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Abstract: Orthotropic decks are composed of deck plate, ribs, and cross-beams and are frequently used in industry to span long distances, due to their light structures and load carrying capacities. Trapezoidal ribs are broadly preferred as longitudinal stiffeners in design of orthotropic decks. They supply the required stiffness to the orthotropic deck in traffic direction. Trapezoidal ribs are chosen in industrial applications because of their high torsional and buckling rigidity, less material and welding needs. Rib width, height, spacing, thickness of deck plate are important parameters for designing of orthotropic decks. In the scope of this study, rib width to height and rib spacing to deck plate thickness ratios are assessed by means of the stresses developed under different ratios of these parameters. For this purpose a FE-model of orthotropic bridge is generated, which encompasses the entire bridge geometry and conforms to recommendations given in Eurocode 3 Part 2. Afterwards necessary FE-analyses are performed to reveal the stresses developed under different rib width to height and rib spacing to deck plate thickness ratios. Based on the results obtained in this study, recommendations regarding these ratios are provided for orthotropic steel decks occupying trapezoidal ribs.

Subjects: Computer Aided Design (CAD); Structural Engineering; Transportation Engineering

Keywords: steel bridge; orthotropic deck; trapezoidal rib; Eurocode 3; FEM

ABOUT THE AUTHOR
Abdullah Fettahoglu, born 1979 in Trabzon is an assistant professor in Bursa Orhangazi University in Turkey. He worked in private sector and in universities prior to his current employment. He lectures structural mechanics, steel structures, highways, and railways. His research areas are orthotropic bridges, road pavement materials, highways and railways. His doctoral study and most of his articles are on the design recommendations of orthotropic steel bridges. The findings of this article highlight the dimensional ratios of orthotropic deck in terms of stresses developed in the deck. Therefore, other researchers focused on this subject can produce stress reducing new design solutions for orthotropic decks by means of the results provided by this article.

PUBLIC INTEREST STATEMENT
The three bridges spanning Bosphorus and connecting Asia and Europe together in Istanbul are the most important bridges of Turkey. All of these bridges have orthotropic deck structure and are designed by foreign companies, since the design of this type of structure is not well known to Turkish engineers. In this article, design of this bridge type is investigated using current engineering methodologies. Improving design methods and knowledge sharing related to this subject with international colleagues will be a benefit for Turkey for future projects.
1. Introduction

Construction of orthotropic decks with deck plate, cross-beams and trapezoidal ribs going through the cutouts in cross-beam webs started approximately in 1965 and is still 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, Tatora cable-stayed bridge (Honshu Shikoku Bridge Authority, 2005), Trans-Tokyo Bay Crossing steel box-girder bridge (Fujino & 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, Dorton, & Alexander, 2002). In Troitsky (1987), Huang and Mangus (2008), Hoopah (2004), Korniyiv (2004), and Choi, Kim, Yoo, and Seo (2008), Design Manual of Orthotropic Bridge-2 (2012) 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 3–5 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). 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. 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 an inevitable part of orthotropic decks to span long distances. In Figure 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 should be performed with respect to fatigue analysis because of repetitive wheel loads varying in type and magnitude. Therefore, 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 history in the absence of FEM. In addition to the correct

![Figure 1. Orthotropic deck with (a) open and (b) closed ribs (American Institute of Steel Construction, 1963; Connor et al., 2012).](image-url)
analysis and according to the design of orthotropic decks, their fabrication, shipping to construction area, and 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). In the scope of this study, the analysis based on conventional techniques of orthotropic bridges is summarized in the next section. Afterwards, FEM applied in this study is introduced in Section 3. In Section 4, the influence of width to height ratio of trapezoidal rib on stress distribution in orthotropic deck is handled. Subsequently, rib spacing to deck plate thickness ratio is evaluated and results are supplied in Section 5. Consequently, conclusions and recommendations for the design of orthotropic decks 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 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 Figures 2–8. In Figure 2, transverse flexural stress develops in deck plate and rib as a result of local deck plate deformation.

