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Fatigue assessment for deck plates in orthotropic bridge decks

Since the 1960s, orthotropic deck plates of highway bridges have been built with large cold-formed trapezoidal stiffeners supporting a deck plate with a thickness of approx. 12 mm. The maximum cross-beam spacing is approx. 4 m. A number of these bridge decks in The Netherlands suffer from fatigue cracks in the deck plate. First cracks have been observed after about 30 years in service. In one particular movable bridge, the cracks were found after only seven years. In many other countries, this type of crack has not yet been observed. This article provides a fatigue assessment procedure for deck plates. The procedure is calibrated with the conditions and observations in The Netherlands. It gives a fatigue life prediction and takes account of inspection results quantitatively. Although aspects such as the type and thickness of the surface finishes and the traffic load may vary between countries, the principles of the assessment procedure in this article are generally applicable and can be used to identify reasons for differences in fatigue life and to develop strategies for increasing the life.

1 Introduction

Since the 1950s, steel highway bridges have been constructed with a steel deck structure called an orthotropic deck. These decks were constructed with a deck plate supported by stiffeners of various shapes and by cross-beams and main girders [1]. The deck plate acts as the top flange of the deck girders. Since about 1965, a new generation of orthotropic decks has been used, with cold-formed trapezoidal stringers, so-called troughs. This permitted cross-beam spacings of up to approx. 4 m, see Fig. 1. Although this general concept was applied in many countries, there were differences between countries regarding aspects such as the thickness and composition of the surface finishes. The number of heavy goods vehicles (HGVs) travelling across bridges also varied between highways and between countries.

In The Netherlands, fatigue cracks have been observed in the deck plate at

the intersection with the trough stringer and the cross-beam web in a number of the bridges that were constructed between 1960 and 1990. Since then, the traffic load has increased in terms of weight and number of vehicles. In addition, the wheel configuration has changed due to the use of single, heavily loaded wheels ("super singles") instead of twin-wheel systems. This was not anticipated in the design.

This article describes a numerical procedure developed in several studies in The Netherlands to verify the fatigue life of deck plates regarding the

initiation and growth of cracks in this deck plate detail. Main information sources are *De Jong* [7], *Kolstein* [6] and *Dijkstra* [3]. The procedure has proved to be reliable in view of observations during inspections and it allows the deck plate thickness to be determined for various input parameters such as fatigue load, thickness and composition of surface finishes and the inspection schemes as practised in The Netherlands.

Although details of the procedure may require modification to account for variations between countries in terms of geometry, surface finishes or loading, the principles of the procedure are generic and applicable to steel decks in other countries. The procedure could help to identify reasons for differences in fatigue lives and to develop strategies for increasing the life at reasonable cost. In this respect, the procedure may be relevant for countries where this type of crack has not yet been observed, but where – due to further increases in traffic loads – fatigue cracks as described in this article could develop in the future.
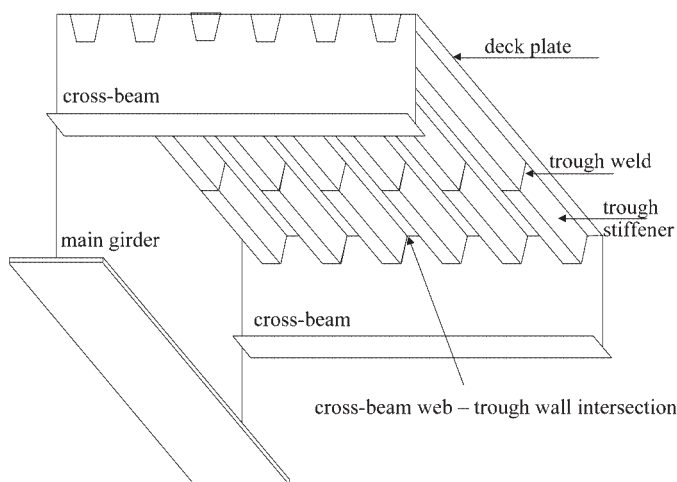
Fig. 1. *General view of orthotropic deck structure*

2 Description of the problem
2.1 Types of fatigue cracks

The heavy fatigue load and the complex stress pattern (with high stress concentrations) in orthotropic decks have resulted in fatigue cracks in several bridges. Various types of cracks have been observed in the orthotropic deck. An especially severe type of crack – which could eventually result in traffic accidents – is the crack in the deck plate that grows from the weld root between the trough and the deck plate at the junction with the cross-beam, see Fig. 2 and [5]. This type of fatigue crack has been observed in various bridges in The Netherlands. Table 1 gives an overview of deck plate cracks observed in a number of Dutch highway bridges. In particular, the bascule bridge of the 2nd Van Brienenoord Bridge has shown a short fatigue life due to the extreme number of HGVs around the port area of Rotterdam.

2.2 Case description: Van Brienenoord Bridge

The 2nd Van Brienenoord Bridge was completed in 1990. In the summer of 1997, deterioration was observed in the movable part of the bridge. During an inspection, a number of fatigue cracks were found in the top side of the deck plate by visual inspection at the most heavily loaded lanes in the deck of the bascule of this bridge (Fig. 3).

