CETRA 2018

5th International Conference on Road and Rail Infrastructure
17–19 May 2018, Zadar, Croatia

Road and Rail Infrastructure V

Stjepan Lakušić – EDITOR

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CETRA[2018]
5th International Conference on Road and Rail Infrastructure
17–19 May 2018, Zadar, Croatia

TITLE
Road and Rail Infrastructure V, Proceedings of the Conference CETRA 2018

EDITED BY
Stjepan Lakušić

ISSN
1848-9850

ISBN
978-953-8168-25-3

DOI
10.5592/CO/CETRA.2018

PUBLISHED BY
Department of Transportation
Faculty of Civil Engineering
University of Zagreb
Kačićeva 26, 10000 Zagreb, Croatia

DESIGN, LAYOUT & COVER PAGE
minimum d.o.o.
Marko Uremović · Matej Korlaet

PRINTED IN ZAGREB, CROATIA BY
“Tiskara Zelina”, May 2018

COPIES
500

Zagreb, May 2018.

Proceedings of the
5th International Conference on Road and Rail Infrastructures – CETRA 2018
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EDITOR
Stjepan Lakušić
Department of Transportation
Faculty of Civil Engineering
University of Zagreb
Zagreb, Croatia

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FATIGUE ASSESSMENT DUE TO NON-STANDARD DETAILING OF ORTHOTROPIC BRIDGE DECKS

Andjelko Vlasic, Nijaz Mujkanovic, Gordana Hrelja Kovačević
University of Zagreb, Faculty of Civil Engineering Zagreb, Croatia

Abstract

Modern norms define design guidelines for designers and contractors regarding acceptable detailing and manufacturing of bridge orthotropic deck plates. The convenience of these design guidelines is that for most deck elements, a specified detail can be accepted without additional fatigue verification. Consequently, these details were used in the design documentation of the Mainland – Čiovo Island Bridge in Trogir. But, in one of the steel workshops, during orthotropic deck manufacturing, considerable discrepancies from design detailing were noticed. These discrepancies comprise additional spacer plates placed between the rib and the deck plate to simplify the welding process, which largely influences fatigue resistance. To approve these changes and to allow for the finished orthotropic deck to be used in the bridge superstructure, additional fatigue assessment was performed. This paper shows the numerical model used for attaining stress ranges in the rib to deck welds, and the fatigue assessment according to HRN EN 1993-1-9. Ultimately, fatigue verification passed, and manufactured deck was approved. But, nevertheless the detailing used in the manufacturing process was abandoned for all further deck segments, and it is not to be recommended for use.

Keywords: orthotropic bridge deck, fatigue, detailing, welding, numerical model

1 Bridge design

1.1 Mainland – Čiovo island bridge in Trogir orthotropic deck superstructure

The bridge in question, for which the fatigue assessment was done, is Mainland – Čiovo island bridge in Trogir, currently being erected. Bridge superstructure is a continuous girder with total length of 521.58 m spanning over 14 spans: 20.58+28.0+32.0+5*40.0+34.8+41.2+34.8+2*40.0+32.0 m. The bridge cross-section is a three-cell steel (S355J2+N) box with vertical webs and curved intrados (Figure 1). The depth is constant and amounts to maximum 1,682 mm in the longitudinal bridge axis. The bottom flange between inner webs is concaved, then straight to outer webs, and it meets convex circular cantilevered cross beams with the common tangent line.
The orthotropic deck plate t = 14 mm follows the transverse slope of the roadway. It is longitudinally stiffened by 250 mm deep closed trough type stiffeners t = 8 mm, spaced at 600 mm axis to axis and supporting the deck plate at 300 mm. The elevated footway plates t = 12 mm are longitudinally stiffened by open stiffeners h/t = 200/16 mm. Webs are vertical and support the deck plate at 3,800 mm (inner webs) and 7,080 mm (outer webs) cross spacing. The thickness of inner webs is t = 20 mm and of outer webs t = 12 mm. The thickness of the concave circular bottom plate between inner webs is variable from minimum t = 12 mm in spans to maximum t = 30 mm at supports, stiffened by closed trough type stiffeners h/t = 220/8 mm. The remaining parts of the bottom plate have variable thickness t = 18–30

DOI: https://doi.org/10.5592/CO/CETRA.2018.960

mm. Bottom flanges of cantilevered cross beams are ≠ 300*20 mm, and webs of variable depth 250–680 mm are 12 mm thick. Diaphragms are spaced at 4.0 m, equipped with holes Ø 650 mm to enable passage inside the box beam. Their thickness amounts to t = 12 mm in spans and t = 20 mm above supports [1].