![Figure 2. Transverse flexural stress in deck plate and rib (Connor et al., 2012).](image)

![Figure 3. Transverse deck stress from rib differential displacements (Connor et al., 2012).](image)

![Figure 4. Longitudinal flexure and shear in rib acting as a continuous beam on flexible floor beam supports (Connor et al., 2012).](image)
In Figure 3, panel deformation yields transverse deck stress from rib differential displacements.

In Figure 4, longitudinal flexure and shear in rib acting as a continuous beam on flexible floorbeam supports results from rib longitudinal flexure.

In Figure 5, cross-beam in plane flexure results in flexure and shear in cross-beam acting as beam spanning between rigid girders.

In Figure 6, out-of-plane flexure of cross-beam web at rib due to rib rotation occurs as a result of cross-beam distortion.

In Figure 7, rib distortion causes Local flexure of rib wall due to cross-beam cutout.

In Figure 8, axial, flexural, and shear stresses develop from supporting girder deformations.

3. Analysis of orthotropic deck using FEM

So as to compare stresses developed under 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 under different parameter values are given in the subsequent sections. The FE-model of the bridge is generated using SHELL 181, which is illustrated in Figure 9.

The FE-model of Huurman et al. (2002) inspired the researcher to create FE-model of the bridge used in this research (Fettahoglu, 2012; Fettahoglu & Bekiroglu, 2012; Fettahoglu, 2013a, 2013b, 2013c). 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 Figure 10). 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. Figure 11 depicts the perspective front view of the whole orthotropic steel bridge, while Figure 12 shows the wheel loads and their arrangement on the entire bridge geometry. To decrease further the number of nodal unknowns solely the quarter of the bridge shown in Figure 13 is modeled by applying the necessary boundary conditions. As a result, number of elements and nodes in the FE-model of the bridge are 284 010 and 293 491 respectively, when rib width, height and spacing are 300, 275 and 300 mm, respectively. However, element and node numbers vary slightly, when rib width, height, or spacing is changed. Width of pedestrian road and deck plate in transverse direction are 1.1 and 6.3 m, respectively, while width of deck plate changes, when rip spacing changes.

Figure 9. Shell 181 finite element, which is used in this study (ANSYS, 2010).

Figure 10. (a) Connection of cross-beam to main girder, (b) connection of deck plate to pedestrian road.

Figure 11. Traditional orthotropic steel bridge as to Eurocode 3 Part 2 (2006).

\( x_u = \text{Element } x\text{-axis, if } x \text{ is not defined by user.} \\
\text{x = Element x-axis, which is defined by user.} \)
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. The initial height, width and spacing of the ribs used in orthotropic deck are 275, 300, and 300 mm, respectively. However, one of these parameters is changed in FE-analyses to evaluate its effect on results, while number of ribs and other dimensions are kept constant.

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 FE-analyses are based on geometric and material nonlinear theories, details of which can be found in Fettahoglu and Bekiroglu (2012).

Wheel loads and areas used in this study on FE-model of the bridge are given in Figure 13.

| Table 1. Material properties |
|-----------------------------|
| **Yield strength of steel (fy)** | 355 N/mm² | **Shear module (G)** | 81,000 N/mm² |
| **Ultimate strength (fu)** | 510 N/mm² | **Poisson ratio (υ)** | 0.3 |
| **Elasticity module (E)** | 210,000 N/mm² | **Density (ρ_{steel})** | 78.5 kN/m³ |
4. Rib width to height ratio

Rib width to height ratio of trapezoidal longitudinal stiffener is assessed by means of the stresses revealed for different height of ribs, while the width of rib is always kept as 300 mm. Results are compared with each other to interpret the deformation and stress behavior of the bridge for small, moderate, and high values of rib height. Figure 14 shows the parameters used in this study to evaluate the effect of dimensional ratios on stresses developed in orthotropic deck. Here a, b, e and t are rib width, height, spacing, and deck plate thickness, respectively. Results are illustrated mainly in two different ways, namely using contour graphics and tables. Since they both are scalar values and represent all displacement and stress components well, respectively at the point of interest, displacement vector sum and von Mises stress are chosen to interpret the results using contour graphics. Second, tables are used to present the values of vectorial stress components. Table 2 shows the stresses developed in deck plate, cross-beam and rib separately for different rib width to height ratios. In the FE-analyses all dimensions except rib height are always kept constant to be able to understand the effect of rib width to height ratio on deformation and stress behavior clearly. In the scope of this study, five different FE-analyses are performed for rib height values of 150, 200, 275, 300, and 375 mm, while other dimensions of bridge are kept constant.

