ce/papers Proceedings in civil engineering

Ernst & Sohn A Wiley Brand

Check for updates

ORIGINAL ARTICLE

eurosteel AMSTERDAM 2023

Ernst & Sohn A Wiley Brand

Fatigue verification of orthotropic bridge decks with the hot-spot method – Background to TS 1993-1-901

Johan Maljaars[1,2] | Richard Pijpers[1] | Milan Veljkovic[3] | Frank van Dooren[4]

Correspondence

Prof.dr.ir. Johan Maljaars
TNO, Department of Reliable Structures
Molengraaffsingel 8
2629 JD Delft
The Netherlands
Email: johan.maljaars@tno.nl

[1] TNO, Delft, The Netherlands
[2] Eindhoven University of Technology, Eindhoven, The Netherlands
[3] Delft University of Technology, Delft, The Netherlands
[4] Rijkswaterstaat, Utrecht, The Netherlands

Abstract

The design of orthotropic bridge decks (OBD) is driven by fatigue. The Eurocode on fatigue of steel structures – prEN 1993-1-9:2023 – provides fatigue resistance (S-N) curves for relevant details in OBD. These curves are to be used with the far field (nominal) stress. However, due to the complex geometry and loading of OBD, the nominal stress is not well defined for many details. In addition, cracks have been found in practice in OBD at locations for which S-N curves are not provided. In order to overcome these issues, experimental and numerical studies are carried out on the use of local stresses (hot-spot stresses where possible) for the fatigue design of OBD. The different stress extraction method also implies the use of different S-N curves. A large number of fatigue tests carried out in the past have been re-evaluated to determine the hot-spot stress S-N curves. Additional fatigue tests have been carried out for details lacking data. The study has resulted in a guideline to design OBD for fatigue with the hot-spot stress method. This guideline, TS 1993-1-901:2023, will be published as a Technical Specification (TS) to the standard prEN 1993-1-9:2023.

This paper provides a short background to the newly created TS. It demonstrates the derivation of S-N curves based on available tests for a number of details. It highlights fatigue relevant details that are missing in prEN 1993-1-9:2023 and that are added to TS 1993-1-901. It explains how to evaluate cracks starting from the root of the weld, for which the hot-spot stress method is traditionally not applicable. Finally, the paper demonstrates how the method can be applied in the fatigue design, including the application of the load in finite element models.

Keywords

Fatigue, Fatigue Test Data, Orthotropic Bridge Deck, Finite Element Method, Eurocode, TS 1993-1-901

1 Introduction

Orthotropic bridge decks (OBD) transfer the loads exerted by traffic to the main superstructure of a bridge. They consist of a deck plate supported in longitudinal (traffic) direction by closed or open stiffeners. The deck and the stiffeners are supported by crossbeams in transverse direction, Fig. 1.

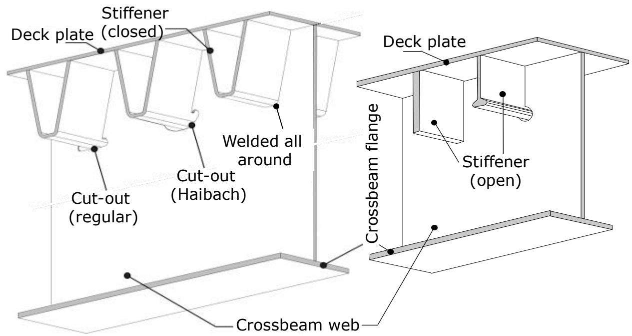


Figure 1 OBD structures, with main elements named.

OBD are popular for long span bridges and movable bridges because of their high strength-to-weight ratio. However, OBD are sensitive to fatigue deterioration and fatigue is usually decisive for the structural design. The American AASHTO guideline [1] and the European standard prEN 1993-1-9 [a] [2] give tables with the characteristic fatigue resistance of most fatigue-sensitive details in OBD. These tables apply to the nominal stress method, i.e., far field stress ranges should be used in the verification. However, OBD exhibit large stress gradients and the nominal stress is not defined for such a case. This is problematic when FE models consisting of shell or solid elements are used to compute stresses.

