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Recommended properties of elastic wearing surfaces on orthotrotropic steel decks

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Abstract. Orthotropic decks composed of deck plate, ribs, cross beams and wearing surface are frequently used in industry to span long distances due to their light structures and load carrying capacities. As a result they are broadly preferred in industry and there are a lot of bridges of this type exist in the world. Nevertheless, some of them cannot sustain the anticipated service life and damages in form of cracks develop in steel components and wearing surface. Main reason to these damages is seen as the repetitive wheel loads, namely the fatigue loading. Solutions to this problem could be divided into two categories: qualitative and quantitative. Qualitative solutions may be new design methodologies or innovative materials, whereas quantitative solution should be arranging dimensions of deck structure in order to resist wheel loads till the end of service life. Wearing surface on deck plate plays a very important role to avoid or mitigate these damages, since it disperses the load coming on deck structure and increases the bending stiffness of deck plate by forming a composite structure together with it. In this study the effect of Elastic moduli, Poisson ratio and thickness of wearing surface on the stresses emerged in steel deck and wearing surface itself is investigated using a FE-model developed to analyze orthotropic steel bridges.

Keywords: steel bridge; orthotropic deck; wearing surface; bonding layer; FEM

1. Introduction

Construction of orthotropic decks with deck plate, cross-beams and ribs going through the cut-outs in cross beam webs is widely used in industry (Jong 2007). Orthotropic deck structure is a common design, which is used worldwide in fixed, movable, suspension, cable-stayed, girder, etc. bridge types. In Japan, Akashi Kaikyo suspension bridge, Tatara cable stayed bridge (Honshu Shikoku Bridge Authority 2005), Trans-Tokyo Bay Crossing steel box-girder bridge (Fujino and Yoshida 2002), which are among the longest bridges in the world, have orthotropic deck structure. In France Millau viaduct has a box girder with an orthotropic deck with trapezoidal stiffeners (Virlogeux 2004). In England, Germany and Netherlands there are a lot of steel highway bridges having orthotropic decks (Jong 2007). In USA San Francisco Oakland Bay Bridge, Self Anchored Suspension Span in California and in Italy Strait of Messina Bridge are examples of orthotropic steel bridges. In Turkey, the Golden Horn Bridge, First Bosphorus Bridge and Fatih Sultan Mehmet Bridge are also examples of orthotropic steel bridges (Kennedy *et al*. 2002). In Troitsky

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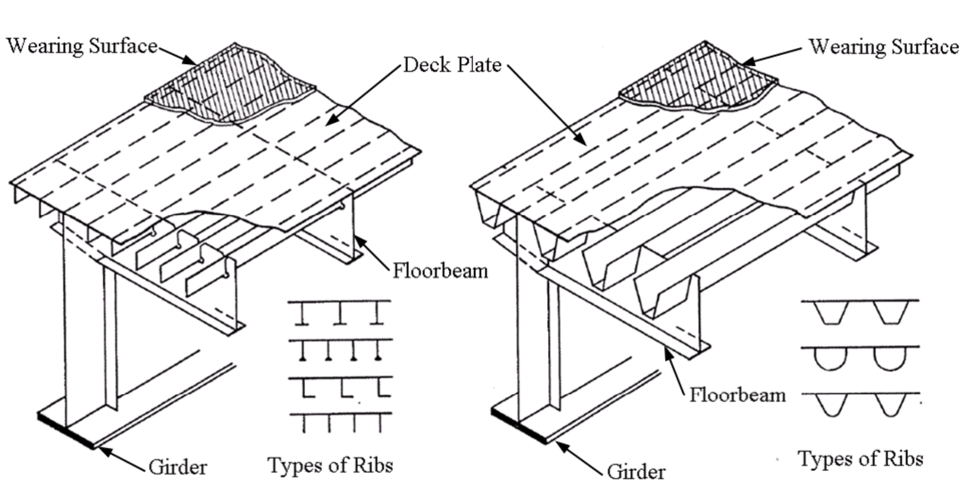
(1987), Huang and Mangus (2008), Hoorpah (2004), Korniyiv (2004), Choi et al. (2008), Connor *et al*. (2012), Kozy et al. (2011) and Wolchuk (2014) examples of bridges, in which orthotropic deck is used, are given in detail. The spacings of longitudinal stringer and cross beam are in general 300 mm and 3 m to 5 m respectively. In addition to orthotropic deck structure, wearing surface lying on deck plate and main girders transmitting load to supports are two important components of orthotropic bridges. While wearing surface might be of asphalt or concrete, main girder might be of a girder, a truss, a cable stayed or a tied arch system.

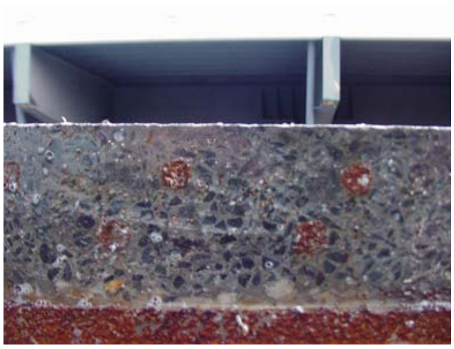
Orthotropic decks resist against corrosion by means of traditional anti-corrosive paintings used in industry. The top of the orthotropic deck is covered by wearing course and individual ribs are sealed with end plates to prevent moisture from entering the interior of the rib (Connor *et al*. 2012). In addition to protection measures taken against to corrosion, repair methods of corroded steel are also interest of research (Aoki *et al*. 2013). Deck plate forms the flanges of ribs, cross-beams and main girders, hence leads an integral behavior of whole orthotropic deck and results in fewer material use (Luo *et al*. 2010). The closed ribs became dominant on open ribs in industry, because they have much more torsional, buckling rigidities, distribute wheel loads much better on deck plate, require half amount of welding than open ribs, provide less steel material needed in bridge orthotropic deck and so lighter dead load, which makes them also cost effective against orthotropic decks of open ribs. As a result they have become inevitable part of orthotropic decks to span long distances. In Fig. 1 types of closed ribs are given as trapezoidal, U-shaped and V-shaped forms, in which trapezoidal ribs became paramount in time. Experienced cracks in orthotropic bridges revealed that the design of orthotropic decks shall be performed with respect to fatigue analysis because of repetitive wheel loads varying in type and magnitude (Sim and Uang 2012, Yamada and Ishikawa 2011, Han *et al*. 2013, Aygul *et al*. 2012). As a result, the fatigue strengths of orthotropic deck details are provided by engineering standard, Eurocode 3, Part 1-9 (2003). To calculate stresses developed under wheel loads the solution method chosen shall enclose the entire bridge geometry, which can be achieved today using FEM instead of conventional analytical and numerical methods used in the history in the absence of FEM. In addition to the correct analysis and accordingly design of orthotropic decks, their fabrication, shipping to construction area and