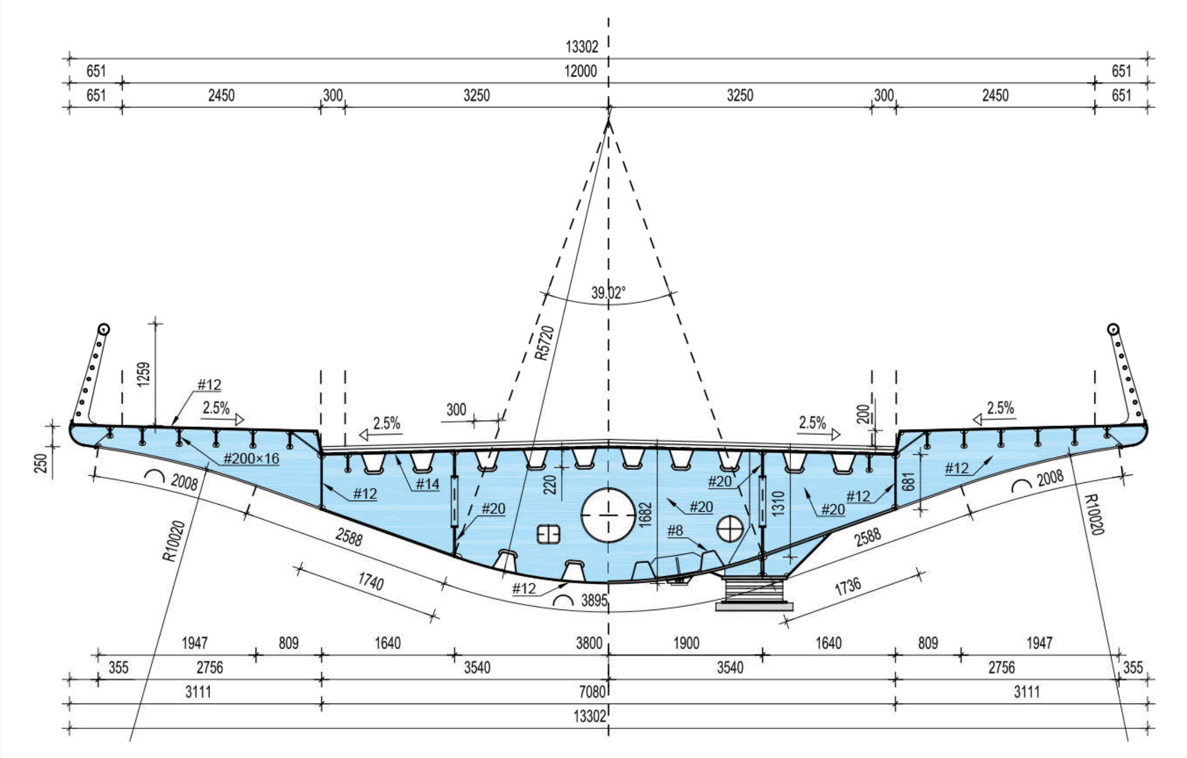


Figure 1 Mainland – Čiovo island bridge in Trogir cross section [1]

1.2 Design code for fatigue resistance of bridge orthotropic deck

Examining governing design code for bridge design, and for the aforementioned bridge this code is Eurocode, following should be emphasized [2]:

1) Bridges should be designed for fatigue for the duration of their working life.
2) Principles of the limit state design (Chapter 2.2 (4) [2]) state that "the required fatigue life should be achieved through design for fatigue and/or appropriate detailing" given in Annex C [2].
3) Chapter 4 (5) [2]: Required fatigue life for orthotropic steel decks in bridges can be attained by structural detailing.
4) Chapter 9.1.2 (1) [2] – Design of road bridges for fatigue: "Fatigue assessment should be carried out for all bridge components unless the structural detailing complies with standard requirements for durable structures established through testing."
5) Annex C.1 gives recommendations for structural detailing of highway bridge orthotropic decks. The recommendations relate to the type of the stiffener, minimum thickness of the deck plate and stiffener, the splices of the deck plate, and the connection between the deck plate and webs of the girder, stiffener and crossbeams. Minimum stiffens of stiffeners should be selected in accordance with the traffic category and distance between crossbeams. Weld between the closed section stiffener and the deck plate should be butt weld. Thickness of the weld is regulated in Table C.4 [2]. This table gives guidelines for fabrication of each structural detail of closed stiffener orthotropic deck plate. These guidelines give us a standardization level for all road bridge orthotropic decks.