Because of a lack of definition of nominal stress, some important details at which cracks are observed in practice [3]0[5] are even not mentioned in prEN 1993-1-9 [2], see Fig. 2. As a solution for the mentioned issues, a new fatigue design approach for OBD based on geometric (i.e.

[a] The latest versions of the next generation of Eurocodes are referred to here because they contain the most up-to-date design information.

https://doi.org/10.1002/cepa.2664 wileyonlinelibrary.com/journal/cepa

local) stresses is developed. This approach is provided in the new Technical Specification TS 1993-1-901 [6], which is supplementary to prEN 1993-1-9 [2]. This paper gives a short background.

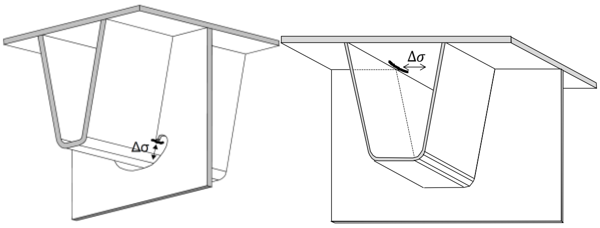


Figure 2 Fatigue-sensitive details not covered in prEN 1993-1-9 [2].

2 Stress computation

The type of stress adopted depends on the type of detail. Where possible, the hot-spot stress method is used as the design stress parameter, Eq. (1). This applies to weld toes, but in the underlying study of this paper, it appeared a good parameter also for the deck plate crack growing from the root of the single sided stiffener-to-deck weld, Fig. 3(a). The stress used for weld throat cracks in that same weld is given with Eq. (2), with the geometry parameters a and e as in Fig. 3(b). The effect of the weld dimension and the lack of penetration expressed through Eq. (2) has been verified by a comparison with the effective notch stress method, the latter applied in accordance with the IIW fatigue recommendations [7]. The stress used for verifying weld throat cracks of double sided fillet or partial penetration welds is taken from prEN 1993-1-9 [2], Eq. (3). Finally, the stress parameter for cracks in the base metal starting from the crossbeam cut-out is taken as the maximum principal stress parallel to the cut-out at the cut-out edge.

$$\sigma_{hs} = 1.5\sigma_{0.5t} - 0.5\sigma_{1.5t} \quad (1)$$

$$\sigma_{wf} = \frac{F}{a} + \frac{6}{a^2}\,(M + F \cdot e) \quad (2)$$

$$\sigma_{wf} = \frac{F}{a} + \frac{M}{a(a+t)} \quad (3)$$

Where F is the normal force per unit length and M is the bending moment per unit length.

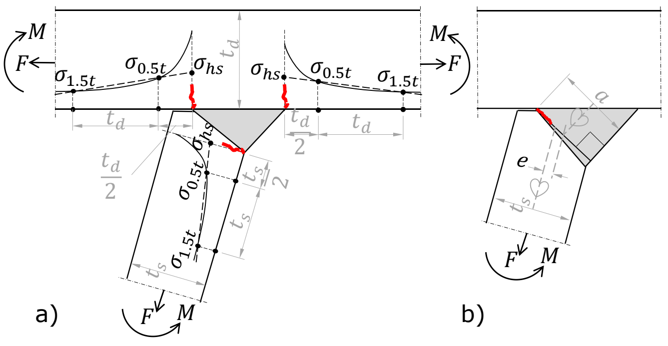


Figure 3 Stress parameters for the details at the stiffener-to-deck weld a) hot-spot stress b) weld throat stress.

The stress per detail can be computed using the finite element method. A model consisting of solid elements including the modelling of welds and their lack of penetration, if any, is suited for all details. However, such a model requires significant effort from engineers to build such a model for OBD. A model composed of shell elements requires less effort but the hot-spot stress is not always computed accurately with such a model. A method is therefore proposed –based on a combination of the methods in [8] and [9] – that gives a satisfactory approximation of the hot-spot stress with shell elements. This method, where the shell elements in the vicinity of the welds are assigned a larger plate thickness, is described in detail in the companion paper [10].