(a) Deck with open ribs (b) Deck with closed ribs

Fig. 1 Orthotropic deck with open and closed ribs (AISC 1963, Connor *et al*. 2012) workmanship shall be done flawlessly and with care to obtain the desired service life. For that reason all steps until and during the construction of orthotropic decks require necessary quality control measures so as to provide the required service life. Because of their higher initial costs, if orthotropic steel decks are produced under permanent surveillance of quality control measures, they can supply a 100-year service life, which is demonstrated by laboratory studies (Connor *et al*. 2012). Wearing surface on deck plate plays a very important role to avoid or mitigate the damages of orthotropic deck, since it disperses the load coming from vehicle wheel on deck structure and increases the bending stiffness of deck plate by forming a composite structure together with it. Pouget *at al*. (2012) presented a very good article about viscous bituminous wearing surfaces on orthotropic decks. Buitelaar *et al*. (2004) performed researches regarding reinforced high performance concrete (RHPC) as elastic wearing surface on orthotropic decks. They recommend RHPC for renovation of existing damaged wearing surface and construction of new wearing surface as well. They modeled RHPC with linear elastic material properties using their FE-model to estimate stress reduction factors due to RHPC. The elastic module of ultra high performance concrete (UHPC) can vary even between 60 and 100 GPa. The RHPC overlay is a combination of a HPC strength class C110 (based on special pre-blended materials reinforced with both steel and acrylic fibers) and welded mesh reinforcement (consisting of two specially produced mats Ø8 mm

(a)



(b)

Fig. 2 (a) Reinforcement principle; and (b) RHPC overlay on deck plate (Buitelaar *et al*. 2004)

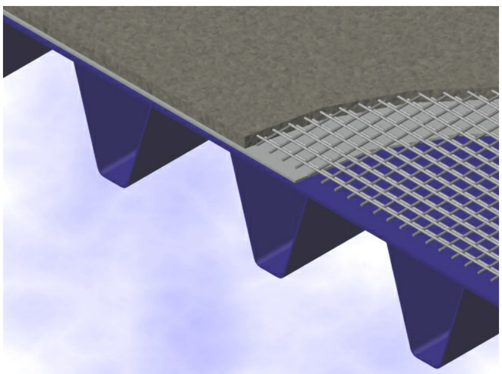


Fig. 3 RHPC surfacing system (Jong 2007)

50 × 100 mm positioned on top of each other such that a total of three layers of Ø8 mm rebars spaced at 50 mm is obtained) (Buitelaar 2002). The mesh reinforcement is placed on a Ø8 mm rebar used as a spacer. Thus, the total amount of reinforcement is approximately 24 kg/m^2 traditional reinforcement plus 5 kg/m^2 steel fibers. The total thickness of the RHPC overlay is in this specific case 50 mm. The concrete cover on the reinforcement is thus only 18 mm. If the thickness of the layer should be increased, the reinforcement can be adjusted. To replace the existing wearing course with a RHPC overlay, the bonding between the steel deck plate (thickness 10-12 mm) and the overlay is of crucial importance to secure total deck rigidity and a uniform "monolithic" behavior under all circumstances. For that reason the first investigations were focused on creating a bonding zone that met all requirements. A bonding zone can easily be created by connecting the mesh reinforcement and the steel deck plate by welds, but this might result in undesirable local peak stresses. Therefore research was carried out to find the optimal bonding agent. The best method turned out to be the use of a two-component epoxy based adhesive with sprinkled-in bauxite aggregates. After hardening of the epoxy, the overlay is cast. The surface will be shot blasted. No additional wearing course will be applied (Buitelaar *et al.* 2004).

Jong (2007) also recommends the same principles and conclusions on the modeling and application of RHPC in his dissertation. The material properties of the linear-elastic modeled wearing surface and bonding layer beneath it are given below:

- Material properties of the RHPC surfacing are: Elastic module E = 50,000 MPa (Braam and Mulder 2002, Buitelaar *et al.* 2004), Poisson ratio v = 0.3, density ρ = 2,400 kg/m^3, thermal expansion coefficient $\alpha = 12 \times 10-6$. The concrete is modeled as a homogeneous layer. The reinforcement bars and cracks in the tension zones of the concrete are not modeled.
- Material properties of the bonding layer between steel and RHPC surfacing are from the product data sheet of Sikadur 30 (Sika 2006): Elastic module E = 12,800 MPa, Poisson ratio v = 0.3, density ρ = 1650 kg/m^3, thermal coefficient $\alpha = 90 \times 10-6$. Research has shown that Sikadur 30 is a good epoxy bonding layer for RHPC on steel (Poulis *et al.* 2000). Therefore in the FE-models the properties of Sikadur 30 were used (Jong 2007).

As a loading in FE-model of Jong (2007) a wheel footprint is used, which is 320 mm long, 220 mm wide, and has a load of 35 kN. This is in fact wheel type A, the so-called single, from Eurocode 1 Part 2 (2003). The standard axle load of 70 kN for the steering axle with wheel A is divided by two.

However, in this study elastic module of wearing surface is chosen varying between 20 and 35 GPa, which are identical to normal concrete wearing surfaces. The range of Poisson ratio is chosen varying between 0.15 to 0.30 (Erdoğan and Erdoğan 2006). Because aim of this study is determination the influence of material properties and thickness of normal concrete wearing surfaces on steel decks high elastic module values possessed by RHPC is not used in this study. In the first stage, bonding layer is ignored and a normal concrete wearing surface is considered, which is fully bonded to the top of deck plate. Then stresses developed in orthotropic deck are assessed for different elastic module and Poisson ratio pairs of normal concrete wearing surface. In the second stage, bonding layer is modeled as described by Jong (2007) and the results are evaluated depending upon variance of bonding layer's shear module and wearing surface's thickness. It is of great importance to note that, wearing surface term used in this study refers to concrete wearing surface plus bonding layer (if considered in FE-model as in the case of second stage). In the first stage of FE-analyses, wearing surface term refers solely to concrete wearing surface, since concrete wearing surface is fully bonded to deck plate without a bonding layer between them. Thus, bonding layer is only involved in the second stage of FE-models.

In the scope of this study the analysis based on conventional techniques of orthotropic bridges are summarized in the next section. Afterwards, FEM applied in this study is introduced in Section 3.

In Section 4 the influence of elastic moduli, Poisson ratio and thickness of elastic wearing surface as per stress distribution in orthotropic deck is handled. Consequently, conclusions and recommendations regarding material properties and thickness of wearing surface are given in the last section.