According to the Table C.4 [2], stiffener to deck plate connection should be done with recommendations shown in Figure 2. If these standards are abided, no additional fatigue verification is needed. The Main design of Mainland – Čiovo island bridge in Trogir was done according to those standards, and thus, no fatigue verification was needed or done for orthotropic deck.

4) Stiffener-deck plate connection (manual and partially mechanized welding process), weld preparation angle α in dependence of the welding process and accessibility	independent on stress level in deck plate	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1 Tolerances for weld preparations to be met ad 1b Requirement 1	Starts and stops to be removed This requirement also applied to local welds, e.g. at stiffener-stiffener connections with splice plates, see 16).

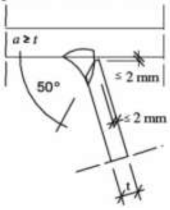


Figure 2 Recommendations concerning stiffener to deck plate weld from EN 1993-2 [2]

2 Workshop manufacturing defects

During the workshop manufacturing of the superstructure segments (Figure 8), serious deviations from the design standard detailing were recorded. Using his own discretion, one of the Sub-Contractors used additional steel plates as spacers to assist him during welding of longitudinal ribs to deck plate (Figure 3). These plates were welded to ribs prior to longitudinal rib to plate welding, thus disrupting the continuity and quality of this weld. This manufacturing detailing could no longer be accepted as design code recommended regarding attained fatigue resistance.
Due to quantity of orthotropic deck segments already produced in this fashion, the Client requested additional fatigue verification to be made if the existing segments are to be accepted and used in bridge superstructure. Otherwise, they are to be redone. Requested fatigue verification is to be done for the connection weld between the stiffener web and the deck plate.