3 Fatigue resistance

The computed stress is generally significantly higher than the nominal stress (if the nominal stress can be defined). However, the fatigue strength is also higher. prEN 1993-1-9 [2] defines the characteristic fatigue resistance, $\Delta\sigma_C$, as the stress range at which the detail survives $2{\cdot}10^6$ cycles with a 5% lower prediction bound. Annex B of the standard gives $\Delta\sigma_C$=90 MPa for load carrying fillet welds, $\Delta\sigma_C$=112 MPa for ground flush butt welds and $\Delta\sigma_C$=100 MPa for all other weld types, in all cases for weld toe cracks verified with the hot-spot stress method, see the appendix at the end of this paper. Tests on specific details in OBD show that the actual characteristic resistance is often higher than the Annex B values. Important reasons are:

- The plate thickness of some OBD components – notably the stiffener – is significantly smaller than the standard thickness of $t \approx 25$ mm used as the reference for prEN 1993-1-9 [2]. Welds in relatively thin plates can have a relatively high fatigue resistance [11].
- Many details in OBD are loaded in compression, in bending, or in a combination of these two. Despite of the residual stress, the mean stress can still be relevant in welded joints [12].
- An OBD is generally tolerable to cracks. Growing cracks often enter areas of remote stress, causing a relatively low crack growth rate for some details [13].

For these reasons, test data are collected from literature and additional fatigue tests are carried out in the authors' laboratories for those details, weld procedures, or plate thicknesses for which the number of tests found in the literatures was small or nil. In total, more than 1000 fatigue test results are collected and evaluated, but not all are used for establishing the characteristic fatigue strength because run-outs should be excluded according to the Eurocode system [14] or because the loading mode applied in the tests (e.g. tension-tension) deviated from that in bridge decks (e.g. bending). Fig. 4 gives examples of data for some details together with the S-N curve established from it. The S-N curves have a predefined inverse slope parameter $m_1 = 3$. The curves are derived using the procedure in [14] and subsequently they are rounded down to the nearest predefined curve.

Linear elastic fracture mechanics simulations – based on BS 7910 [15] and validated with tests – are used to further support the characteristic fatigue resistance. This is done because limited test data are available for a number of details. For example, the amount of data found for OBD with open stiffeners is limited. The data of all details including the sources, tests carried out in the author's laboratories and the statistical evaluation of the data, can be found in [16][17].