In terms of deformation behavior of the whole structure max. deformation occurs, when the shortest rib height is used as seen in Figure 15. Max. deformation developed in deck plate decreases, while rib height increases, that can be seen in Figures 15, 17, 19, 21, and 23. So rib height is a parameter to limit the deformations rising in deck plate and also in the wearing surface, which lies directly on deck plate and deforms simultaneously together with deck plate. Figure 25 indicates that variation of deck plate deformations depending on rib height is not linear. Slope of this variation reduces as the rib height increases. After rib height of 300 mm, which is equal to rib width, the change in deformation is so less, which can be neglected.

In Table 2 max. stresses develop as follows,

| Type of structure | Type of Stress (MPa) | b = 150 mm (a/b = 2.00) | b = 200 mm (a/b = 1.50) | b = 275 mm (a/b = 1.09) | b = 300 mm (a/b = 1.00) | b = 375 mm (a/b = 0.80) |
|-------------------|----------------------|------------------------|------------------------|------------------------|------------------------|------------------------|
|                   | max. tens. | max. comp. | max. tens. | max. comp. | max. tens. | max. comp. | max. tens. | max. comp. | max. tens. | max. comp. |
| Deck plate        | $\sigma_x$     | 129.11     | 121.92     | 128.74     | 128.17     | 135.11     | 136.05     | 136.26     | 137.25     | 139.59     | 138.18     |
|                   | $\sigma_y$     | 113.12     | 88.80      | 79.22      | 80.88      | 60.02      | 73.94      | 58.09      | 72.42      | 55.25      | 69.21      |
|                   | $\sigma_z$     | ~ 0        |            |            |            |            |            |            |            |            |            |
| Cross-beam at field | $\sigma_x$     | 399.00     | 319.15     | 244.09     | 178.07     | 117.22     | 85.91      | 130.50     | 92.31      | 184.32     | 131.78     |
|                   | $\sigma_y$     | 12.40      | 9.62       | 12.85      | 10.29      | 12.33      | 10.41      | 11.95      | 10.31      | 10.52      | 13.31      |
|                   | $\sigma_z$     | 58.96      | 196.85     | 78.91      | 207.72     | 113.51     | 212.20     | 124.86     | 212.67     | 162.08     | 215.79     |
| Rib               | $\sigma_x$     | 190.02     | 217.58     | 127.83     | 137.08     | 84.14      | 80.49      | 81.32      | 77.53      | 81.16      | 76.98      |
|                   | $\sigma_y$     | 193.38     | 343.99     | 164.39     | 265.29     | 145.48     | 180.98     | 143.92     | 156.59     | 150.53     | 146.37     |
|                   | $\sigma_z$     | 294.23     | 284.85     | 330.03     | 238.43     | 244.22     | 164.45     | 216.18     | 161.68     | 212.38     | 169.39     |
• In deck plate,
  • $\sigma_x$ develops direct under wheel load, due to bending of deck plate. Max. comp. $\sigma_x$ develops at top, while max. tens. $\sigma_y$ develops at bottom of deck plate.
  • $\sigma_y$ develops under wheel load due to bending of deck plate. Two exceptions exist out of this situation, when $a/b = 1.5$ and $0.8$. At these situations max. tens. $\sigma_y$ develops at the top of deck plate, where deck plate is connected to normal cross-beam.
  • $\sigma_z$ is zero in all points.
• In cross-beam at field,
  • For \( b = 150 \) and \( 200 \) mm \( \sigma_x \) develops at rib web to normal cross-beam connection, where cope hole starts. In other points, \( \sigma_y \) develops at normal cross-beam cope hole rounds.
Figure 25. Variation of max. disp. vector sum as to rib height.

Figure 26. Variation of max. von Mises stress as to rib height.

