deck plate (1)

Introduction. DPS01 is the code for a fatigue crack in the deck plate at the crossing of crossbeam and trough girder. This crack is visible in figure 8. Detail B in figure 5 is also a DPS01 crack. The growth of this crack is divided in three phases. In figure 9 these three crack phases are visible. The crack initiation (phase 1) is at the root of the longitudinal fillet weld between the trough web and the deck plate at the intersection of the crossbeam and the continuous closed trapezoidal stiffeners. After the initiation phase the crack growth is in vertical direction from the bottom side to the topside of the deck plate (phase 2). After the crack is grown through the deck plate the crack growth is in horizontal/longitudinal direction (phase 3). The crack has a semi-elliptical surface. The length of the cracks at the bottom side of the deck plate is approximately four times the deck plate thickness longer as the crack length at the topside of the deck plate.

The DPS02 crack is also visible in figure 8. This crack is roughly the same as the DPS01 crack. The difference is that this crack is located in the field between the crossbeam, where the DPS01 crack is located at the intersection of trough and crossbeam. Another difference with the DPS01 crack is that crack growth in vertical direction (phase 2) and horizontal direction (phase 3) is taking place simultaneously. The consequence is that long invisible cracks are likely to grow.

Mechanical background. The mechanical background of the DPS01 crack is as follows. The crossbeam web supports only between two troughs the deck plate. Between the webs of one single trough the crossbeam web does not support the deck plate. A local wheel load at the deck plate straight above a trough causes a deflection of the deck plate. The deck plate between two troughs and also the trough webs are welded to the crossbeam web. This causes in fact a clamping moment at the deck plate with high stress concentration factors in the deck plate, when it is loaded with a heavy vehicle wheel. Due to this construction and clamping moments high local bending stresses will arise, thus causing fatigue cracks in the deck plate.

The DPS01 and DPS02 cracks may be a threat for the traffic safety of the deck structure. This threat arises only if cracks are grown very large, e.g. when the cracks are trough the thickness of the deck plate and have reached a length of approximately 50 cm in longitudinal direction. In case of cracks with such dimensions the risk of an indentation or a hole in the deck plate due to a heavy local wheel load exists. Because the deck plate is an integral part of the crossbeam, where it acts as the top flange, there is also a risk that the load bearing capacity of the crossbeam is not sufficient for a heavy truck.

The deck plate cracks appear both in decks of fixed bridges with a thick asphaltic surfacing as well as in decks of movable bridges that are usually surfaced with a thin epoxy layer. The presence of a thick asphaltic layer however reduces the stress cycles hence resulting in a longer lifespan. Thin epoxy layers do not reduce the stresses in the deck plate, consequently having no effect on the lifespan of the deck structure.
**Visual observation.** Visual observation is one of the important methods to detect fatigue cracks in the deck plate. Three major advantages of visual observations are the low costs, the speed of execution and the unneccessity of removing surfacing layers. Due to the fact that the deck plate cracks initiate at the inner side of the closed trough profile, at the root of the longitudinal weld, cracks are only visible in phase 3 of the growth process, when the cracks are grown through the deck plate and have reached a length at the top side of the deck plate. This is the major disadvantage of visual observation. Detecting fatigue cracks in phases 1 and 2 is possible with NDT technologies.

A distinction is made in visual observations between cracks in movable bridge decks with a thin epoxy surfacing layer and on the other hand cracks in bridge decks with a thick asphaltic surfacing layer. Figure 10 is a photo of the visual observation of the cracks in the Van Brienenoord Bridge in summer 1997. (Kolstein, 1998). Damage of the epoxy surfacing in the wheel tracks of the heavy vehicle lane is the first indication of deck plate cracks.

In figure 10 this damage is visible. Clearly visible are the locations of the trough webs under the deck plate at a centre-to-centre distance of 300 mm. Figure 11 shows a detail of a visual observation. Clearly visible is the crack in the steel deck plate. Deformation of the steel deck plate at the crack location is responsible for this visibility. In an earlier stage of the crack growth process the steel plate at both sides of the cracks is not deformed and the crack is more difficult to see, because it is real hair crack with small dimensions.

Fatigue deck plate cracks in bridge decks with a thick asphaltic surfacing are more difficult to detect. Visual observations in the asphalt layer are difficult to interpret. The age of the surfacing layer, the thickness and stiffness of the surfacing layer, the thickness of the steel deck plate, and the fatigue cracks in the steel deck plate cracks are some factors, which govern the deterioration of the asphalt layer. A lot of experience and good workmanship is needed to determine the causes of a deteriorating asphalt structure. It is difficult to judge whether visual observed damage in the asphalt layer is due to fatigue deck plate cracks or to other causes. Some photos of degradation of the asphalt structures in the Netherlands are presented.