The deck of the bridge consists of a deck plate of thickness t = 12 mm and trough stiffeners with a wall thickness of 6 mm spaced at a distance of 600 mm. The troughs are 300 mm wide at the top and 105 mm at the bottom, and they are 325 mm deep. Thus, the deck is supported every 300 mm by a trough wall. A thin surface finish was applied to the top of the deck plate, consisting of epoxy with a minimum thickness of 6 mm. These dimensions are typical for movable bridges in The Netherlands for the period between

Fig. 3. Typical through-crack in deck plate (surfacing = 6 mm epoxy)

1960 and 1990. The Van Brienenoord Bridge has continuous troughs at the cross-beam web location. There are no cross-beam web plates in the troughs.

Visual inspection revealed the through-cracks in the deck plate at junction of the cross-beam web plate and trough wall. A partial penetration weld was used between the trough wall and the deck plate. The crack starts at the root of the trough-to-deck-plate weld (see Fig. 2 left, indicated by i). The crack grows as a semi-elliptical crack into the deck plate of the upper deck surface (see Fig. 2 right) and along the root of the weld at the lower surface of the deck plate. As the cracks start on the inside of the connection, visual inspection cannot detect small crack initiations. With advanced inspection techniques, cracks were found at every junction of cross-beam web plate and trough wall in the most heavily loaded lanes.

The absence of the cross-girder web plate in the trough is responsible for a high stress concentration at the point of initiation. As a result, large stress ranges occur due to a wheel load travelling in the centre of a trough. In the case of linear elastic material, the stress ranges are entirely compressive. The welding process has caused additional residual compressive stresses at this location. Therefore, fatigue cracks
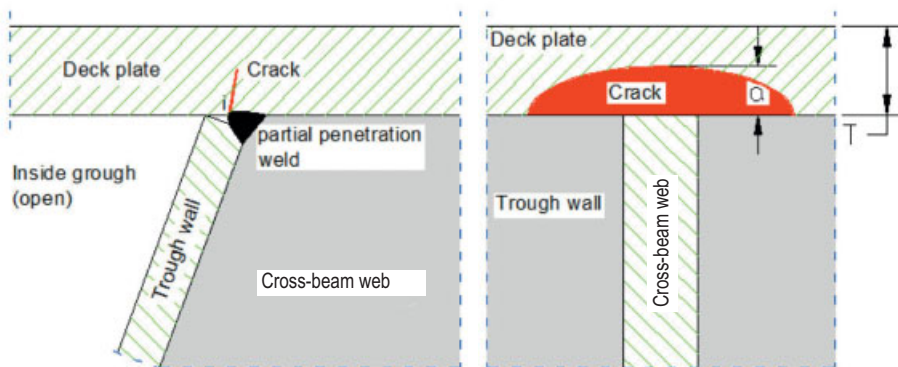

Fig. 2. Location of crack in deck plate (t = deck plate thickness, a = crack depth)

Table 1. Overview of deck plate cracks observed in a number of Dutch highway bridges (source: De Jong [7])

Bridge	Type	Year of completion	First visible crack detected in…	Age [years]
Ketelbrug	movable	1968	1998	30
Scharsterrijn	movable	1972	2002	30
2nd Van Brienenoordbrug	movable	1990	1997	7
Calandbrug	movable	1969	1998	29
Brug Zijkanaal C	movable	1969	2003	34
Julianabrug	movable	1966	2001	35
Calandbrug	fixed	1969	2002	33
Brug Hagestein	fixed	1980	2002	22
Galecopperbrug	fixed	1971	2002	31
Moerdijkbrug	fixed	1976	2001	25
Boogbrug Beek	fixed	1968	2004	36

were not expected at this location in the design. However, due to the high stress concentration factor (SCF), the resulting strain may exceed the strain value accompanying yielding. Consequently, the next stress cycles will have a tensile component responsible for fatigue crack initiation and crack growth.

3 Fatigue prediction model

A fatigue prediction model has been developed in order to investigate the cause of the problem further and determine the required dimensions for orthotropic bridge decks in new designs. The model comprises four aspects:
1) A stress analysis to determine the stress at the initiation point.
2) A load model that represents the fatigue loading on Dutch highways.
3) A classification of the detail.
4) A fatigue life analysis to determine the life from installation to cracking.

3.1 Stress analysis

Various finite element (FE) models have been developed to determine the hot-spot stress range in the deck plate at the location of the crack initiation. The models indicate that the highest hot-spot stress range occurs at the weld toe – i.e. location i in Fig. 2 – for a wheel load centred over a trough at the junction with a cross-beam. Further analyses have indicated that the hot-spot stress can be accurately approximated with a simple analytical model. The prediction model has been developed for deck plates with dimensions according to Table 2. These dimensions are applied to the bridges mentioned in Table 1.