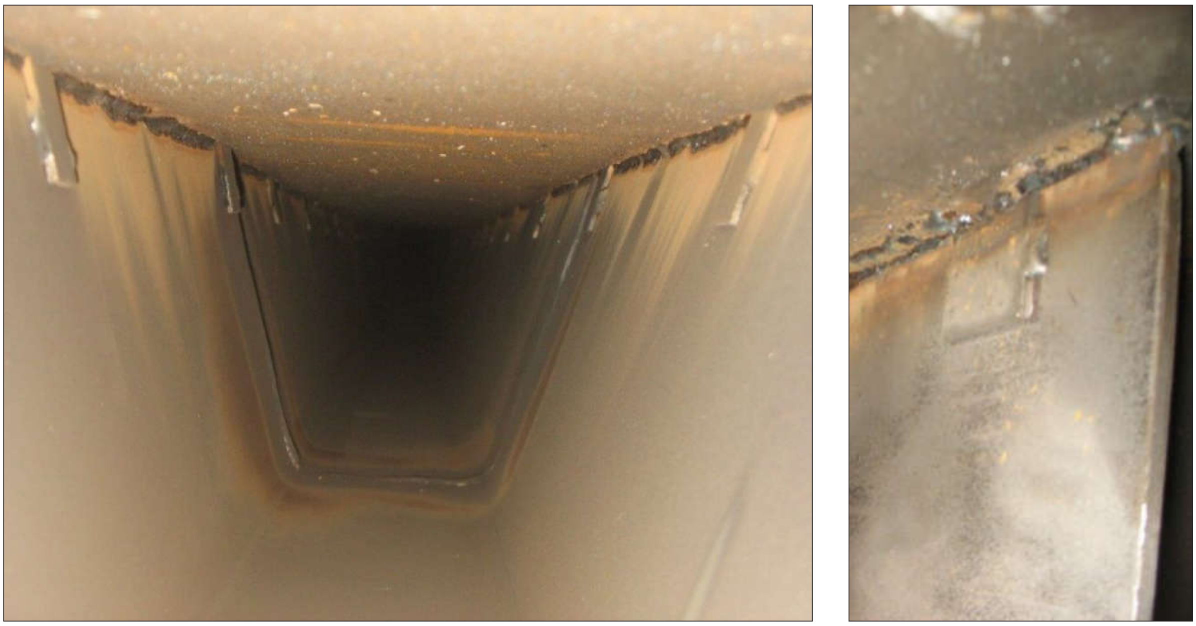


Figure 3 Steel plate spacers added during manufacturing

3 Finite element model for fatigue stress calculation

For the purpose of fatigue assessment, a complex 3D finite element model was made [3]. In order to calculate stresses in the rib to plate details of orthotropic plate, 2D plate elements were used. The model depicted first bridge dilatation of whole nine spans. However, only one part of this dilatation was modelled using 2D plate elements due to reduction of model size and computation time savings. This detailed 2D elements model part comprised part of the one span, its adjacent bearing area, and part of the other span (Figure 4).

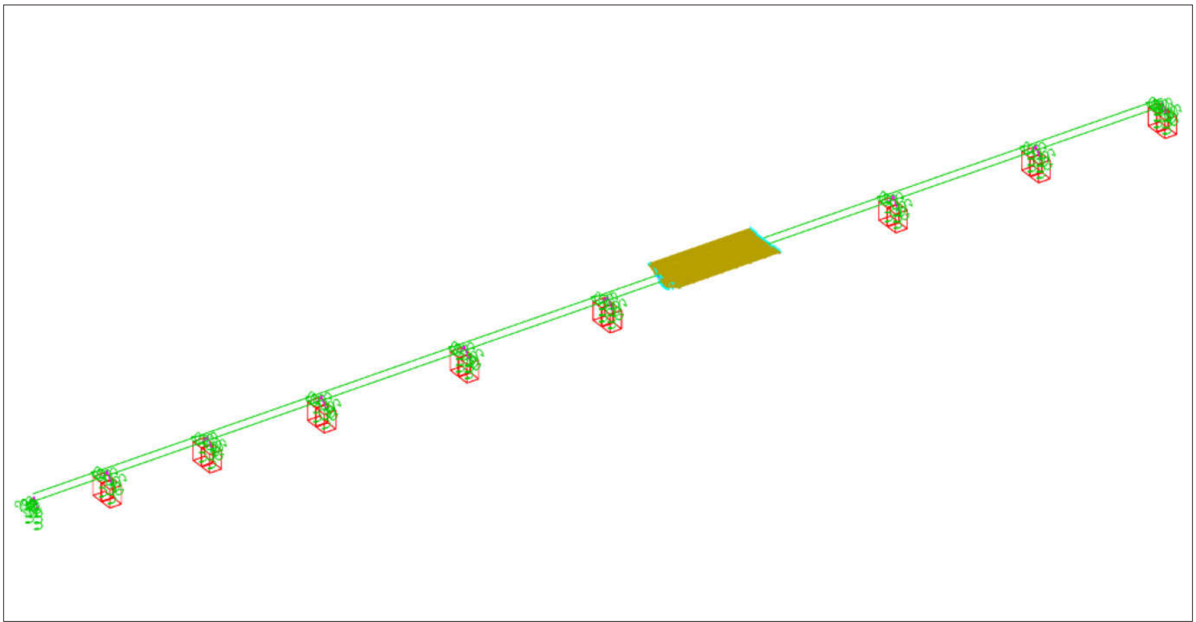


Figure 4 Mixed 1D (beam) and 2D (plate) elements model of first dilatation

Both positive and negative girder bending moments could thus be analysed. Plate elements were used to model individually all section elements (Figure 5) – deck plate, ribs, cross girders, diaphragms, stiffeners, flanges, openings with stiffening rings, bearing plates... Total length of this detailed model part is 28 m (seven 4 m orthotropic deck plate spans).