Almost every fixed bridge in the Netherlands has a deck thickness of 10 mm, and the centre-to-centre distance between the trough webs is usually 300 mm. Furthermore almost every fixed bridge is supplied with a layer of 50 mm mastic asphalt. In case the construction differs from this standard bridge construction, the interpretation of visual observations should be done with carefullness. Once again is stated that with visual observations from the surfacing layer only deck plate cracks that are already grown through the deck plate (phase 3) are visible.

Figure 12 shows a photo of the asphaltic surfacing of the Caland Bridge in the A15 in the harbour area of Rotterdam. This photo shows severe damage in the asphaltic surfacing. However there were no fatigue cracks grown through the deck plate at this locations. The damage of this surfacing layer is for the main part normal aging of the asphalt. At some locations the asphalt is broken into pieces. At these locations fatigue cracks grown through the deck plate are possible. On the other hand the total degradation of the asphalt layer points towards aging of the asphalt structure.
Because deck plate cracks had already been observed several times before on the Caland Bridge the bridge owner wants to be on the safe side. Therefore the asphalt layer has been removed at the most damaged locations. A subsequent NDT inspection showed no fatigue cracks grown through the deck plate. After this NDT inspection a new asphalt layer has been applied.

Figure 12 also shows some very long longitudinal cracks in the asphalt layer. The distance between this cracks is approximately 300 mm, which is the centre-to-centre distance between the trough webs. This crack mode of the asphalt surfacing is in general no indication for fatigue cracks in the steel deck. The deck plate thickness of 10 mm is relatively small. Wheel loads of heavy vehicles impose a deflection of the steel deck plate and the surfacing layer. This deflection causes a strain in the surfacing layer. The maximum strain occurs at the locations of the trough webs. In some circumstances these strains exceed the ultimate strain of the material. At that moment a crack in the asphalt layer is generated. These circumstances are mainly cold temperatures, because the asphalt material shows a rather brittle material behaviour at cold temperatures. These longitudinal cracks in the asphalt layer have been observed at several bridges. Also on relative new bridges with a moderate traffic flow, and also in relative new surfacing layers.

Figures 13 and 14 display a kind of spider’s web in the asphalt layer. This observation in combination with a relative good condition of the rest of the asphalt layer is an indication of a deck plate crack that is grown through the deck plate. In figure 14 are also the longitudinal cracks at the location of the trough webs visible. These kinds of spider webs are observed several times on several bridges. Usually the asphalt layer is removed on that location and a NDT inspection is executed. In some cases cracks grown through the deck plate have been found, in some case no cracks have been found and also in some cases cracks have been found that are not grown through the deck plate.

In Bridge Hagestein in the highway A27 (Utrecht-Antwerp) a kind of spider’s web was visible. After the removal of the asphalt structure and blasting the deck plate the crack in figure 15 was visible. The length of this crack was 65 cm. A few repair techniques are available for this crack type. The simplest solution is a weld at the crack location. Another solution is welding a plate on top of the deck plate and applying a new asphalt layer. The disadvantage of both solutions is the bad fatigue behaviour. The advantage however is the speed of execution. Both solutions are therefore temporarily. These repair techniques and other techniques are described in (Boersma, 2003)

**Detected cracks.** In 1997 deck plate cracks have been discovered in the bascule bridge Van Brieneenoord. This was the first bridge in the Netherlands where this kind of cracks occurred. After 1997 in several bridges deck plate cracks have been detected. Table 1 summarizes the bridges in the Netherlands with fatigue cracks in the deck plate. On some bridges one single crack is detected, on some bridges a few cracks are detected and on some bridges a lot of cracks are detected.

Table 1. Detected deck plate cracks
Cracks in the longitudinal weld between deck plate and trough wall (2)

Introduction. TRDPL01 is the code for a fatigue crack in the longitudinal weld between the trough and the deck plate. This crack is shown in figure 16. The crack initiation is at the root of the longitudinal fillet weld between the trough web and the deck plate. The initiation point can be at every location in longitudinal direction, except the intersection of the crossbeam and the continuous closed trapezoidal stiffeners. After the initiation phase the crack growth is in thickness direction of the weld. After the crack is grown through the weld the crack growth is in horizontal/longitudinal direction along the weld. The TRDPL01 cracks in the longitudinal weld do not form a threat for the traffic safety and the integrity of the deck structure, because the deck plate is still functioning and the possibilities of redistribution of loads.