The basis of the analytical model is a 2D beam clamped at the location of the trough walls. These clamps are
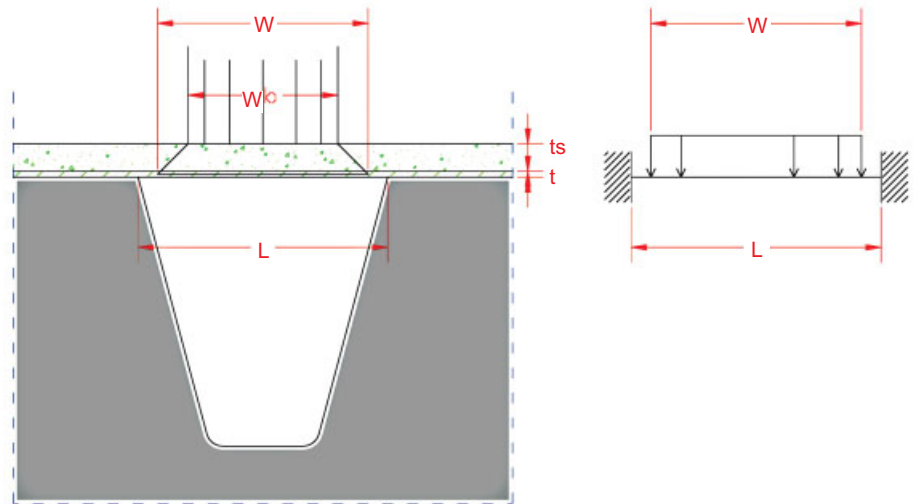

Fig. 4. Basis of the analytical model for determining the stresses in the deck plate at the crack initiation point i

due to the presence of the cross-beam web. The stress is to be determined at the support of this 2D beam. Fig. 4 gives an example of a tyre load centred over the trough. In the same way, it is possible to predict the stress for cases where the tyre load is not centred over the trough. The contribution of the surfacing finish to the model is two-fold:
1. The load is spread due to the surfacing finish.
2. The surfacing finish adds bending stiffness.

Ad 1. It is assumed that the load spreads at 45° both transversely and longitudinally from the tyre contact area. Thus, the distributed load q acting on a unit depth of the 2D beam is:

$$q = \frac{F_a}{n_a} \cdot \frac{l_b}{(w_b + 2t_s + t) \cdot (l_b + 2t_s + t)}$$

(1)

where:
F_a axle load
n_a number of wheels per axle
l_b length of tyre contact area
w_b width of tyre contact area
t_s thickness of surface layer
t thickness of deck plate

In the case of twin tyres in combination with a thick surface layer, care should be taken to count the overlapping part of the loaded area only once. Eq. (1) needs to be modified for such a case.

Detailed FE calculations have indicated that the stress at the initiation point as determined with this analytical model deviates slightly from that of the FE models. An SCF is introduced to account for this difference. The SCF is $(1.3 - 0.0094 \cdot t)$ for a thin epoxy surfacing and 1.4 for an asphalt surfacing 50 mm thick.

Ad 2. An epoxy surfacing usually has such a low stiffness and thickness that its contribution to the bending stiffness of the 2D beam is ignored. The modulus of elasticity of the asphalt layer E_{asp} depends on the type of asphalt and its temperature T_{asp}. The load duration has a small effect on the modulus of elasticity and is neglected here. Tests by *Verburg* and *Van Gogh* [9] have indicated that the modulus of elasticity of asphalt concrete applied to bridge decks in The Netherlands can be approximated by:

$$E_{asp} = 17000 - 590 \cdot T_{asp}$$

(2)

with T_{asp} in [°C] and E_{asp} in [N/mm²], and a minimum value of 0 N/mm². A membrane is applied between the steel deck plate and the asphalt surface. Due to the low stiffness of this membrane, the deck plate and asphalt layer can be approximated as working fully

Table 2. Dimensions of bridge decks typically used in bridges in The Netherlands built between 1960 and 1990

	Movable bridges	Fixed bridges
Deck plate thickness t	t = 12 mm	t = 10 mm
Surfacing finish of thickness t_s	epoxy, t_s = 6 mm	asphalt, t_s = 50 mm
Span of deck plate L[1]	L = 300 mm – 6 mm	L = 300 mm – 6 mm
Cross-beam web thickness t_c	t_c = 10 mm or 12 mm	t_c = 10 mm or 12 mm
Cross-beam depth h_c	h_c = approx. 1000 mm	h_c = approx. 1000 mm

[1] Span L = centre-to-centre distance of trough walls (300 mm) – 1 x trough wall thickness (6 mm)

non-compositely. Consequently, the contribution of the asphalt surfacing to the stress at the crack initiation point is described by a multiplication factor R:

$$R\left(T_{asp}\right) = \frac{El_{steel}}{El_{asp}\left(T_{asp}\right) + El_{steel}} \qquad (3)$$

3.2 Load model

Two load models need to be considered:
1. A model representing the traffic loads on Dutch highways.
2. A model representing the asphalt temperature.