2. Analysis of orthotropic deck using conventional methods

Orthotropic steel decks of bridges are subject to fluctuating wheel loads of different magnitudes. Wheel loads are first dispersed by wearing course and introduced in deck plate. Subsequently, longitudinal stringers transmit wheel loads to cross beams. Finally wheel loads are transferred

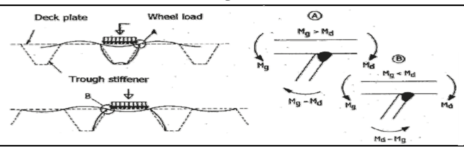
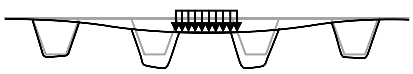
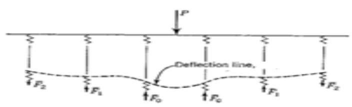
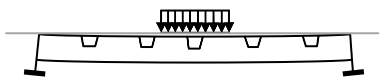
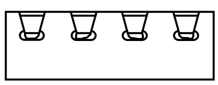
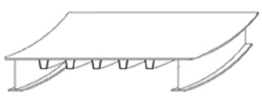
System	Action	Figure	Result
1	Local deck plate deformation		Transverse flexural stress in deck plate and rib.
2	Panel deformation		Transverse deck stress from rib differential displacements
3	Rib longitudinal flexure		Longitudinal flexure and shear in rib acting as a continuous beam on flexible floorbeam supports
4	Cross-beam in-plane flexure		Flexure and shear in cross-beam acting as beam spanning between rigid girders
5	Cross-beam distortion		Out-of-plane flexure of cross-beam web at rib due to rib rotation
6	Rib distortion		Local flexure of rib wall due to cross-beam cut-out
7	Global		Axial, flexural, and shear stresses from supporting girder deformations

Fig. 4 Decomposing of orthotropic bridge deck into subsystems (Connor *et al*. 2012)

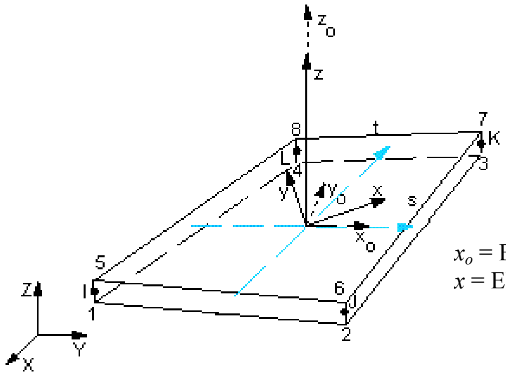
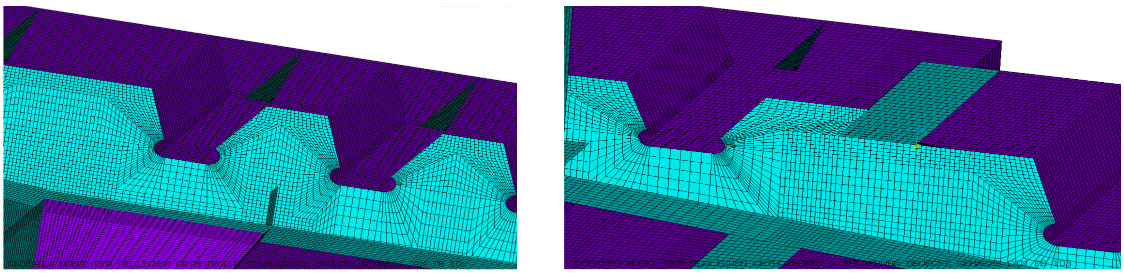


Fig. 5 Shell 181 finite element, which is used in this study (ANSYS 2010)



(a) (b)

Fig. 6 (a) Connection of cross-beam to main girder; (b) connection of deck plate to pedestrian road

from cross beams over main girders to the supports. Although an orthotropic deck forms an integrated structure to resist against wheel loads, the assumed load transmitting scheme is generally accepted as given in Fig. 4.

3. Analysis of orthotropic deck using FEM

So as to compare stresses developed for different structural thicknesses, spacings and spans, all dimensions of the bridge shall be defined as variables in ANSYS (2010). Therefore an algorithm to provide this condition is written by means of APDL (Ansys Parametric Design Language). Afterwards thicknesses, spacings and spans of structural parts, which are of interest, are entered in ANSYS using this algorithm. Stresses developed for different parameter values are given in the subsequent sections. The FE-model of the bridge is generated using SHELL 181, which is illustrated in Fig. 5.

The FE model of Huurman *et al*. (2002) inspired the researchers to create FE-model of the bridge used in this research (Fettahoglu 2012, Fettahoglu and Bekiroglu 2012, Fettahoglu 2013a, b, c, 2014a, b). However, in the FE-model, which is generated using ANSYS (2010) and used in this study stiffened main girder and pedestrian road are also generated, which are not included in the FE-model of Huurman *et al*. (2002) (See Fig. 6). Because of mesh refinement process the number of nodal unknowns increase excessively and as a result spans of the bridge used in this research are chosen as short as possible. Fig. 7 depicts the perspective front view of the whole orthotropic steel bridge, while Fig. 8 shows the wheel loads and their arrangement on the entire bridge geometry.

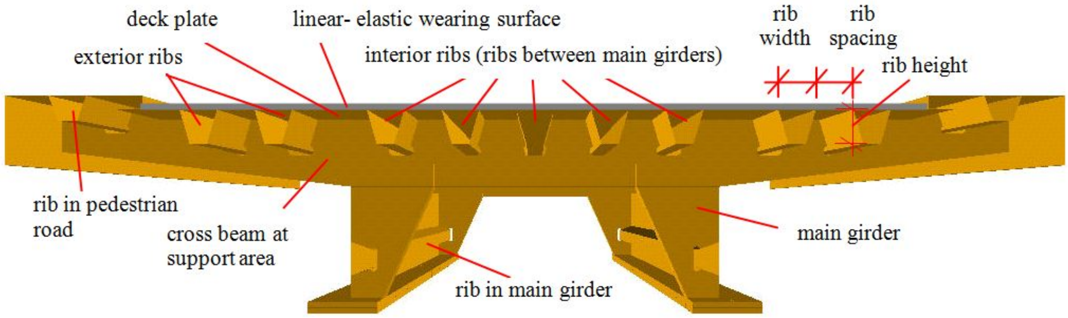


Fig. 7 Traditional orthotropic steel bridge as to DIN FB 103 (2003)

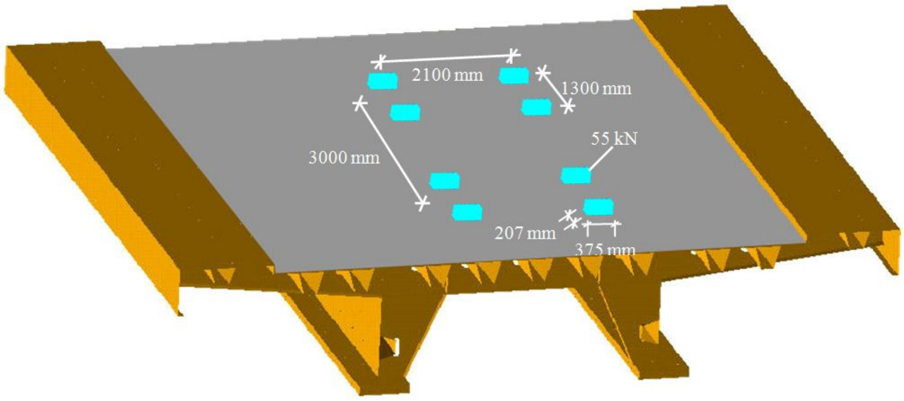


Fig. 8 Wheel loads used in the FE-analyses

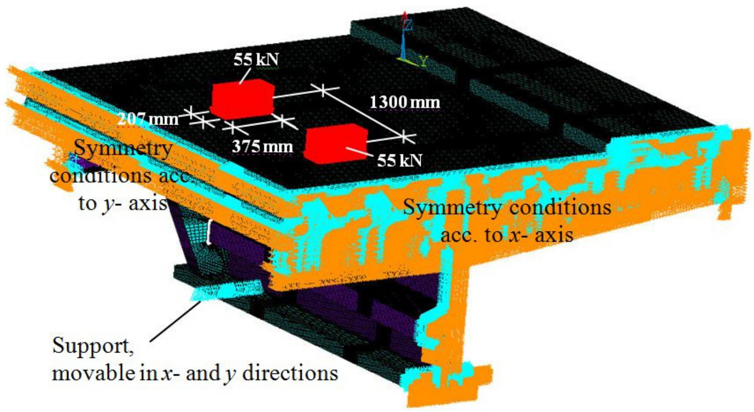


Fig. 9 FE-model and boundary conditions of bridge quarter

Table 1 Material properties

Yield strength of steel (f_y)	355 N/mm^2	Shear module (G)	81,000 N/mm^2
Ultimate strength (f_u)	510 N/mm^2	Poisson ratio (υ)	0.3
Elasticity module (E)	210,000 N/mm^2	Density (ρ_{steel})	78.5 kN/m^3