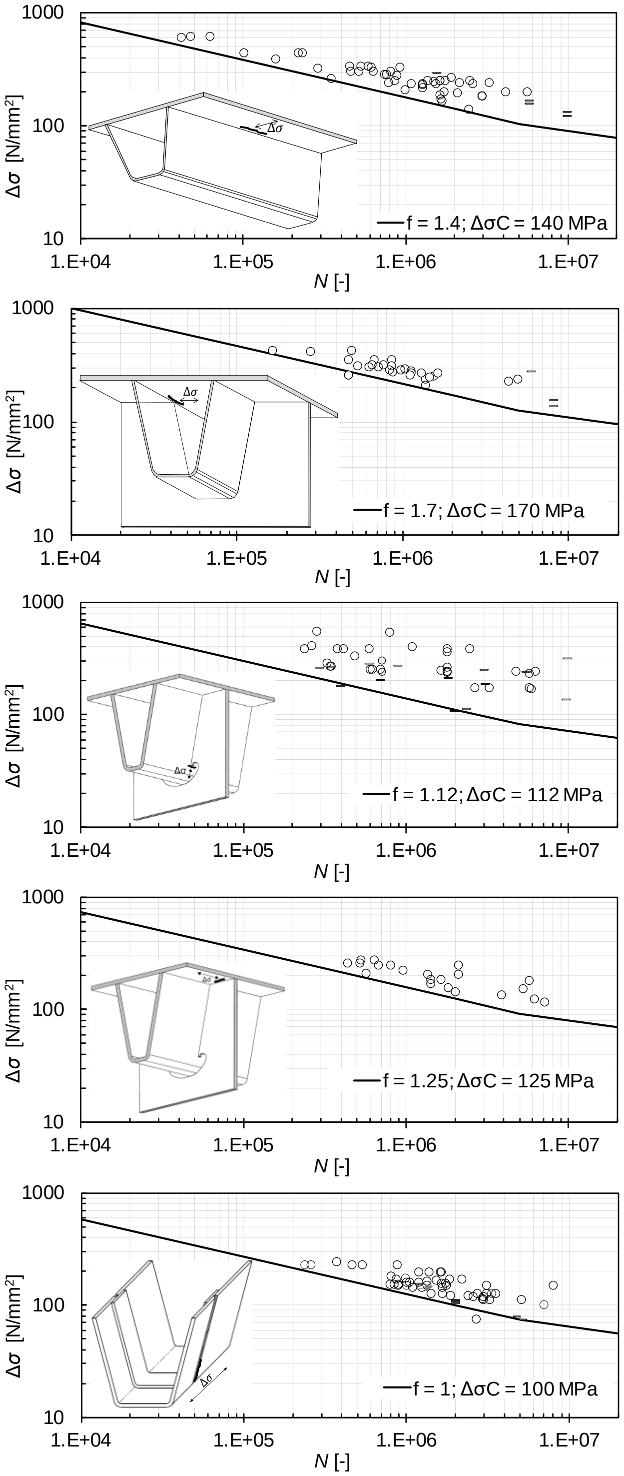


Figure 4 Examples of test data and S-N curves derived from these test data, with factor f according to Eq. (4).

To preserve the link with the characteristic fatigue resistance in prEN 1993-1-9 [2] (including the hot-spot stress resistance in Annex B), the resistance $\Delta\sigma_c$ in prTS 1993-1-901 [6] is presented using a detail-specific enhancement factor f:

$$\Delta\sigma_c = f\Delta\sigma_{c,Table} \tag{4}$$

where $\Delta\sigma_{c,Table}$ is the standard characteristic fatigue resistance according to [2]. Factor f is indicated in the graphs of the example details in Fig. 4. Considering all details, factor f ranges between $1 \leq f \leq 2$.

4 Weld geometry

The test data and the evaluations with the effective notch stress method show that the geometry of the stiffener-to-deck weld has a large influence on the fatigue resistance of multiple crack types. The weld of the stiffener-to-deck weld should not be too small to allow for smooth weld toe transitions. Contact between the edge of the stiffener and the deck plate should be ensured to obtain a high fatigue resistance for weld root cracks. This can be achieved in practice by pressing the stiffener against the deck plate during welding. The lack of penetration of the weld should be limited for a high fatigue resistance against weld throat cracks and weld toe cracks in the stiffener web. These conditions have resulted into geometrical restrictions as indicated in Fig. 5(a). Since the restrictions may not be easy to comply to in practice, the authors are currently studying if factors f can be derived for the case that the restrictions are not fulfilled. Relaxing the restrictions – at the cost of lower factors f – might be cost efficient e.g. in case of OBD in local roads with low lorry numbers.

The weld geometry appears also crucial for the stiffener splice joint with backing strip. The distance between the two sides h_6 (Fig. 5(b)) should be sufficiently large to prevent premature failure due to cracks initiating from the weld root. For the same reason, tack welds should be continuous and should be applied inside the shape of the final weld. Stop start positions of the welding operation should be located outside the highly stressed region, i.e., in the upper half of the stiffener.

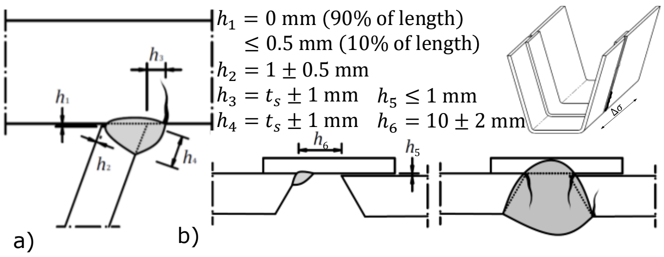


Figure 5 Geometrical restrictions to some welds a) the stiffener-to-deck weld; and b) the stiffener splice weld with backing strip.

5 Loads

A fatigue design based on local stresses further requires a load model of sufficient accuracy. A factored single vehicle load model (called lambda model, using FLM3 or LM71 as the basis for road or railway bridges, respectively, in prEN 1991-2 [18]) is allowed in most European countries for the design of bridge superstructures. However, these models are not designed for influence lines shorter than 20 m [19] and they are shown to be inaccurate for short span structures. Another load model is therefore to be used.