Ad 1. The number of HGVs crossing the Van Brienenoord Bridge is determined on the basis of traffic measurements. On average, 850 000 HGVs per year were counted for the most heavily loaded lane (right-hand lane) during the period 1990–1997. Weigh-in-motion (WIM) measurements on a representative highway bridge in The Netherlands (Moerdijk Bridge) have been analysed. A fatigue load model is proposed that is based on these WIM measurements (slightly modified from *Otte* [8]). The fatigue load model consists of a set of HGVs comparable with that of fatigue load model 4 (FLM4) in EN 1991-2 [12], but with modified types and percentages of HGVs, wheelbases, wheel types and distribution of transverse wheel track locations, see Table 3 and Fig. 5. The prediction of future changes in traffic characteristics has been accounted for. Fig. 6 is a comparison of the stress spectra at the crack initiation location of the Van Brienenoord Bridge with the proposed fatigue load model and with FLM4.

Ad 2. Asphalt temperatures have been measured at Moerdijk Bridge by *Huisman* [4]. Based on these measurements, a curve fit model has been developed that describes the asphalt temperature as a function of the air temperature T_{air}, the daily hours of sunshine H_{sun} and the daily hours of daylight H_{light}:

$$T_{asp}\left(t\right) = T_{air}\left(t\right) + H_{sun}\left(t\right) \cdot H_{light}\left(t\right) \times$$

$$\times \frac{0.5 + 0.5\sin\left(\dfrac{t \cdot \pi - 4\pi}{6}\right)}{12} \qquad (4)$$

Table 3. Fatigue load model based on WIM measurements for Moerdijk Bridge (after [8])

HGV type[1]	Wheel-base [m]	Axle loads [kN][1)2)]			1990–2010[3)]				post-2010			
					axle	percentage [%]			axle	percentage [%]		
		a	b	c		a	b	c		a	b	c
1	5.2	35	55	70	A*	2.0	1.4	0.6	A*	2.0	1.4	0.6
		40	70	100	B*				B*			
2	3.8	55	75	90	A*	4.0	2.8	1.2	A*	3.5	2.45	1.05
	1.3	50	80	125	B*				B*			
		40	60	100	B*				C*			
3	3.8	55	60	70	A*	11.5	8.05	3.45	A*	8.5	5.95	2.55
	6.6	55	75	110	B*				B*			
	1.3	35	55	85	C*				C*			
		35	55	85	C*				C*			
4	3.8	60	70	80	A*	28.5	19.95	8.55	A*	27.0	18.9	8.1
	5.6	50	95	125	B*				B*			
	1.3	25	60	90	C*				C*			
	1.3	25	60	90	C*				C*			
		25	60	90	C*				C*			
5	2.8	60	70	80	A*	4.0	2.8	1.2				
	1.3	40	60	90	B*							
	5.6	60	90	115	B*				not applicable			
	1.3	45	80	105	B*							
	1.3	45	80	105	B*							
		45	80	105	B*							
6	4.2	60	75	90					A*	4.0	2.8	1.2
	1.3	70	95	125					B*			
	4.2	45	70	95					C*			
	3.8	45	80	100	not applicable				C*			
	1.3	40	65	85					C*			
		40	65	85					C*			
7	2.8	60	70	80					A*	5.0	3.5	1.5
	1.3	40	60	90					B*			
	5.6	60	90	115					B*			
	1.3	45	80	105					C*			
	1.3	45	80	105	not applicable				C*			
	4.2	45	80	105					C*			
	1.3	35	60	85					C*			
		35	60	85					C*			

[1] In total 21 heavy goods vehicles (HGVs) are considered: types 1a to 7c.
[2] The axle loads have to be multiplied by a dynamic amplification factor of 1.1 and a trend factor which is 1.2 per 100 years, with 1998 as the reference year.
[3] For the period up to 1990, the same traffic distribution is considered as in the period 1990–2010, but all axle types C* should be replaced by axle types B*.
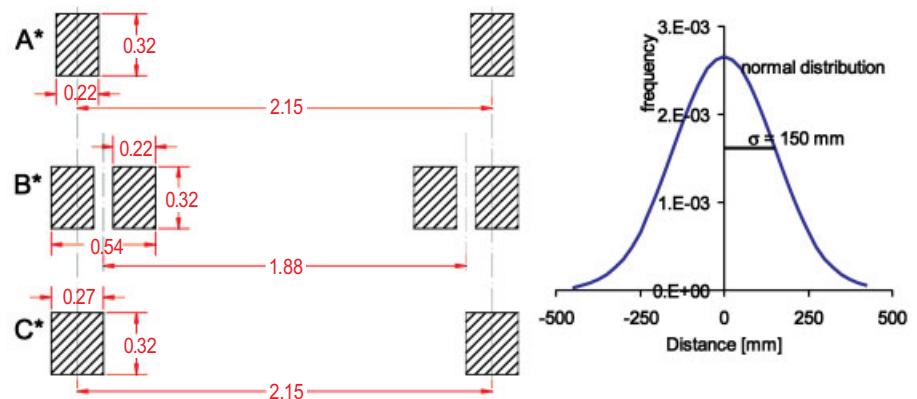