Table 2 Variation of elasticity module and Poisson ratio as per first stage FE-analyses

FE-analysis name	Elasticity module (MPa)	Poisson ratio	FE-analysis name	Elasticity module (MPa)	Poisson ratio
T4E20P15NL*		0.15	T4E30P15		0.15
T4E20P15		0.15	T4E30P20	30,000	0.20
T4E20P20	20,000	0.20	T4E30P25		0.25
T4E20P25		0.25	T4E30P30		0.30
T4E20P30		0.30	T4E35P15		0.15
T4E25P15		0.15	T4E35P20	35,000	0.20
T4E25P20	25,000	0.20	T4E35P25		0.25
T4E25P25		0.25	T4E35P30		0.30
T4E25P30		0.30			

* This FE-analysis is same as T4E20P15, however geometrical non-liearity is considered to assess its influence on the results. All materials used in Fe-analyses obey Hooke’s Law

Table 3 Variation of bonding layer’s shear module and thickness of wearing surface as per second stage FE-analysis

FE-analysis name	Shear module of bonding layer (MPa)	Thickness of wearing surface (mm)	FE-analysis name	Shear module of bonding layer (MPa)	Thickness of wearing surface (mm)
T25MS5	5		T25MS4923		25
T25MS500	500	25	T50MS4923	4,923	50
T25MS50000	50,000		T75MS4923		75

To decrease further the number of nodal unknowns only the quarter of the bridge shown in Fig. 9 is modeled by applying the necessary boundary conditions. As a result, number of elements and nodes in the FE-model of the bridge are 1 469 148 and 1 535 833 respectively, when rib width, height and spacing are 300 mm, 275 mm and 300 mm respectively. Width of pedestrian road and deck plate in transverse direction are 1.1 m and 6.3 m respectively.

The bridge analyzed in this study spans 6 m in longitudinal direction and has stiffened main girders at supports, normal main girders at field (outside support areas), 2 exterior ribs at each side, 5 interior ribs in the middle, 1 rib in each main girder and 1 rib in each pedestrian road.

According to Capital 3.2 of Eurocode 3 Part 1-1 (2001) material properties of the selected steel material (S 355H) are given in Table 1.

The variation of elasticity module and Poisson ratio of concrete wearing surface in the first

Table 4 Displacement and stresses developed in the orthotropic deck

Type of structure	max. disp. vector sum (mm)	T4E20P15NL		T4E20P15		T4E20P15^{nodal}	
Whole structure	U_{sum}	1.159		1.160		1.160	
Type of stress in deck*		max. tens.	max. comp.	max. tens.	max. comp.	max. tens.	max. comp.
Concrete surface	sx	4.563	5.962	4.561	5.969	4.532	5.955
	sy	4.250	4.180	4.247	4.185	4.246	4.180
	sz	2.234	5.879	2.238	5.875	2.184	5.805
	sv	9.720		9.712		8.642	
Deck plate	sx	41.023	97.482	41.117	97.452	29.223	64.778
	sy	24.626	32.534	24.613	32.502	24.363	16.322
	sz	18.057	45.522	18.067	45.597	15.455	39.801
	sv	157.492		157.391		110.759	
Cross-beam at field	sx	97.039	65.071	97.178	64.999	95.106	63.985
	sy	~ 0.000					
	sz	92.630	146.686	92.632	146.737	91.531	145.986
	sv	141.799		141.826		141.519	
Rib	sx	49.702	46.931	49.907	47.121	49.163	46.476
	sy	91.502	89.074	91.872	89.576	52.641	68.402
	sz	120.924	90.631	121.402	91.030	88.323	69.494
	sv	122.178		122.588		87.628	

* Here sx, sy, sz, and sv denote transverse, longitudinal (in-plane), vertical (to element plane) and v. Mises stress respectively

stage of analyses is given in Table 2 below. In the first stage of FE-analyses, the thickness of elastic wearing surface is always take as 40 mm (Here 40 mm is equal to concrete wearing surface's thickness, which rest directly on deck plate without any bonding layer between them) in all FE-analyses.

In the second stage of FE-analyses, elasticity module and Poisson ratio of concrete are 30 GPa and 0.3 respectively.

Before the first and second stage of FE-analyses, the analysis named T4E20P15NL is performed, whether geometrical non-linearity is effective on results. It is seen from Table 4, that FE-analyses done without considering any non-linearity do not influence the results. Averaged nodal values of results are also tabulated in Table 4 for comparison with non-averaged results. As a conclusion, there is almost no difference between the results of three FE-analyses in all both cases.

4. Influence of elastic moduli, poisson ratio and thickness of elastic wearing surface on orthotropic deck

4.1 General deformation behavior of bridge

With respect to Fig. 10 the change of wearing surface's Poisson ratio effects max. displacement of bridge very slightly, that the effect of Poisson ratio on it can be neglected. The max. change of max. displacement vector sum (U_{sum}) depending on Poisson ratio is only 1.2%. According to Fig. 10, U_{sum} is much more influenced by the elasticity module of wearing surface, when there exists a full bond between concrete wearing surface and deck plate. The rise of wearing surface's elasticity module from 20 GPa to 35 GPa leads to 8-11% decline of U_{sum}. If the bonding layer is modeled and not a full bond between concrete wearing surface and deck plate is considered, we can state that any increase of bonding layer's shear module or in wearing surface's thickness elicits to decrease of U_{sum} (See Figs. 11 and 12). It is seen from Fig. 12 that increasing wearing surface's thickness from 25 mm to 75 mm results in 44.44% fall of U_{sum}. Finally, wearing surface's U_{sum} is recommended as a controlling parameter on U_{sum}, not the material properties of normal concrete wearing surface.