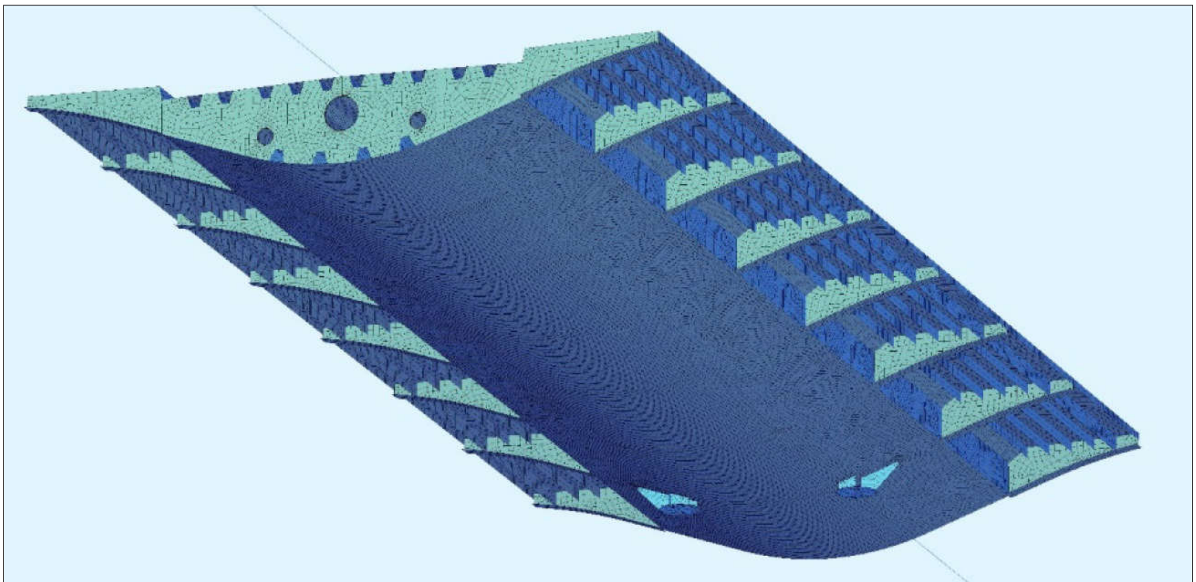


Figure 5 2D (plate) element part of the bridge superstructure

Bearing was modelled as elastic bedding under the bearing plate with stiffness according to elastomer thickness calculation (vertically 6.05×10^6 kN/m^3, horizontally 7.4×10^3 kN/m^3). Rest of the bridge dilatation was modelled using beam elements with cross sections comprising entire box girder. Beam and plate parts of the model were joined together with fixed coupling connections – all the nodes of plate elements in the transition section were coupled with the end node of the beam section. This way, a continuity of the superstructure for static analysis was achieved. Thus, the loads can be applied on the complete static system and the correct edge conditions are met for the detailed plate elements part of the model. Such a model allows analysis of the global effects (box section girder bending), and local effects (bending of orthotropic deck plate) simultaneously. Loads are modelled according to fatigue vehicle Load Model from EN 1991-2 (Figure 6) [4]. Load area for each wheel is defined as $60/(0.58^2) = 178.4$ kN/m^2. Position of load train was varied as traveling through two spans, with two possible lanes – one in the middle of the cross section, and other on the edge of the roadway.