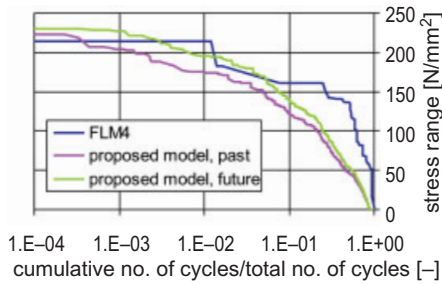*Fig. 5. Axle types and distribution of HGVs across the width*

Fig. 6. Stress spectrum for location i

Fig. 8. Fraction of numbers of HGVs as a function of asphalt temperature

with t being the time in [h]. It should be noted that the equation results in an average temperature. On cloudy days, T_{asp} is lower than predicted with this model, and higher on sunny days. The monthly average values for T_{air}, H_{sun} and H_{light} are provided by the meteorological institute. For The Netherlands, this results in a year-round asphalt temperature according to Fig. 7.

The number of HGVs is not equally distributed over time. Based on measurements in [2], the distribution of HGVs per day is approximated by:

$$n_{day}(t) = \frac{1}{24} + 0.03 \cdot \sin\left(\frac{t \cdot \pi - 6\pi}{12}\right)$$
$$-0.005 \cdot \sin\left(\frac{t \cdot 3\pi - 6\pi}{12}\right)$$

(5)

Eq. (2), (4) and (5) result in the fraction of the number of HGVs n_{frac} accompanied by certain asphalt stiffness, see Fig. 8. This is used in order to determine the stress multiplication factor due to asphalt R according to Eq. (3). Fig. 8 is based on equations typical for Dutch highways. However, the same methodology as provided here can be applied to other countries.

3.3 Fatigue classification of deck plate crack

Several fatigue test series have been carried out to assess the detail classification. The test results are summarized
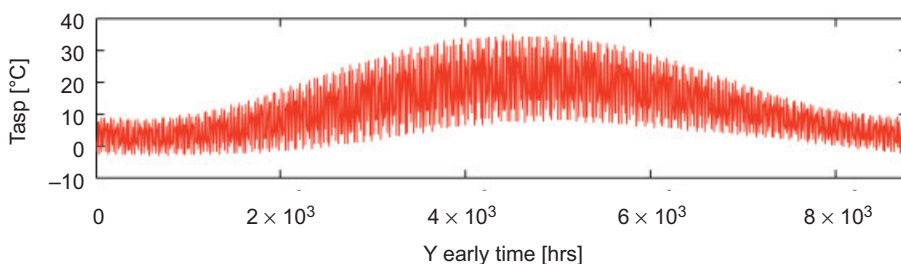

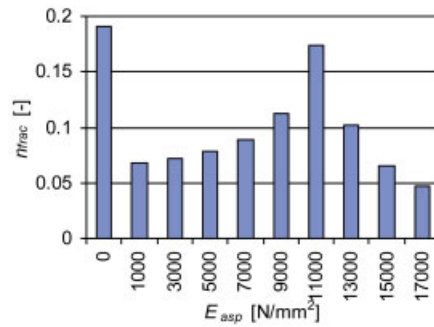Fig. 7. Year-round asphalt temperature

in *Kolstein* [6]. The fatigue strength at 2 million cycles corresponding with the observation of a through-thickness crack was determined as $\Delta\sigma_{c,m} = 197$ N/mm² and $\Delta\sigma_{c,m-2sd} = 147$ N/mm² (where $\Delta\sigma_{c,m}$ = mean fatigue strength and $\Delta\sigma_{c,m-2sd}$ = mean – 2 × standard deviation). However, *Kolstein* [6] recommended using a criterion of 10 % strain fall instead of a through-thickness crack. This stricter criterion is based on the fact that visual inspection of small cracks is not feasible in the case of a real deck with surfacing. Thus, it is better to avoid crack propagation of this type of crack into the deck plate. The fatigue strength at 2 million cycles corresponding with 10 % strain fall is $\Delta\sigma_{c,m} = 180$ N/mm² and $\Delta\sigma_{c,m-2sd} = 125$ N/mm². These values are used in the current assessment.

3.4 Fatigue life analysis

The fatigue life of the Brienenoord deck was calculated using average fatigue strength values for a through-thickness crack ($\Delta\sigma_{c,m} = 197$ N/mm² with $\gamma_M = 1.0$) and the spectrum in Fig. 6 for "proposed model, past". A fatigue life prediction using the damage rule of *Palgrem-Miner* then results in an average life of 7.4 years. This is determined for the case where the central wheel position (i. e. distance 0 mm in Fig. 5b) is centred over the trough. A modification in this assump-

tion has a relatively small influence on the life prediction: when the central wheel position is changed by 150 mm (i. e. located directly over the trough wall), the predicted life increases by approx. 20 %. On the contrary, an accurate fatigue load model appears to be important for the life prediction: when FLM4 is used instead of the proposed model, the predicted life is 2.0 instead of 7.4 years.