Mechanical background. The mechanical background of the TRPDL01 is as follows. The webs of the troughs support the deck plate. The deck plate is in fact a girder with multiple supports at distances of 300 mm normally. Local wheel loads cause a deflection of the deck plate between the trough webs at the location of the wheel load. The adjacent fields of the deck plate deflect in upwards direction. Due to this deflections the trough webs are bended.

Due to the limited bending stiffness of the trough profiles, the trough webs are elastic and not rigid supports for the deck plate. The spring stiffness of the supports depends on the distance to the crossbeam. This elastic behaviour results in a maximum deflection at the location of the wheel load. Bending moments in deck plate and trough webs will arise due to these deformations. The bending moment in the trough web results in stresses in the longitudinal weld between deck plate and trough web. This stress in the trough web is the main cause of fatigue cracks in the longitudinal weld.

This crack type appears both in decks of fixed bridges and movable bridges. The thick asphaltic layer reduces the stress cycles in a fixed bridge compared to a movable bridge, hence resulting in a longer lifespan for fixed bridges. Thin epoxy layers do not have this effect.

The crack growth rate is strongly dependent on the quality of the longitudinal weld. From fatigue tests it is clear that the gap between trough profile and the under

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Build</th>
<th>First visual fatigue cracks</th>
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<tbody>
<tr>
<td>Ketel Bridge, movable</td>
<td>1968</td>
<td>1998</td>
</tr>
<tr>
<td>Scharsterrijn, movable</td>
<td>1972</td>
<td>2002</td>
</tr>
<tr>
<td>Van Brieneoord, movable</td>
<td>1990</td>
<td>1997</td>
</tr>
<tr>
<td>Caland Bridge, movable part</td>
<td>1969</td>
<td>1998</td>
</tr>
<tr>
<td>Caland Bridge, fixed part</td>
<td>1969</td>
<td>2002</td>
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<tr>
<td>Bridge Hagestein, fixed</td>
<td>1980</td>
<td>2002</td>
</tr>
<tr>
<td>Galecopper Bridge, fixed</td>
<td>1971</td>
<td>2002</td>
</tr>
<tr>
<td>Juliana Bridge, movable</td>
<td>1966</td>
<td>2001</td>
</tr>
<tr>
<td>Bridge Scharber, fixed</td>
<td>1973</td>
<td>2003</td>
</tr>
<tr>
<td>Moerdijk Bridge, fixed</td>
<td>1976</td>
<td>2001</td>
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side of the deck plate is one of the factors determining the fatigue behaviour of the weld (Kolstein, 1996). Another factor is the size of the weld. A full penetration weld shows a much better fatigue behaviour than a fillet weld of 3 mm.

**Visual observations.** Visual observation is the most important method to detect cracks in the longitudinal weld. The difference with the visual inspection of deck plate cracks is that the inspection must be executed from the underside of the bridge deck structure. The advantages are the low costs and the speed of execution. This cracks type originates also from the inner side of the trough. Therefore the crack is only visible when it is grown through the weld and has reached a certain length. Due to the fact that this crack type does not threaten the traffic safety, inspections are less necessary. For the easiness of visual detection of this crack type it is very useful to paint the underside of the bridge deck white in the region where the wheel loads of the heavy vehicles are driving. Figure 17 shows a crack in the weld between deck plate and trough web. In contrast with figure 16 this crack is located near the crossbeam.

**Detected cracks.** The TRDPL01 crack type is already known for a long time. The Moerdijk Bridge in the A16 motorway from Rotterdam to Antwerp is a well-known example of a bridge with this crack type. In 1999 the longitudinal welds at the locations of wheel loads are replaced with a full penetration weld. This crack type has already been observed in the Galecopper Bridges, the Kreekrak Bridges, Bridge Hagestein, Scharberg Bridge and several other bridges.

**Cracks in the trough splice joint (3)**

**Introduction.** TRPS01 is the code for a fatigue crack in the trough splice joint. This splice joint is made in various ways. The most common detail is given in figure 18. The trough splice detail under transverse deck welds consists of a small trough length fitted on site between the troughs already welded in the factory. Backing strips allow for welding.

The crack usually initiates from the root of one of the welds at the bottom side of the trough. In several bridges cracks have been observed in the parent material originating from the weld connecting the backing strip to the trough and cracks originating from the weld root. Figure 18 shows the trough in elevation together with two cross-sections of the trough web. The cracks in the trough and in the connection are indicated. Cracks in the trough splice joint have been detected in several bridges in the past. This crack type does not threaten the traffic safety, because of the possibilities of redistributing the loads, provided that crack dimensions are limited. A considerable part of the observed cracks in the trough splice joint are due to a bad weld quality.