4.2 Concrete wearing surface

Regarding Figs. 13, 14, 16 and 17 increasing bonding layer's shear module or wearing surface's thickness leads in general decreasing of stresses in concrete wearing surface. When bonding layer is ignored and concrete wearing surface is assumed as fully bonded to deck plate, v. Mises stress decreases depending on material properties of concrete wearing surface as illustrated

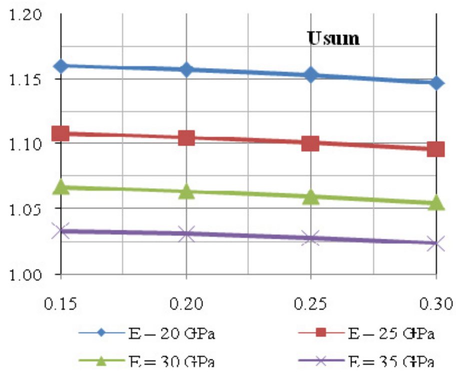


Fig. 10 Max. deformation vector as per wearing surface Poisson ratio for varied elastic moduli of wearing surface, in case of full bond

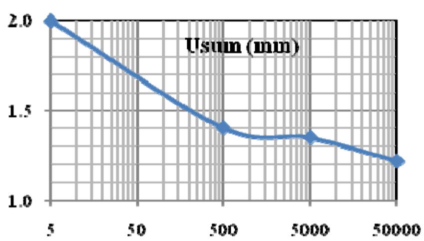


Fig. 11 Max. deformation vector as per bonding layer shear module (on logarithmic scale) risen in the bridge

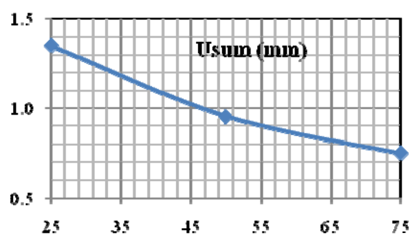


Fig. 12 Max. deformation vector as per wearing surface elastic module (on logarithmic scale) risen in the bridge

in Fig. 15. As a thumb rule it can easily be stated that the parameters to decline the stresses in wearing surface from strongest to weaks are: Thickness and elasticity module of concrete wearing surface, shear module of bonding layer and Poisson ratio of concrete wearing surface respectively.

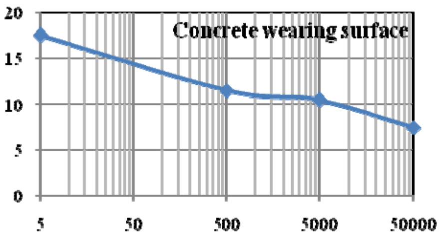


Fig. 13 Max. v. Mises stress as per bonding layer's shear module (on logarithmic scale) risen in concrete wearing surface

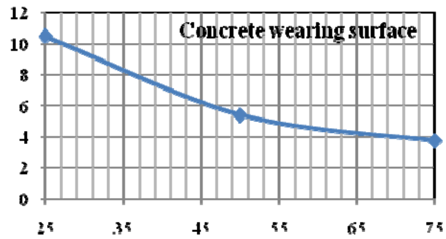


Fig. 14 Max. v. Mises stress as per wearing surface's thickness risen in concrete wearing surface, in case of full bond

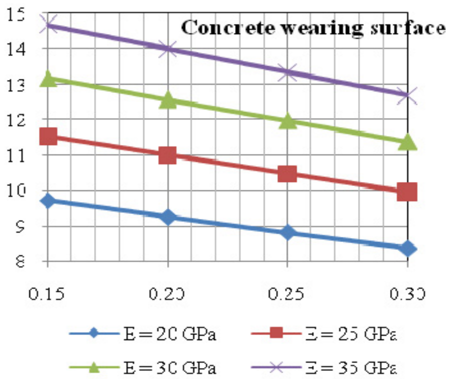


Fig. 15 Max. v. Mises stress in concrete surface as per concrete wearing surface's Poisson ratio for its different elasticity moduli, in case of full bond

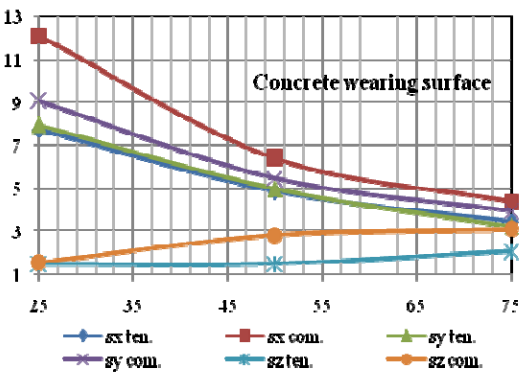


Fig. 16 Max. stresses as per wearing surface's thicknesses risen in concrete wearing surface, in case of full bond

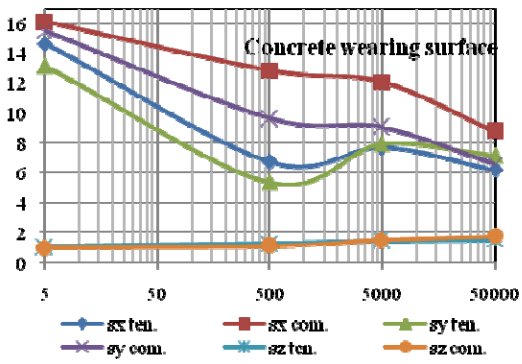


Fig. 17 Max. stresses as per bonding layer's shear module (on logarithmic scale) risen in concrete wearing surface

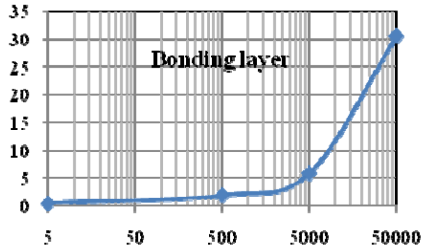


Fig. 18 Max. v. Mises stress in bonding layer as per bonding layer's shear module

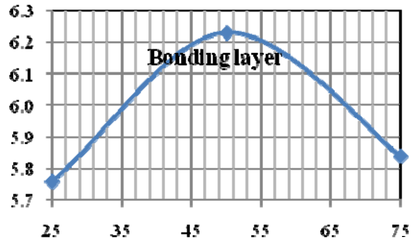


Fig. 19 Max. v. Mises stress in bonding layer as per wearing surface's thickness

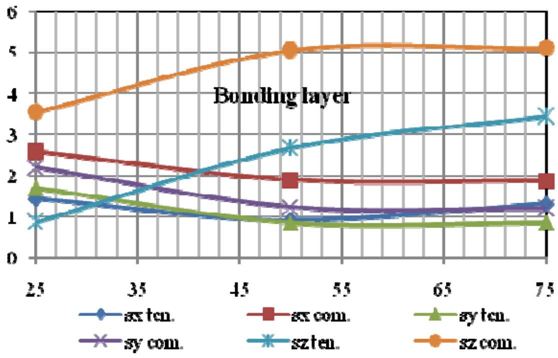


Fig. 20 Max. stresses in bonding layer as per wearing surface's thickness

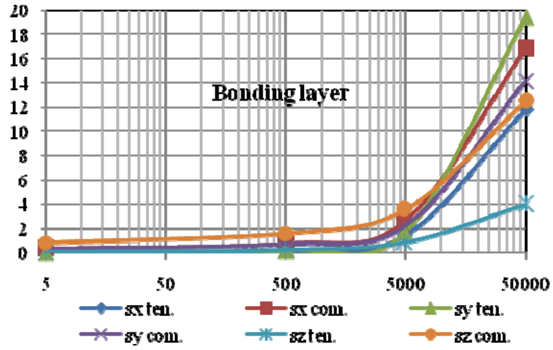


Fig. 21 Max. stresses in bonding layer as bonding layer's shear module

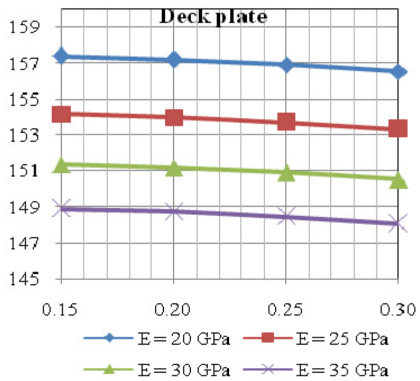


Fig. 22 Max. v. Mises stress in deck plate as per concrete wearing surface's Poisson ratio for its different elasticity moduli, in case of full bond