Comparing the predicted life of 7.4 years with the actual life of 7 years at the time of discovering cracks in the deck plate, it can be concluded that the prediction is accurate. Using a comparable model as outlined in this paper, *De Jong* calculated the fatigue lives of the deck plates of the bridges in Table 1 [7]. For all bridges, the calculated life using average fatigue strength values agrees well with the observed life: the average ratio between the calculated life and the real life was 0.95 and the standard deviation was 0.15. This gives confidence in the model.

4 Design for new bridge decks
4.1 Required deck plate thickness using the prediction model

The validated prediction model will now be used to determine the required deck plate thickness for the design of new bridges. The geometries considered have a trough wall thickness of 8 mm and a centre-to-centre distance of the trough walls of 300 mm. The surface layer is varied: either 60 mm asphalt or 6 mm epoxy surfacing. For the latter case, an alternative is considered in which the centre-to-centre distance of the trough walls is reduced to 220 mm. The required deck plate thicknesses are provided for a design life of 50 years and 100 years.

The calculations are carried out using the proposed fatigue load model and using a design fatigue strength $\Delta\sigma_{c,m-2sd} = 125$ N/mm². Long deck plate cracks may cause serious traffic accidents. However, the main load-bearing function of the bridge remains intact even for long deck plate cracks. For this reason, a partial factor $\gamma_M = 1.15$ is applied in the design calculations. This factor is provided in EN 1993-1-9 [10] for safe life and low consequences of failure. The resulting deck plate thicknesses are provided in Table 5. A similar table is provided in the Dutch National Annex to EN

Table 5. Design values for deck plate thickness t [mm] [3)]

Type of road	No. of HGVs per year	Asphalt surface[1)] t_s = 60 mm, L = 292 mm [2)]		Epoxy surface t_s = 6 mm, L = 292 mm		Epoxy surface t_s = 6 mm, L = 202 mm	
		t_d = 50 yrs	t_d = 100 yrs	t_d = 50 yrs	t_d = 100 yrs	t_d = 50 yrs	t_d = 100 yrs
cat. 1	$2 \cdot 10^6$	18 (16)	19 (17)	21	22	16	17
cat. 2	$0.5 \cdot 10^6$	17 (14)	18 (15)	19	20	15	16
cat. 3	$0.125 \cdot 10^6$	15 (12)	16 (13)	17	18	13	14

[1)] Figures in brackets result when the stress reductions due to load spread and surface stiffness are both considered. Figures not in brackets result only when the load spread effect is considered.
[2)] Span = centre-to-centre distance of trough walls – 1 × trough wall thickness.
[3)] The deck plate thickness can be reduced by approx. 1 mm if through-thickness cracks are considered acceptable.

1993-2. Since the model is validated for dimensions according to Table 2, Table 5 cannot be used for cases where the bridge geometry deviates too much from the dimensions in Table 2.

Table 5 provides two values for the deck plate thickness for an asphalt surface. The figures in brackets result if the model described in section 3 is applied. The figures not in brackets result if the stiffness of the surfacing layer is ignored and only the load-spreading effect of the surfacing is taken into account. The authorities in The Netherlands have adopted the latter case in the design of new bridges for the following reasons:
– The asphalt surface may be cracked in practice.
– There is significant uncertainty and variation in the yearly asphalt temperature.

Table 5 indicates that the design deck plate thickness can be relatively large. The failure criterion used in the calculations is strict (section 3.3). If through-thickness cracks are considered acceptable, the calculations can be carried out with $\Delta\sigma_{c,m-2sd}$ = 147 N/mm^2 instead of $\Delta\sigma_{c,m-2sd}$ = 125 N/mm^2. In this case, the design deck plate thickness is reduced by approx. 1 mm. A further reduction is possible when applying a thicker surface layer such as 80 mm thick asphalt. Additional FE calculations are required to determine the SCF for this situation.

4.2 Considering inspections in the design

The required deck plate thickness can be reduced if the deck plate is inspected regularly. As an example, the required inspection interval is calculated below for the case of a movable bridge with a thin epoxy surfacing.

Firstly, a fracture mechanics (FM) analysis is carried out in order to obtain an insight into crack growth at this detail. The stress intensity factor required for this analysis is based on the equations for a T-stub joint in BS 7910 [13]. Several literature sources – including *Dijkstra* [3] and BS 7910 – give a description of the FM calculation. The crack growth is determined with the following equation:

$$\frac{da}{dN} = C\left(\Delta K^m - \Delta K_{th}^m\right) \qquad (6)$$

where:
da/dN   crack growth per stress cycle [mm]
ΔK stress intensity factor, a function of crack growth, detail geometry and stress level [N/mm$^{3/2}$]
m, C, ΔK_{th} material-dependent crack growth parameters

The FM calculation is carried out using M+2SD crack growth parameters for a stress ratio R = –1, with values m = 3 [–], C = 3 · 10^{-13} [N, mm] and ΔK_{th} = 80 [N/mm$^{3/2}$]. A partial

factor γ_M = 1.15 is applied. The initial defect selected is a semi-elliptical crack with a depth of 0.15 mm and width of 0.30 mm.