Figure 27. Variation of max. longitudinal tension stress in deck plate as to rib height.

Figure 28. Variation of max. in-plane stresses in cross-beam as to rib height.
For $b = 150$ and $200$ mm $\sigma_y$ develops at the bottom flange of normal cross-beam. Max. comp. $\sigma_y$ develops at the top of flange plate, while max. tens. $\sigma_y$ develops at the bottom of flange plate.

Max. tens. and comp. $\sigma_z$ stresses develop at the rounds of cope hole. This cope hole is located in the interior side next to main-girder.

In ribs,

For $b = 150$ and $200$ mm $\sigma_y$ develops at the rib bottom flange next to main-girder. Max. and min. $\sigma_y$ stresses are both located in the longitudinal mid of bridge. $\sigma_y$ develops at the bottom flange of rib, where rib web connects to stiffened cross-beam in other points.

For $b = 150$ and $200$ mm max. tens. $\sigma_y$ develops at the rib bottom flange, longitudinally at the wheel load position. Max. comp. $\sigma_y$ develops at the rib, cross- beam and cope hole connection point. This rib is located next to main-girder. For $b = 275$ and $300$ mm max. tens. $\sigma_y$ develops at the rib web closer to stiffened cross-beam. Max. comp. $\sigma_y$ develops at the rib web to normal cross-beam connection. This rib is located next to main-girder. For $b = 375$ mm both max. $\sigma_y$ stresses develop at the rib web closer to stiffened cross-beam longitudinally.

| Table 3. Stresses developed in structural parts for different rib spacings ($a = 300$ mm, $t = 12$ mm, $b = 275$ mm) |
| --- |
| **Type of structure** | **Type of stress (MPa)** | $e = 150$ mm ($e/t = 12.5$ and $a/e = 2.00$) | $e = 225$ mm ($e/t = 18.75$ and $a/e = 1.33$) | $e = 300$ mm ($e/t = 25$ and $a/e = 1.00$) | $e = 375$ mm ($e/t = 31.25$ and $a/e = 0.80$) | $e = 450$ mm ($e/t = 37.5$ and $a/e = 0.67$) |
| **Deck plate** | $\sigma_x$ | 122.88 | 116.39 | 137.44 | 129.40 | 135.11 | 136.05 | 157.61 | 159.15 | 182.05 | 179.53 |
| | $\sigma_y$ | 65.47 | 75.88 | 73.2 | 89.39 | 60.02 | 73.94 | 107.69 | 121.01 | 132.69 | 140.60 |
| | $\sigma_z$ | ~ 0 | | | | | | | | | |
| **Cross beam at field** | $\sigma_x$ | 39.14 | 26.08 | 73.61 | 53.21 | 117.22 | 85.91 | 148.90 | 110.33 | 160.35 | 130.72 |
| | $\sigma_y$ | 4.93 | 5.51 | 9.95 | 7.88 | 12.33 | 10.41 | 12.7 | 11.63 | 12.04 | 11.78 |
| | $\sigma_z$ | 26.09 | 81.87 | 56.60 | 151.53 | 113.51 | 212.20 | 152.95 | 221.21 | 157.21 | 187.85 |
| **Rib** | $\sigma_x$ | 27.48 | 25.00 | 32.11 | 32.33 | 84.14 | 80.49 | 108.94 | 110.66 | 142.21 | 140.79 |
| | $\sigma_y$ | 41.98 | 78.19 | 75.57 | 106.43 | 145.48 | 180.98 | 167.70 | 202.93 | 137.11 | 229.88 |
| | $\sigma_z$ | 69.27 | 60.63 | 88.30 | 77.67 | 244.22 | 164.45 | 299.61 | 218.40 | 310.11 | 231.25 |
For $b = 150$ mm max. tens. $\sigma_z$ develops at the inner side of rib web, cross-beam and cope hole connection, while max. comp. $\sigma_z$ develops at the outer side of rib web at the same place. This rib is located next to main girder. For $b = 200$ mm max. tens. and comp. $\sigma_z$ stresses develop at the opposite webs of the rib, which is adjacent to main-girder. For $b = 275$ and 300 mm max. tens. $\sigma_z$ develops at the rib web to normal cross-beam connection, while max. comp. $\sigma_z$ develops at the rib web to stiffened cross-beam connection. This rib is located next to main girder. For $b = 375$ mm max. tens. and comp. $\sigma_z$ stresses develop at the opposite webs of the rib, which is adjacent to main girder.