4.3 Bonding layer

As to Fig. 18, excessive value of bonding layer's shear module (50 GPa) causes a high value of v. Mises stress in bonding layer, that is 30,465 MPa and according to Fig. 19 increasing thickness of wearing surface has a top value of v. Mises stress in bonding layer as 6.23 MPa. As a result, it is recommended to choose bonding layer's shear module around 5,000 MPa and to increase the thickness of wearing surface to design of elastic wearing surface on orthotropic decks. Figs. 20 and 21 depict the variation of normal stresses depending on wearing surface's thickness and bonding layer's shear module.

4.4 Deck plate

Fig. 22 shows that Poisson ratio is not effective on stresses emerged in deck plate. Elasticity module of concrete wearing surface has an effect up to approximately 5.5%, which can also easily be neglected. According to Fig. 23 the excessive shear module value, 50,000 MPa of bonding layer leads only to 11.35% decrease of v. Mises stress in deck plate. With respect to Figs. 24 and 25 increasing wearing surface's thickness yields in a beneficial decrease of v. Mises and compressive transverse stress (sx com.) in deck plate. Increasing wearing surface's thickness from 25 mm to 75 mm yields in 30.19% decrease of v. Mises stress in deck plate. According to Fig. 26 increasing bonding layer's shear module effects tensile transverse stress (sx ten.), nevertheless just slightly other normal stresses in deck plate.

4.5 Cross-beam

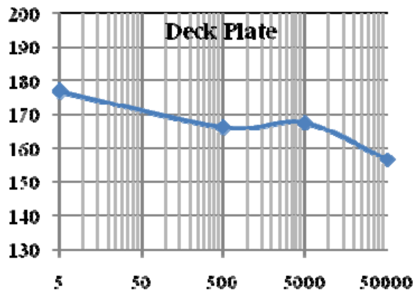


Fig. 23 Max. v. Mises stress in deck plate as per bonding layer' shear module

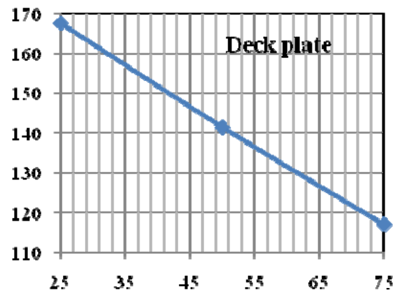


Fig. 24 Max. v. Mises stress in deck plate as per concrete wearing surface's thickness, in case of full bond

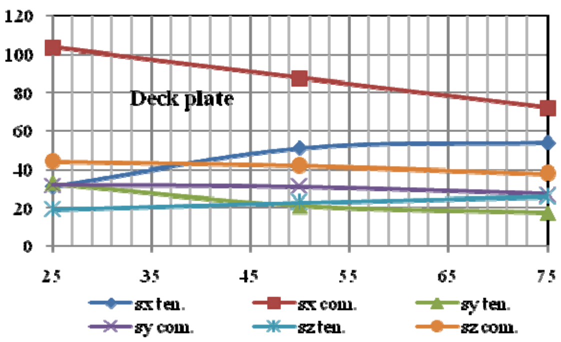


Fig. 25 Max. stresses in deck plate as per concrete wearing surface's thickness, in case of full bond

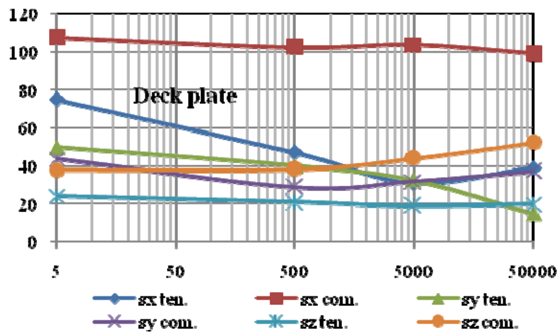


Fig. 26 Max. stresses in deck plate as per bonding layer's shear module

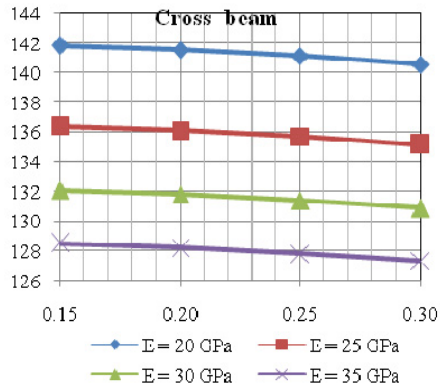


Fig. 27 Max. v. Mises stress in cross-beam as per concrete wearing surface's Poisson ratio for its different elasticity moduli, in case of full bond

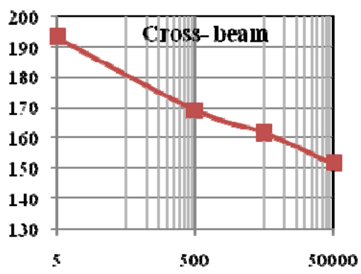


Fig. 28 Max. v. Mises stress in cross-beam as per bonding layer' shear module

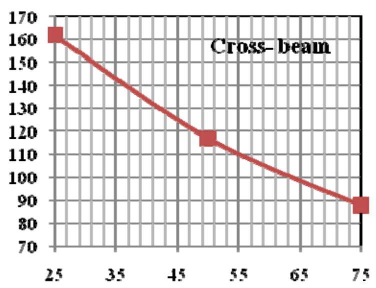


Fig. 29 Max. v. Mises stress in cross-beam as per wearing surface's thickess, in case of full bond

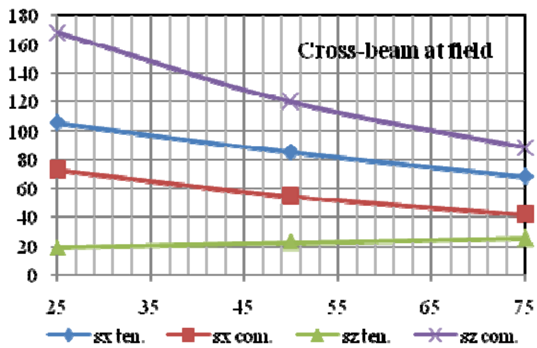


Fig. 30 Max. stresses in cross-beam as per wearing surface's thickess, in case of full bond