FLM4 in [18] is developed for components with short influence lines in road bridges [19]. It consists of five lorries that should cross the structure of design in certain quantities. This model is used for the verification of OBD in [6]. Similarly, Annex D of [18], consisting of twelve train types, is used for OBD in railway bridges. As an alternative to these models, recorded traffic – e.g., through weigh-in-motion – may be used. Both models require the computation of the accumulated fatigue damage D:

$$D = \sum_i n_i / N_i \tag{5}$$

where n_i is the number of applied cycles of stress range $\Delta\sigma_i$ and N_i is the number of cycles to failure (according to the S-N curve) for the same stress range.

Application of the load model for railway bridges is relatively straightforward. In road bridges, however, the lateral position of the vehicles varies. FLM4 provides a distribution for the lateral position, Fig. 6 a). The load effect depends on the lateral position especially for the details close to or in the deck plate. Since the location of the distribution on the bridge deck depends on the alignment of the lanes, which is unknown in the design, the most adverse one should be selected. However, the most adverse position is different for different details. For this reason, three locations of the lateral distribution need to be considered for OBD with closed stiffeners, namely, with the centre of the two-wheel axles located: 1) above the stiffener, 2) in between two stiffeners, and 3) above a stiffener web, see Fig. 6 b). The damage D should be computed for each of these lateral positions (Eq. 5) and it should be verified that each of them is equal to or lower than 1.

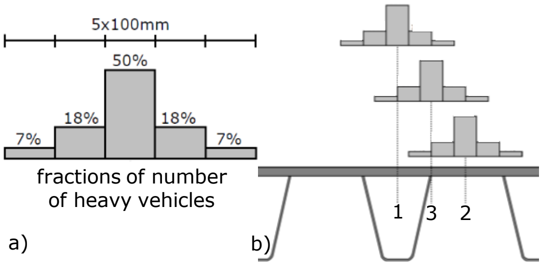


Figure 6 Distribution of the lateral vehicle position a) for FLM4 in prEN 1991-2 [18] and b) locations of the distributions on the bridge deck, referring to of the centre of two-wheel axles.

Evaluations using a predecessor of [6] have shown that the maximum fatigue damage is captured with reasonable accuracy if these three locations are considered, even if the most adverse position is somewhere in between [20]. Two positions of the lateral distribution are deemed sufficient for OBD with open stiffeners, namely, centred above the stiffener and centred between two stiffeners.

It should be noted that the tyre contact areas given for FLM4 in [18] are too long and slightly too narrow compared to measurements. This may result in a non-conservative design, see [21] for a comparison and a proposed modification to the tyre contact area. This is an issue with the load model only and it has therefore not been adjusted in prTS 1993-1-901 [6].

6 Asphalt

Many OBD in road bridges are equipped with asphalt pavement. The asphalt may cause dispersion of the load through the asphalt layer. More importantly, it may increase the flexibility of the deck plate, causing more stiffeners to participate in the load transfer, Fig. 8. However, both effects depend on the stiffness of the pavement, which depends on the composition, the temperature and the strain rate. The asphalt stiffness can further increase in time due to ageing or reduce due to fatigue. This complicates the evaluation of load effects in the steel OBD.

Three approximations of the asphalt contribution to stress reduction in the steel structure are included in prTS 1993-1-901 [6]:

- The least conservative option – but also the most complex of the three for engineers in practice – considers the temperature dependency of the asphalt stiffness. The pavement needs to be included in the finite element model with at least two layers of solid elements. The stresses should be computed five times, for five different Young's moduli of asphalt related to five temperatures. The fraction of heavy vehicles per temperature bin is given. It is based on measured temperature fluctuations per day and per year and on the distribution of the number of vehicles over a day [22]. The temperature-dependent stiffness is given for two asphalt compositions. They are measured at strain rates representative of flowing traffic [3]. Ageing and fatigue are ignored, which is usually a conservative approximation. The membranes applied between asphalt layer and deck may be modelled assuming a certain shear stiffness or ignoring their shear stiffness.
- Alternatively, the asphalt can be modelled in the same way as above, but using a single asphalt stiffness associated with the maximum (measured) temperature. This option is more conservative, but also requires less work from the engineer because the model is evaluated for one temperature only.
- The most conservative approximation – but the least complex one – does not use explicit modelling of the asphalt layer. Only the increased load contact surfaces due to load dispersion through the asphalt are incorporated. The dispersion is taken at a spread-to-depth ratio of 1 horizontally to 2 vertically down to the level of the centre plane of the deck plate. This spread-to-depth ratio is a conservative approximation based on simulations with a model comparable to that of the first option. It is noted that prEN 1991-2 [18] uses a dispersion for ultimate limit state of 1 to 1 down to the level of the centroid of the deck plate. Ultimate limit state has not been considered in this work. The dispersion of 1 to 1 appears too optimistic for the fatigue limit state according to the (linear elastic) simulations.