As the crack grows, the stress at the tip decreases due to the greater distance to the cross-beam. The stress as a function of the distance to the cross-beam is determined with FE calculations. These calculations indicate that the stress is reduced to 40 % of the maximum value at a distance of 80 mm from the cross-beam, see Fig. 9. Note that this stress reduction is determined for a deck plate thickness t = 12 mm. In the remaining part of this section, it is assumed that this reduction is also representative of other thicknesses. The stress reduction as a function of the crack dimensions is taken into account in the calculation of ΔK.

As a check of the procedure described above, the stress range corresponding with a through-thickness crack after $2 \cdot 10^6$ cycles is determined with the FM calculation. The resulting stress range is 144 N/mm^2 (γ_M = 1.0). This value is close to the fatigue strength determined by tests ($\Delta\sigma_{c,m-2sd}$ = 147 N/mm^2).
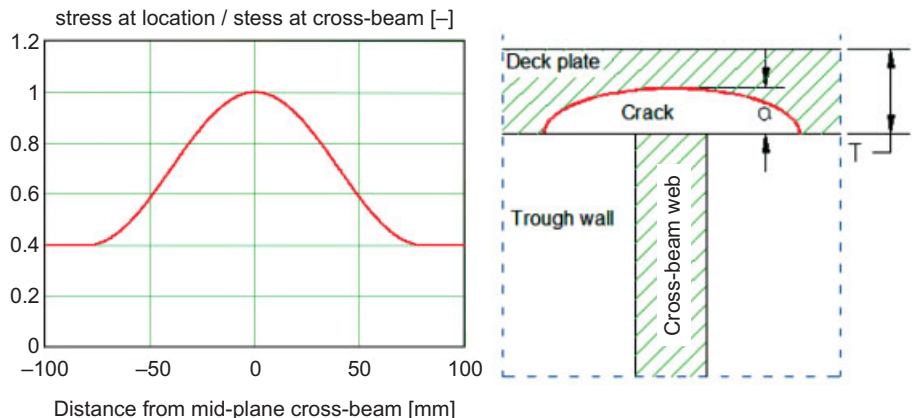

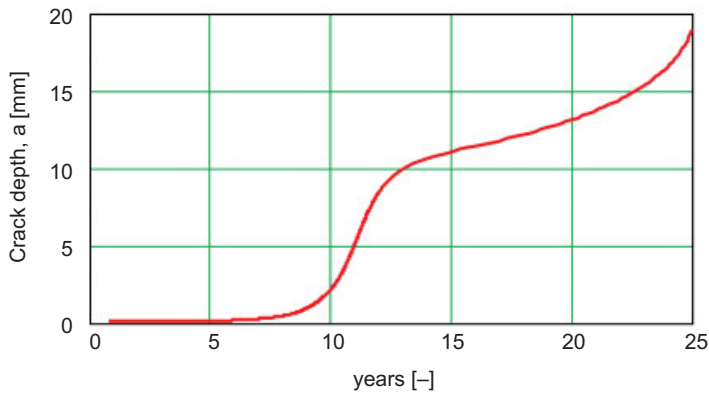*Fig. 9. Stress path in deck plate near cross-beam web*

Fig. 10. Design crack growth as function of the fatigue life for an orthotropic deck with t = 19 mm, t_s = 6 mm (epoxy), L = 292 mm and γ_M = 1.15

Fig. 10 presents the resulting crack growth determined with FM for an orthotropic deck with t = 19 mm, t_s = 6 mm (epoxy), L = 292 mm and γ_M = 1.15. The calculated fatigue life for a through-thickness crack is 25 years. As a comparison, the life determined with the S-N approach for $\Delta\sigma_{c,m\text{-}2sd}$ = 147 N/mm² is 33 years.

Secondly, the inspection technique is considered. In The Netherlands, inspections are carried out on a regular basis using the pulsed eddy current technique. This technique enables deck plate cracks to be detected through the surface finishes. The method is fast and accurate, but can only determine cracks that have grown through the plate thickness and are 100 mm long. For this reason, additional inspections are carried out. A deck plate crack that is just beginning can be detected with high accuracy using time-of-flight diffraction (TOFD). The inspection is carried out from above the deck and is able to detect cracks that have not yet grown through the deck plate. However, the surfacing must be removed before the inspection can take place. For the calculation, it is assumed that the maximum life of the epoxy surfacing is 10 years. Thus, a TOFD inspection can be carried out every 10 years. If, in an inspection, a crack has been detected with a depth of, for example, 3 mm, this information can be used to update the crack growth calculation and with that the residual life. An example is shown in Fig. 11, where a crack has been found with a depth of 3 mm after 20 years (green dot). The figure also shows the mean crack growth curve (purple) and the mean + 2 standard deviation crack growth

curve (blue). The probable crack growth after detection of the crack is shown by the green shading. It is possible to determine the residual fatigue life with the necessary reliability if the distributions of all relevant parameters – such as the stress intervals, the number of cycles and the crack growth parameters – are known. Where this information is not known, a safe but conservative approach is to assume crack growth parallel to the first design curve (dark green curve, indicated by an arrow). In this example, the end of life of the updated calculation is reached 14 years after the last inspection. Before this end of life, a new inspection will have taken place. Thus, the inspection interval of 10 years is sufficient.