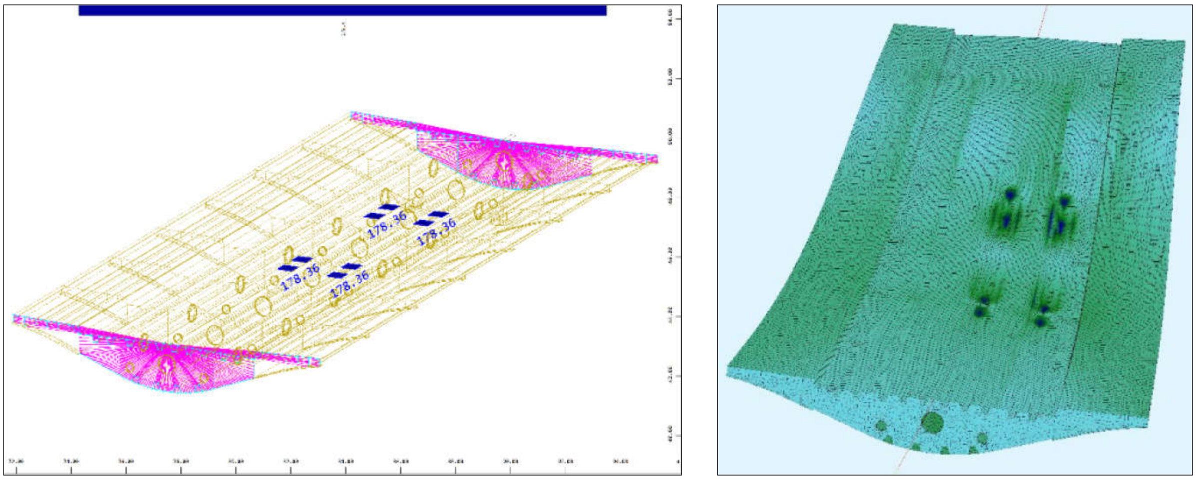
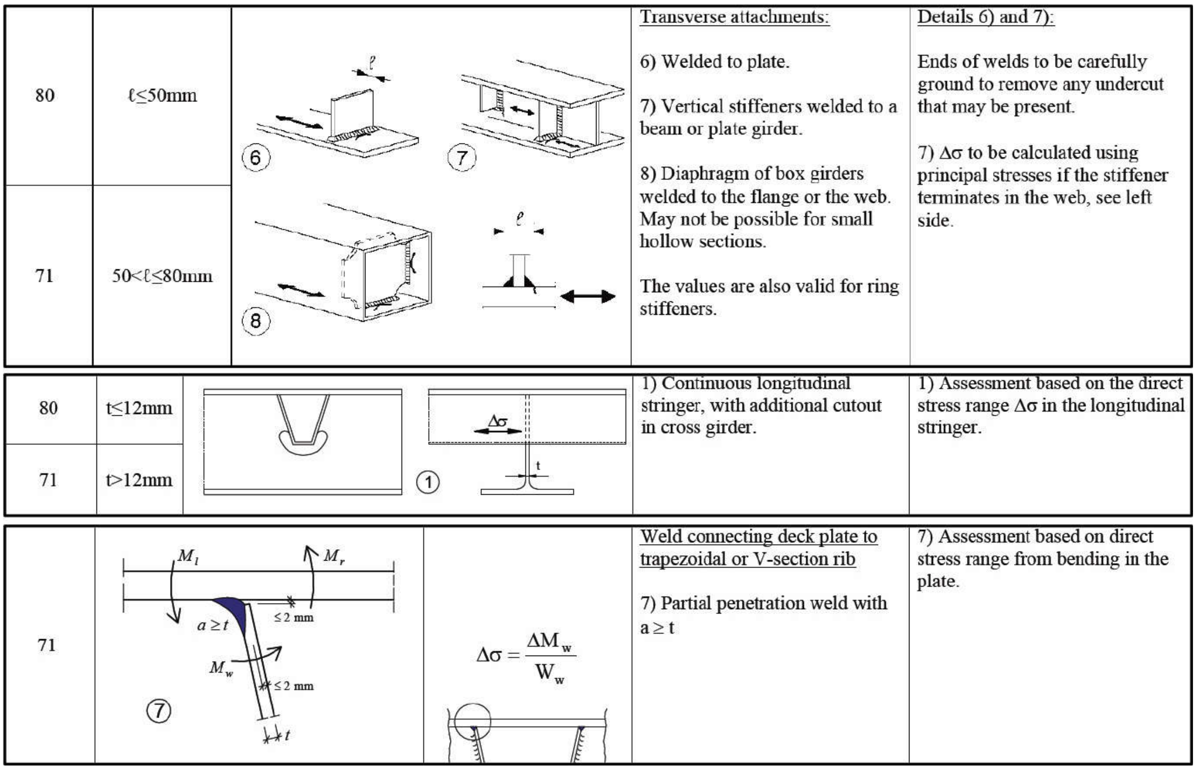


Figure 6 Load and deformation from fatigue load model

4 Fatigue assessment

Fatigue assessment was done for the critical detail described previously, which is the weld connection of the stiffener to deck plate, on the inner edge of the stiffener where the spacer plates are welded (Figure 3 right). Governing stress for fatigue is taken from middle of stiffener web thickness, on the top of the stiffener web, where it connects to deck plate. Recorded stresses are von Mises stresses, which depict von Mises yield energy for converting complex stress state into a single one-dimensional stress state comparable to uniaxial yield strength. Stress is taken from nodal values of finite element model, which are larger than element values. Envelope of positions of the load train yield maximum tension and compression stress in the mid span of the orthotropic plate between diaphragms which are spaced at 4 m, and just above the diaphragm. Differentials ($\Delta\sigma$ – Table 1) between maximum compression and tension for both mid span of orthotropic plate and above the diaphragm are calculated for fatigue verification. Details of the welds in the critical weld between stiffener and the plate are chosen as category 71 according to more unfavourable constructional detail from table 8.4 and 8.8 given by EN 1993-1-9 (Figure 7) [5]. These constructional details were chosen due to following criteria:

a) welded connections of spacer plates to webs of stiffeners were not done in controlled environment,
b) mandatory execution class of construction is EXC4,
c) welded connection can best be described by detail 7 from table 8.4, or detail 1 from table 8.8, given in EN 1993-1-9 [5].