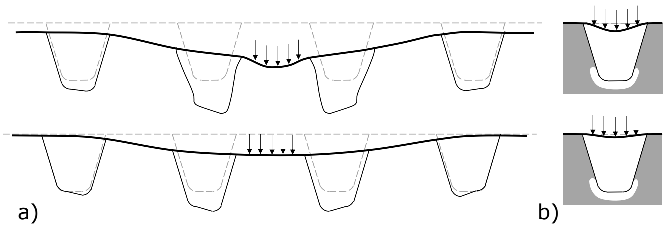


Figure 7 Effect of asphalt stiffness on the load distribution in OBD a) between crossbeams b) at the crossbeam location: top = low stiffness; bottom = high stiffness.

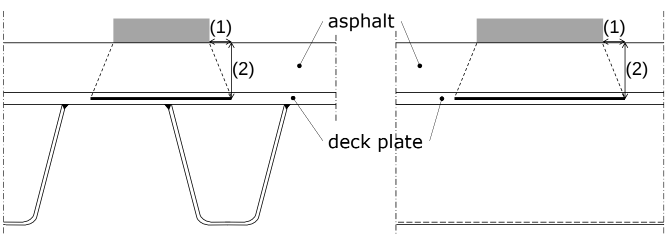


Figure 8 Load dispersal in the third mentioned approximation of asphalt pavement.

7 Experience from practice

The verification of OBD using local stresses as proposed in this work (and adopted in [6]) has been applied by engineers in practice for the design or the assessment of the OBD of four bridges in The Netherlands, see Fig. 9. In addition, predecessors of the method have been applied to 17 bridges, see e.g. [23][24]. Some general lessons learned from these designs are:

- Engineering offices can straightforwardly apply the method, but it requires quite some work because the number of analyses is large. Good bookkeeping is therefore required and automatization of the process is advantageous. It should be noted that FLM4 and Annex D are the required fatigue load models also for the superstructure of bridges in the Netherlands. Hence, the engineering offices involved were already experienced with designing for fatigue using these models.
- The OBD structures resulting from the analyses have the same general lay-out as the recommendations given in the informative Annex C of prEN 1993-2 [25], but some dimensions and details are different, such as a slightly thicker deck plate following from [6] and slightly stricter tolerances of the weld dimensions, especially that of the stiffener-to-deck plate weld.
- OBD with stiffeners fitted in between crossbeams have generally shown to perform worse compared to OBD with stiffeners passing through the webs of crossbeams. Annex C of prEN 1993-2 [25] therefore allows the former type of OBD only in case of light traffic (without a definition of light). Such a limitation is not given in [6], but the fatigue resistances of the decisive details is low, so that such a deck will either not pass in the verification or the plates will be thick in case of heavy traffic.
- Considering the designed and the assessed structures, the authors have the impression that the method in [6] creates larger consistency in the fatigue verification of OBD by different engineering offices.

8 Conclusions

This paper gives a short background of the fatigue design method of orthotropic bridge decks (OBD) put forward in the Technical Specification TS 1993-1-901. The method uses a local stress parameter (the hot-spot stress where possible) as the basis for the verification, because the nominal stress is not straightforwardly defined for structures with large stress gradients such as OBD. The local stresses are generally higher than the nominal stresses (if the latter can be defined), but so are the fatigue resistances.