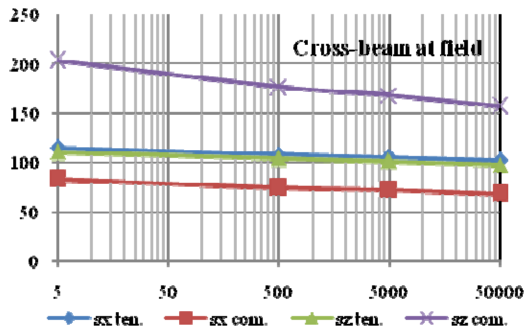


Fig. 31 Max. stresses in cross-beam as per bonding layer's shear module

When there exists a full bond between deck plate and concrete wearing surface, max. v. Mises stress in cross-beam decreases approximately 9.5%, if concrete wearing surface's elastic module changes from 20 GPa to 35 GPa (See Fig. 27). As per Figs. 28 and 31 rise of bonding layer's shear module causes decrease of v. Mises stresses and normal stresses in cross-beam, whereas increasing

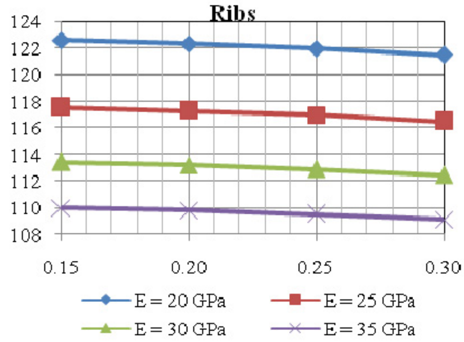


Fig. 32 Max. v. Mises stress in ribs as per concrete wearing surface's Poisson ratio for its different elasticity moduli, in case of full bond

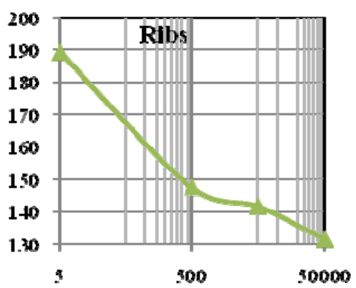


Fig. 33 Max. v. Mises stress in ribs as per bonding layer's shear module

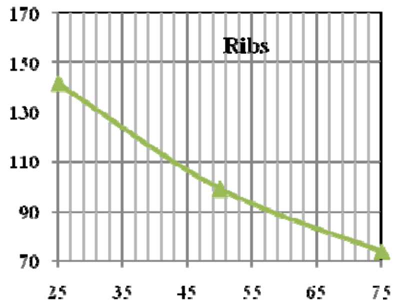


Fig. 34 Max. v. Mises stress in ribs as per wearing surface thickness, in case of full bond

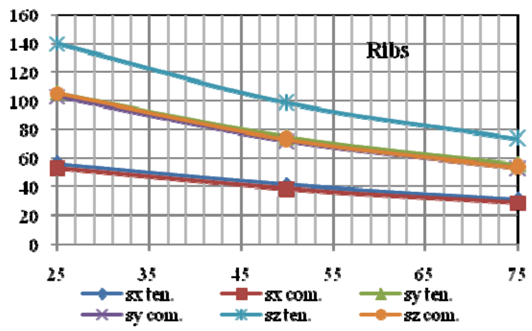


Fig. 35 Max. stresses in ribs as per wearing surface thickness, in case of full bond

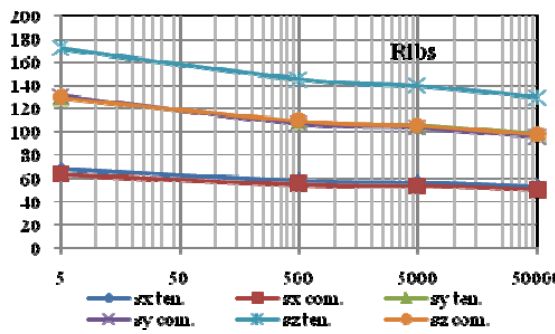


Fig. 36 Max. stresses in ribs as per bonding layer's shear module

wearing surface's thickness greatly reduces normal and v. Mises stresses in cross-beam, as given in Figs. 29 and 30.

4.6 Ribs

Fig. 32 shows that max. v. Mises stress developed in ribs is almost not effected by concrete wearing surface's Poisson ratio, when concrete wearing surface is fully bonded to deck plate. However, increasing concrete wearing surface's elastic module results in approximately 10.2% decrease of v. Mises stress in ribs. It is seen from Figs. 33 and 36 that, normal and v. Mises stresses developed in ribs reduce, when bonding layer's shear module rises. According to Figs. 34 and 35 max. normal and v. Mises stresses decrease heavily with the increase of wearing surface's thickness.

5. Conclusions

In this study a FE-model established by author is used to evaluate the stresses developing in orthotropic decks. It is investigated the influence of material properties and thickness of linear elastic wearing surface (in the case handled in this article, concrete wearing surface) on the stresses revealed in orthotropic decks. Results can be summarized as follows:

- It is seen from the results that Poisson ratio of elastic wearing surface has a vey negligible effect on displacements and stresses developed in any part of orthotropic deck. From this point of view, it is advised utilizing one-dimensional, unconfined tests to obtain elastic module of concrete pavement instead of more expensive confined stress tests. For a Poisson ratio value of concrete pavement any value between 0.15 and 0.30 can be used in FE-analyses.
- Thickness of wearing surface is the dominant parameter to really reduce all stresses and displacements in any part of orthotropic decks.
- Excessive shear module (50,000 MPa) of bonding layer results in very high v. Mises stress in bonding layer. As a result a shear module of around 5,000 MPa for bonding layer is suitable.
- For concrete wearing surfaces fully bonded on deck plate, increasing elastic module of concrete wearing surface from 20 GPa to 35 GPa leads approximately to 50% increase of v. Mises stress in concrete wearing surface, 5.5% decrease of v. Mises stress in deck plate, 9.5% decrease of v. Mises stress in cross-beam and 10.2% decrease of v. Mises stress in ribs.

References

AISC (1963), Design Manual for Orthotropic Steel Plate Deck Bridges, American Institute of Steel Construction, Chicago, IL, USA.

ANSYS (2010), User Manuals, Swanson Analysis Systems, USA.

Aoki, Y., Ishikawa, T., Banno, R., Kawano, H. and Adachi, Y. (2013), "Assessment, upgrading and refurbishment of infrastructures", *IABSE Symposium*, Rotterdam, Netherlands, May.

Aygul, M., Al-Emrani, M. and Urushadze, S. (2012), "Modelling and fatigue life assessment of orthotropic bridge deck details using FEM", *International Journal of Fatigue*, **40**, 129-142.

Braam, C.R. and Mulder, R. (2002), "Contec-Onderzoek kubussen en prisma's, Bepaling druksterkte en elasticiteitsmodule", Rapport 25.5-02-12, Delft, Netherlands. [In Dutch]

Buitelaar, P. (2002), "Ultra thin heavy reinforced high performance concrete overlays", *Proceedings of the 6th International Symposium on Utilization of High Strength / High Performance Concrete*, Leipzig, Germany.