**Mechanical background.** The mechanical background of the TRPS01 is as follows. In the trough profile significant bending moments arise due to the local wheel loads. This results in considerable longitudinal bending stresses in the trough profile. The location of the splice joints of the trough profiles is normally chosen in the region
with the lowest bending moment ranges. However there are considerable stress ranges. These stresses in combination with the presence of a backing strip, lack of penetration and misalignment result in high stress concentrations. Fatigue cracks initiate at the locations of these stress concentrations. The crack growth rate is strongly dependent on the type of the weld between the two troughs (Kolstein, 1996) and the quality of the welds.

Analyses showed that the welded detail of the backing strip to the trough had low fatigue strength. In many cases the splice had been fitted with high external forces, which led to a weld with a very small basis. The weld root of the realised weld was of inadequate quality, because of insufficient shrinkage capacity.

**Visual observation.** Visual observation is the most important method to detect cracks in the stiffener splice joint. The inspection must be executed from the under side of the bridge deck structure. Figure 19 shows a crack in a stiffener splice joint. This crack is already grown relatively large. The crack is clearly visible, due to the movements of the crack and the trough under the traffic flow. In an earlier stage the crack is more difficult to see. For the easiness of visual detection of this crack type it is very useful to paint the underside of the bridge deck white in the region where the wheel loads of the heavy vehicles are driving.

**Detected cracks.** In a lot of bridges in the Netherlands some cracks in the stiffener splice joints have already been observed. During the lifetime of the bridge structure sometimes a fatigue crack in the stiffener splice joint occurs. This indicates that the crack growth is sensitive for the quality of the weld.

**Cracks in the connection between trough and crossbeam (4)**

**Introduction.** The connection between the trough stiffeners and the crossbeam is fabricated in several ways. The troughs pass through the crossbeams or are fitted between the crossbeams, there are additional cope holes for easiness of fit or there is a close fit. These are the two main points. Due to this there are several types of connections trough-crossbeam web, and subsequently several type of fatigue cracks in this connection. In the figures 20, 21, 22, 23 and 24 the different types are indicated. Also the crack types are defined in the connection between trough stiffener and crossbeam. For an extensive description of the trough to crossbeam connections, see (Leendertz, 1995, 2003).

**Mechanical background.** In the first designs of decks with closed stiffeners, the trough was welded to the crossbeam web as shown in figure 20. At first the stiffeners and crossbeams were welded together with a fillet weld. Depending on the realised geometry of the weld cracks were observed at the trough side (C.1) or the crossbeam side (C.2) of the weld both in bridges and tests specimens. Analyses and research showed that the fillet welds had low fatigue strength. Further the increasing traffic caused large numbers of stress intervals in the bottom of the trough. In many cases serious damage has been found. The improved version of the detail has full penetration welds as shown in figure 20, which increases the fatigue strength considerably. Specific
attention must be paid to the rotation effect of the trough. When the crossbeam web becomes relatively rigid, this may become a source of fatigue cracks too.

Figure 21 shows an improved connection. In this connection a continuous trough passes through a cut out in the crossbeam with a close fit and is welded around. Although the details show a good behaviour with respect to fatigue generated by bending moments in the trough and the crossbeam “Vierendeel behaviour”, the detail is not commonly used as the tolerances on trough dimensions and cut out are small. The details showed cracks (C.3 and C.4) in bridge structures and in fatigue test specimens. Special attention must be paid to the rotation effect of the stiffener, which in case of a rigid crossbeam web may cause high stresses. Tests showed that V-shaped stiffeners were more susceptible to fatigue than trapezoidal stiffeners.

As a solution for the fitting problems with the detail C.3, C.4 the cut out in the crossbeam web is enlarged with a cope hole. The traditional cope hole used to have an oval or trapezoidal shape, see figure 22. Research by Haibach and Plasil (Haibach, 1983) resulted in a cope hole with a large radius under the trough and smaller radii near the connection to the trough see figure 23. In some bridges the original cope hole showed cracks in the locations C.5, C.6 and C.7. In bridges with the improved shaped cope holes no cracks have been found until now in bridges in the Netherlands. Analyses show that the improved cope holes result in lower stresses in the cope hole. However the stresses at C.5 and C.8 remain at the same level, which also applies for the stresses in C.6 and C.9.

The locations C.5, C.6, C.8 and C.9 are subject of recent and current research, which is not yet finished (Leendertz, 1995). Based on experience so far it could be concluded that the cope hole proposed by Haibach and Plasil is for many cases an improvement for the crossbeam web. Further much attention shall be paid to the finish of the weld in order to avoid additional stress raisers.