The fatigue resistances obtained from the tests are often higher than the standard resistances in prEN 1993-1-9 [2] because the loading mode in OBD is often more favourable (bending-bending, compression-tension or compression-compression instead of tension-tension), because the plates of some components (notably the stiffener) are relatively thin, and because of the shielding effect in case of growing cracks.

The method should be used with a detailed fatigue load model, because Eurocode's so-called lambda method is not developed for short span components occurring in OBD. The effects of asphalt on the stress levels can be taken into consideration in an approximate way, because realistic modelling of the asphalt stiffness requires a large effort.

Experiences from practice with the method indicate that engineering offices are able to use it straightforwardly. The first impression is that the method has helped in reducing the differences in verifying OBD for fatigue between different engineering offices. The OBD resulting from designs with the method have some differences in dimensions and weld geometries compared to the recommendations given in Annex C of prEN 1993-2 [25].

Bridge: Van Brienenoord
Bridge type: fixed and movable
Structural design: Arup
Stiffeners: Closed
Remark: OBD design and comparison of alternatives

Bridge: Haringvlietbrug
Bridge type: fixed
Assessment: Witteveen & Bos
Stiffeners: Open

Bridge: Schipholbrug
Bridge type: movable
Structural design: Nobleo / IV Infra
Stiffeners: Closed

Bridge: Optimization study
Bridge type: movable
Structural design: RHDHV
Stiffeners: Open & closed

Figure 9 Bridges with OBD designed according to [6].

Acknowledgements

This study is sponsored by Rijkswaterstaat.

Appendix – Hot spot method in prEN 1993-1-9 [2]

Eq. (6) gives $\Delta\sigma_C$ of the hot-spot method in [2] (excl. tubular joints), where $\Delta\sigma_{c,Table}$, β and t_{eff} are given in Table 1.

$$\Delta\sigma_c = \begin{cases} \Delta\sigma_{c,Table} & \text{for } t \le 25\text{ mm} \\ \Delta\sigma_{c,Table}\left(t_{eff}/25\text{ mm}\right)^{\beta} & \text{for } t > 25\text{ mm} \end{cases} \quad (6)$$

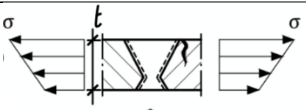
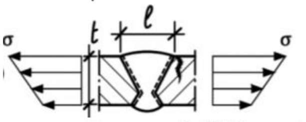
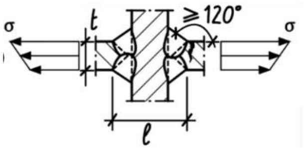
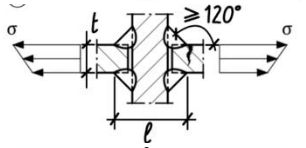
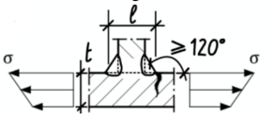
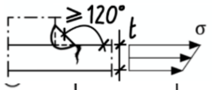
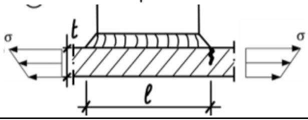
References

[1] AASHTO LRFD (2012). *Bridge design specifications*. American association of state highway and transportation officials.

[2] prEN 1993-1-9 (2023*). Eurocode 3: Design of steel structures - Part 1-9: Fatigue* (Version for enquiry). Brussels: CEN.

[3] De Jong, F.B.P. (2007) *Renovation techniques for fatigue cracked orthotropic steel bridge decks*. Delft: TU Delft.

Table 1 Characteristic fatigue resistance of the hot-spot method in [2].

Detail	$\Delta\sigma_{C,Table}$	β	t_{eff}
	112	0.1	t
	100	0.2	$\min(t; 0.66\ell + 14\text{mm})$
	100	0.3	$\min(t; 0.66\ell + 14\text{mm})$
	90	0.3	$\min(t; 0.66\ell + 14\text{mm})$
	100	0.3	$\min(t; 0.66\ell + 14\text{mm})$
	100	0.3	t
	100	0	-

[4] Huang, Y.; Zhang, Q.; Bao, Y.; Bu, Y. (2019). *Fatigue assessment of longitudinal rib-to-crossbeam welded joints in orthotropic steel bridge decks*. J Constr Steel Res 159, p. 53-66.