80	ℓ≤50mm		Transverse attachments: 6) Welded to plate. 7) Vertical stiffeners welded to a beam or plate girder. 8) Diaphragm of box girders welded to the flange or the web. May not be possible for small hollow sections. The values are also valid for ring stiffeners.	Details 6) and 7): Ends of welds to be carefully ground to remove any undercut that may be present. 7) Δσ to be calculated using principal stresses if the stiffener terminates in the web, see left side.
71	50<ℓ≤80mm			
80	t≤12mm		1) Continuous longitudinal stringer, with additional cutout in cross girder.	1) Assessment based on the direct stress range Δσ in the longitudinal stringer.
71	t>12mm			
71		$\Delta\sigma = \frac{\Delta M_w}{W_w}$	Weld connecting deck plate to trapezoidal or V-section rib 7) Partial penetration weld with a ≥ t	7) Assessment based on direct stress range from bending in the plate.

Figure 7 Recommendations regarding detail category from EN 1993-1-9 [5]

Table 1 Calculation of design value of nominal stress range and fatigue verification

	Mid span	Above diaphragm
$\Delta\sigma$ [MPa]	22	34
$\Delta\sigma_c$ [MPa]	71	71
λ_1	$2.55-0.7\frac{L-10}{70}=2.61$ for L = 4 m	$2.0-0.3\frac{L-10}{20}=2.09$ for L = 4 m
λ_2	1.1	1.1
$\lambda_3=\left(\frac{t_{Ld}}{100}\right)^{\frac{1}{5}}$	1.0 for $t_{Ld}=100$	1.0 for $t_{Ld}=100$
$\lambda_4=\left(1+(k-1)\cdot 0.1\right)^{\frac{1}{5}}$	1.02 for k = 2	1.02 for k = 2
$\lambda=\lambda_1\lambda_2\lambda_3\lambda_4$	2.93	2.35
λ_{max}	$2.5-0.5\frac{L-10}{715}=2.7$ for L = 4 m	1.8
$\Delta\sigma_{E,2}=\lambda\cdot\Delta\sigma$ [MPa]	59.4	61.2
$\gamma_{Ff}\cdot\Delta\sigma_{E,2}\leq\frac{\Delta\sigma_c}{\gamma_{Mf}}$ $\gamma_{Ff}=1.0;\ \gamma_{Mf}=1.15$	**59.4 < 61.7**	**61.2 < 61.7**

Fatigue verification is performed by satisfying condition 6.3 from EN 1993-1-9 [5]:

$$\gamma_{Ff} \cdot \Delta\sigma_{E,2} \leq \frac{\Delta\sigma_c}{\gamma_{Mf}} \tag{1}$$

Calculation [3] is shown in Table 1. According to calculation fatigue verification passes with little to no reserve, showing that the constructional detail is indeed critical regarding fatigue resistance. This is the reason why design recommendations from the code should always be followed. Execution of orthotropic deck box girder superstructure segments in the workshop is shown on Figure 8.

Figure 8 Execution of orthotropic deck box girder superstructure segments in the workshop

5 Conclusion

Design guidelines for orthotropic decks of road bridges are recommended for execution of each deck element. If these recommendations are met, no additional fatigue assessment is needed. Design recommendations are unclear regarding the reason why fatigue check can be omitted. Question remains whether fatigue verification will always be met in such a case, or even if it is not met, orthotropic decks executed with recommended details should be assumed to possess enough fatigue resistance to be accepted as such. This unanswered question was the main reason behind the need to perform fatigue assessment for the orthotropic deck which was executed with non-standard detail of deck to stiffener weld, such as presented in this paper. This assessment comprised decision for the construction detail category, calculation of stresses on a complex and time consuming finite element model for a critical stress point in the weld area, and fatigue verification according to code. It has been concluded that, for this particular case, fatigue verification is met despite the non-standard execution, and such executed deck segments can be used in the bridge superstructure. However, verification showed low fatigue resistance reserve, which is why no deviations from code standards during execution of details are recommended in further practice.

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