Finally, we need to consider the possibility that the inspection method fails to detect a crack that is present in the deck. In general, large cracks can be detected with greater reliability than small cracks. Cracks smaller than a certain size cannot be detected. Thus, the probability of detec-

tion increases with increasing crack size. The probability of detection can be described with the following equation:

$$POD = 1 - \exp\left(\frac{\alpha_{POD} - a}{\beta_{POD}}\right) \tag{7}$$

POD probability of detection
a crack depth measured from bottom of deck plate (Fig. 9)
α_{POD} crack depth that is just detectable
β_{POD} standard deviation of inspected crack, or shape parameter of POD curve

According to *De Jong* [7], the parameters of the POD curve for the TOFD inspection are α_{POD} = 1.5 mm and β_{POD} = 0.5, based on a deck plate thickness of 12 mm. In this example, it is assumed that the same POD curve parameters are valid for a deck plate thickness of 20 mm.

The fact that a crack is present but is not detected needs to be considered in the inspection-based design. As an example, consider a required reliability index of β = 3.6. According to EN 1990 [11], this corresponds with a failure probability of $P_f = \Phi(-0.4 \cdot 0.8 \cdot \beta)$ = 0.11 (assuming a normal distribution). Factor 0.8 in this expression is the sensitivity factor for resistance and factor 0.4 is the factor for a non-dominant variable (crack growth parameters are considered as dominant variables in this example, which needs to be checked). The required POD is in this case $(1-P_f)$ = 0.89. This value together with Eq. (7) can be used to determine the crack dimension that needs to be
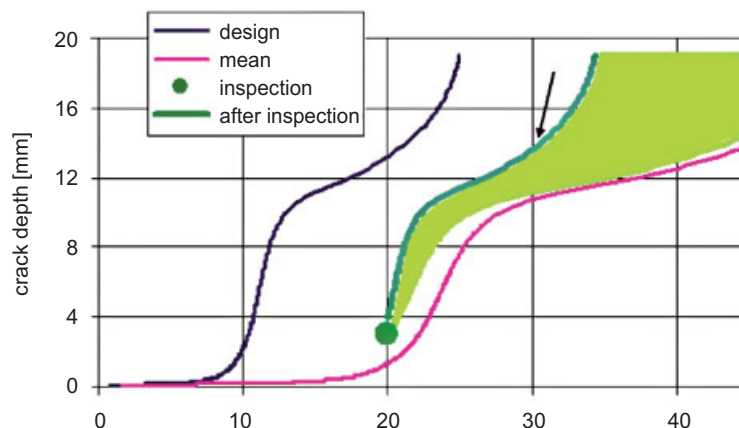

Fig. 11. Residual lifetime based on inspection result

considered in the analyses in case a crack has not been found in the inspection:

$$a_{no\,crack} = \alpha_{POD} - \beta_{POD} \times$$
$$\times \ln\left(1 - \left(1 - P_f\right)\right) = 1.5 - 0.5 \times$$
$$\times \ln\left(0.11\right) = 3 \text{ mm}$$

$$(8)$$

This section shows that it is possible to reduce the deck plate thickness from 22 mm to 19 mm when carrying out accurate inspections every 10 years. For the other configurations in Table 5, a reduction in the required deck plate thickness by approx. 3 mm appears to be feasible as well.

Before the procedure can be applied for large deck plate thicknesses, the following assumptions need to be checked:

– The stress path in the deck plate (Fig. 9) as well as the fatigue strength are derived for a deck plate thickness of 12 mm. It is important to check whether this also applies to thicker deck plates.
– The parameters of the POD curve are determined for a deck plate thickness of 12 mm. It is important to check whether this also applies to thick deck plates.

5 Conclusions and future work

This investigation has resulted in the following conclusions:

– An assessment procedure has been developed to determine the fatigue life of the deck plate of orthotropic bridge decks for cracks as observed in The Netherlands, starting at the welded joint at the junction of deck plate, trough and cross-beam. The procedure consists of a fatigue load model, a mechanical stress model and the classification of the detail.

The fatigue life predicted with this model agrees well with the life observed in practice.

– The fatigue life of the deck plate of the movable Van Brienenoord Bridge was only seven years. This short life is due to the combination of the large number of HGVs crossing the bridge, the thin surfacing and a relatively thin deck plate (12 mm).
– The required deck plate thickness for bridges in Dutch highways with a design fatigue life of 100 years is substantially larger than that applied in the past in The Netherlands.
– Fatigue load model 4 according to EN 1991-2 is conservative when compared with the real fatigue load, even for the busy highway network in The Netherlands.
– It is certainly possible to take account of inspection results in the assessment of the (residual) life of an orthotropic steel bridge deck. By considering the inspections in the design, the required deck plate thickness can be reduced by approx. 3 mm.

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