[5] Yokozeki, K.; Tominaga, T.; Miki, C. (2021). *The effects of rib shape and slit on fatigue properties of orthotropic steel decks*. Weld World 65, 601-609.

[6] prTS 1993-1-901 (2023). *Fatigue design of orthotropic bridge decks with the hot spot stress method* (Final draft). Brussels: CEN.

[7] Hobbacher, A.F. *Recommendations for Fatigue Design of Welded Joints and Components* (2nd Ed). Paris: IIW/Springer.

[8] Eriksson, A.; Lignell, M.; Olsson, C.; Spennare, H. (2003) *Weld evaluation using FEM - A guide to fatigue-loaded structures*. Gothenburg: Industrilitteratur AB.

[9] Niemi, E.; Fricke W.; Maddox, S. (2018) *IIW Collection Designer's Guide: Structural Hot-Spot Stress Approach to Fatigue Analysis of Welded Components*. Singapore: Springer Nature.

[10] Rodenburg, J.D.; Hengeveld, S.T.; Wijnbeld, B.M.M.; Maljaars, J. (2023) *Generic method of local thickening in shell element FEM models for orthotropic bridge*. Eurosteel 2023, Amsterdam. Berlin: CE Papers, Ernst & Sohn.

[11] Sonsino, C.M.; Bruder, T.; Baumgartner, J. (2010) *S-N lines for welded thin joints - suggested slopes and fat values for applying the notch stress concept with various reference radii*, Weld World 54, p. R375–R392.

[12] Hensel, J.; Nitschke-Pagel, T.; Dilger, K. (2017) *Engineering model for the quantitative consideration stresses in fatigue design of welded components*. Weld World 61, p. 997–1002.

[13] Maljaars, J.; Bonet, E.; Pijpers, R. J. (2018). *Fatigue resistance of the deck plate in steel orthotropic deck structures*. Eng Fract Mech 201, p. 214-228.

[14] Drebenstedt, K.; Euler, M. (2018) *Statistical analysis of fatigue test data according to Eurocode 3*, Maintenance, Safety, Risk, Management and Life-Cycle Performance of Bridges, Melbourne. Leiden: CRC Press, pp. 2244–2251.

[15] BS 7910 (2019). *Guide to methods for assessing the acceptability of flaws in metallic structures*. London: BSI.

[16] Maljaars, J.; Pijpers, R.J.M.; Wu, W.; Kolstein, M.H. (2023) *Fatigue resistance of rib to deck, crossbeam to deck and deck to deck welds in orthotropic decks using local stress.* Int J Fatigue (available on-line).

[17] Maljaars, J.; Pijpers, R.J.M.; Rodenburg, J. *Derivation of detail categories for modern European bridge decks*. TNO report R10049.

[18] prEN 1991-2 (2021). *Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges and other civil engineering works* (Version for enquiry). Brussels: CEN.

[19] Croce, P. (2021). *Background to fatigue load models for Eurocode 1: part 2 traffic loads*. Prog Struct Eng Mater 3, p. 335–45.

[20] MC Renovatie bruggen (2010) Muiderbrug Phase 3 Design Report. Amsterdam: Arup.

[21] Rodenburg, J.D.; Maljaars, J.; Hengeveld, S.T.; Vervuurt, A.H.J.M. (2023). *Tyre contact surface for the fatigue design of orthotropic steel bridge decks*. Eng Struct 283, 115869.

[22] Maljaars, J.; van Dooren, F.; Kolstein, H. (2012) *Fatigue assessment for deck plates in orthotropic bridge decks*. Steel Constr 5, p. 93-100.

[23] van Dooren, F.; et al. (2010). *Orthotropic deck fatigue: renovation of 8 bridges in The Netherlands*. Structural Faults & Repairs, 12th Int Congress and Exhibition.

[24] Nagy, W.; et al. (2019) *Fatigue assessment of an existing orthotropic steel deck to comply with future traffic intensity*. 20th Congress of IABSE. New York City: IABSE.

[25] prEN 1993-2 (2022). Eurocode 3: Design of steel structures – Part 2: Steel Bridges (Final document). Brussels:CEN