The floating deck structure as shown in figure 24 has been developed to reduce the effects of fitting and for easy fabrication and assembly. Until now no cracks have been observed in the details C.11, C.12 and C.13. Under traffic loads the deck moves in transverse direction, which generates bending moments in the trough web and the deck plate. These bending moments cause stress intervals in the trough to deck connection that are to be added to the effects from the wheel loads. For this reason cracks in the longitudinal weld grow relatively fast in floating deck structures.

For an extensive description of the mechanical behaviour of the trough to crossbeam connections, see (Leendertz, 1995, 2003).

For the crack types defined in the trough to crossbeam connection can also be said that these cracks do not threaten the traffic safety, because of the possibilities of redistributing the loads, provided that the crack dimensions are limited. A considerable part of the observed cracks in the trough to crossbeam connections are due to a bad weld quality.

**Visual observations.** In figures 25, 26 and 27 are the detected cracks in a connection trough – crossbeam visible. The trough profile is passing through the cut out in the crossbeam web. The crack is already grown from the bottom side of the trough profile to the deck plate. Figure 26 shows at the left side more in detail the bottom of the trough profile and at the right side the connection with the deck plate. Both details
are from figure 25. It is visible in the pictures that this is a crack in the web of the crossbeam. This is crack type C.4 from figure 21. Clearly visible in the detail of the crack at the deck plate is that the crack grows further in the longitudinal weld between the deck plate and the trough profile. This crack is grown relatively large, which is visible in figure 17. Figure 17 is the same connection trough-crossbeam. For the easiness of visual detection of this crack type it is very useful to paint the underside of the bridge deck white in the region where the wheel loads of the heavy vehicles are driving.

**Detected cracks.** In a lot of bridges in the Netherlands various cracks in the connection between the trough profile and the cross beam have already been observed. During the lifetime of the bridge structure sometimes a fatigue crack in the connection occurs. This indicates that the crack growth is sensitive for the quality of the weld and the fit of the parts constructing the connection.

**Conclusions**

Although the advantages of steel bridge decks, fatigue cracks are a significant problem for orthotropic steel bridge decks. Various types of cracks are detected in bridges in the Netherlands. Especially cracks in the steel deck plate are threatening the traffic safety if they reach a certain length.

Considering the amount of observed fatigue cracks in the deck plate of existing bridge structures two major questions should be answered: 1. How to solve this problem for existing bridges? 2. What are necessary changes in the designs for new bridge decks?

**References**


ENV 1991-2:2003, Eurocode 1; Basis of design and actions on structures; Part 2: Traffic loads on bridges


Figures

Figure 1. Cross-section of the battle deck. 1) wearing course, 2) deck plate, 3) longitudinal weld, 4) INP-beam.

Figure 2. Cross-section of the beam grid deck: 1) wearing course, 2) deck plate, 3) T-sections in longitudinal direction, 4) T-sections in transverse direction

Figure 3. Cross-section of a deck with three types of open stiffeners: 1) wearing course, 2a) strip stiffener, 2b) bulb stiffener, 2c) angle stiffener, 3) crossbeam web.
Figure 4. Cross-section of a deck with closed trapezoidal stiffeners through three types of cut outs. From left to right: a close fit, an oval cope hole and a “Haibach” cope hole. 1) wearing course, 2) deck plate, 3) crossbeam web.

Figure 5. Isometric view of an orthotropic steel bridge deck with 2 fatigue cracks
Figure 6. Measure locations traffic flow environment Rotterdam

Figure 7. Measured load spectra axles single, double, super (replace)
fatigue crack DPS01

trough

crossbeam

trough wall

department

fatigue crack DPS02

Figure 8. Deck plate crack

Figure 9. Crack phases deck plate crack
Figure 10. Visual observation deck plate cracking movable bridge

Figure 11. Detail visual observation deck plate cracking movable bridge
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Figure 13. Visual observations indicating deck plate crack
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Figure 17. Cracks in the longitudinal weld trough – deck plate
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Figure 19. Crack in the stiffener splice joint
Figure 20. Fatigue cracks trough - crossbeam connection, trough fitted between crossbeam

Figure 21. Fatigue cracks trough - crossbeam connection, continuous trough passing through the crossbeam

Figure 22. Fatigue cracks trough - crossbeam connection, continuous troughs with traditional cope holes

Figure 23. Fatigue cracks trough - crossbeam connection, continuous troughs with Haibach cope holes
Figure 24. Fatigue cracks trough - crossbeam connection, floating deck structure

Figure 25. Visual observed fatigue crack trough – cross beam connection

Figure 26. Details visual observed fatigue crack trough – cross beam connection
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