Buitelaar, P., Braam, R. and Kaptijn, N. (2004), "Reinforced high performance concrete overlay system for rehabilitation and strengthening of orthotropic steel bridge decks", *Proceedings of 2008 Orthotropic*

Bridge Conference, *ASCE*, Sacramento, CA, USA, August.
Choi, D., Kim, Y., Yoo, H. and Seo, J. (2008), "Orthotropic steel deck bridges in Korea", *Proc*. *of 2008 Orthotropic Bridge Conference*, *ASCE*, Sacramento, CA, USA, August.
Connor, R., Fisher, J., Gatti, W., Gopalaratnam, V., Kozy, B., Leshko, B., McQuaid, D.L., Medlock, R., Mertz, D., Murphy, T., Paterson, D., Sorensen, O. and Yadlosky, J. (2012), "Manual for Design, Construction, and Maintenance of Orthotropic Steel Deck Bridges", U.S. Department Of Transportation Federal Highway Administration, Publication No. FHWA-IF-12-027, Washington, D.C., USA.
Erdoğan, S.T. and Erdoğan, T.Y. (2006), *Sorular ve Yanıtlarıyla Beton*, Türkiye Hazır Beton Birliği, Istanbul, Turkey. [In Turkish]
Eurocode 1 Part 2 (2003), Actions on structures – Traffic loads on bridges, European Committee for Standardization, Brussel, Belgium.
Eurocode 3 Part 1-1 (2001), Design of steel structures – General structural rules, European Committee for Standardization, Brussel, Belgium.
Eurocode 3 Part 1-9 (2003), Design of steel structures – Fatigue, European Committee for Standardization, Brussel, Belgium.
Fettahoglu, A. (2012), "Effect of deck plate thickness on the structural behaviour of steel orthotropic highway bridges", *Advanced in Civil Engineering*, Ankara Turkey, October.
Fettahoglu, A. (2013a), "Arranging thicknesses and spans of orthotropic deck for desired fatigue life and design category", *Int*. *J*. *Adv*. *Eng*. *Tech*., **6**(4), 1512-1523.
Fettahoglu, A. (2013b), "A FEA study conforming recommendations of DIN FB 103 regarding rib dimensions and cross-beam span", *Int*. *J*. *Civil Eng*. *Res*., **4**(3), 197-204.
Fettahoglu, A. (2013c), "Assessment on web slope of trapezoidal rib in orthotropic decks using FEM", *Sigma*, *J*. *Eng*. *Natural Sci*., **32**(1), 52-59.
Fettahoglu, A. (2014a), "Effect of cross-beam on stresses revealed in orthotropic steel bridges", *Steel Compos*. *Struct*., *Int*. *J*., **18**(1), 149-163.
Fettahoglu, A. (2014b), "Optimizing rib width to height and rib spacing to deck plate thickness ratios in orthotropic decks", *Struct*. *Eng*. *Mech*., *Int*. *J*., Rejected.
Fettahoglu, A. and Bekiroglu, S. (2012), "Effect of kinematic hardening in stress calculations", *Advanced in Civil Engineering*, Ankara, Turkey, October.
Fujino, Y. and Yoshida, Y. (2002), "Wind-induced vibration and control of Trans-Tokyo Bay Crossing Bridge", *J*. *Struct*. *Eng*., **128**(8), 1012-1025.
Han, B., Pu, Q.H., Shi, Z., Gao, L.Q. and Zhou, Y. (2013), "Study on orthotropic deck plate fatigue test model scheme of box girder of railway cable-stayed bridge", *Appl*. *Mech*. *Mater*., **361-363**, 1199-1205.
Honshu Shikoku Bridge Authority (2005), Accessed on January 17, 2005. www.hsba.go.jp
Hoopah, W. (2004), "Orthotropic decks for small and medium span bridges in France – Evolution and recent trends", *Proceedings of 2004 Orthotropic Bridge Conference*, *ASCE*, Sacramento, CA, USA, August.
Huang, C. and Mangus, A. (2008), "Redecking existing bridges with orthotropic steel deck panels", *Proceedings of 2008 Orthotropic Bridge Conference*, *ASCE*, Sacramento, CA, USA, August.
Huurman, M., Medani, T.O., Molenaar, A.A.A., Kasbergen, C., Liu, X. and Scarpas, A. (2002), "3D-FEM for the estimation of the behaviour of asphaltic surfacings on orthotropic steel deck bridges", *Proceedings of the 3rd International Symposium on 3D Finite Element for Pavement Analysis*, *Design & Research*, Amsterdam, Netherlands, April.
Jong, F.B.P.de (2007), "Renovation techniques for fatigue cracked orthotropic steel bridge decks", Ph.D. Dissertation, Delft University of Technology, Delft, Netherlands.
Kennedy, D.J.L., Dorton, R.A. and Alexander, S.D.B. (2002), "The sandwich plate system for bridge decks", *International Bridge Conference*, Pittsburgh, PA, USA, June.
Korniyiv, M. (2004), "Orthotropic deck bridges in Ukraine", *Proceedings of 2004 Orthotropic Bridge Conference*, *ASCE*, Sacramento, CA, USA, August.
Kozy, B., Connor, R., Paterson, D. and Mertz, D. (2011), "Proposed revisions to AASHTO-LRFD bridge design specifications for orthotropic steel deck bridges", *J*. *Bridge Eng*., **16**, 759-767.
Luo, R.D., Ye, M.X. and Zhang, Y.Z. (2010), "Study on influences of thickness of flange of U rib on

mechanical behaviors of orthotropic monolithic steel bridge deck system", *Adv. Mater. Res.*, **163-167**, 122-126.
Pouget, S., Suzéat, C., Di Benedetto, H. and Olard, F. (2012), "Modelling of viscous bituminous wearing course materials on orthotropic steel deck", *Mater. Struct.*, **45**(7), 1115-1215.
Poulis, J.A., Borger, D.P. and de Regt, P. (2000), *Metingen Rijkswaterstaat*, *Het verlijmen van diverse materialen op staalplaten ten behoeve van brugdekversteviging*, Rapport HI 2079, Delft, Netherlands. [In Dutch]
Sika (2006), Accessed on January 13, 2006. [In Dutch] www.sika.nl/tds_sikadur30_nl.pdf
Sim, H. and Uang, C. (2012), "Stress analyses and parametric study on full-scale fatigue tests of rib-to-deck welded joints in steel orthotropic decks", *J. Bridge Eng.*, **17**(5), 765-773.
Troitsky, M.S. (1987), *Orthotropic Bridges Theory and Design*, (2nd Edition), The James F. Lincoln Arc Welding Foundation, Cleveland, OH, USA.
Virlogeux, M. (2004), "The viaduct over the River Tarn", *Conference Proceedings Steelbridge*, *OTUA*, Paris, France, June.
Wolchuk, R. (2014), "Empirical design rules for effective utilization of orthotropic decks", *J. Bridge Eng.*, **19**(2), 152-158.
Yamada, K. and Ishikawa, T. (2011), "Fatigue evaluation of rib-to-deck welded joints of orthotropic steel bridge deck", *J. Bridge Eng.*, **16**(4), 492-499.

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