With respect to von Mises stress distribution as rib height increases, the place and value of max. von Mises stress changes. For rib heights of 150, 200, and 275 mm max. von Mises stress rises at the rib web, where it is welded to the web of cross-beam. At this intersection, rib is aligned in traffic direction through cutouts in cross-beam. Therefore, welding between webs of rib and cross-beam is not continuous at the bottom flange of rib, which results in excessive distortion, von Mises stress value and so yielding of material as given in Figures 16 and 18. At rib heights of 275 and 300 mm max. von Mises stress takes much lesser values below yield stress and develops still at the same point, that is the bottom of welding between rib and cross-beam (see Figures 20 and 22). At rib height of 375 mm, which is higher than rib width, max. von Mises stress occurs at the cross-beam cutout edge, when the rib height is higher than rib width. From these results, it is recommended to select rib height equal to rib width to avoid excessive deformations in deck plate, yielding of steel material, and stress concentrations at the intersection point between rib and cross-beam webs under wheel loads used in this study.

Table 2 presents normal stress values as per type of structure and rib width to height ratio. Shear stresses developed in structural parts are so less that they are not included in the assessment of results. Regarding normal stresses, variation of rib height effects longitudinal max tension stress in deck plate, in-plane stresses in cross-beam, and normal stresses in ribs according to the stress values given in Table 2. Dependence of other stress components on rib height is of no importance as seen in Table 2. Increasing rib height results in lesser max. longitudinal tension stress in deck plate as given in Figure 27. Nevertheless, this stress decreases rapidly between rib heights of 150 and 275 mm and the slope of stress decrease becomes very small and can be assumed constant between rib heights of 300 and 375 mm.

Figure 28 illustrates the variation of in-plane stresses in cross-beam. Here, max. vertical compressive in-plane stress increases steadily proportional to rib height. However, this increase has a so
small a slope that the change of this stress component can be disregarded. Max. vertical tensional in-plane stress increases also steadily proportional to rib height with an almost constant slope and the change of this slope cannot be disregarded. This stress component takes its min. value (58.96 MPa) at the rib height of 150 mm and its max. value (162.08 MPa) at the rib height of 375 mm. Max. transversal both tensional and compressive in-plane stresses first decrease between rib heights of 150 and 275 mm, then increase between rib heights of 275 and 350 mm.

In trapezoidal ribs variation of all normal stresses is of importance and assessed subsequently as per Figure 29. Max. transversal stresses decrease rapidly between rib heights of 150 and 275 mm. The decrease of this stress component between 275 and 375 mm is almost of no importance. Max. longitudinal tension stress decreases when the rib height is 300 mm and increases between 300 and 375 mm. Max. longitudinal compressive stress decreases rapidly between rib heights of 150 and 300 mm and slightly between rib heights of 300 and 375 mm. Max. vertical tension stress first increases between rib heights of 150 and 200 mm, then decreases rapidly between rib heights of 200 and 300 mm and then a slight decrease in this stress component occurs between rib heights of 300 and 375 mm. Max. vertical compressive stress decreases rapidly between rib heights of 150 and 275 mm and slightly between 275 and 300 mm, but afterwards increases slightly between rib heights of 300 and 375 mm.
Figure 33. Max. von Mises stress = 106.497 MPa, when \( t = 12 \) mm and \( e = 150 \) mm.

Figure 34. Max. disp. vector sum = 1.796 mm, when \( t = 12 \) mm and \( e = 225 \) mm.

Figure 35. Max. von Mises stress = 144.719 MPa, when \( t = 12 \) mm and \( e = 225 \) mm.

Figure 36. Max. disp. vector sum = 2.793 mm, when \( t = 12 \) mm and \( e = 375 \) mm.

Figure 37. Max. von Mises stress = 295.286 MPa, when \( t = 12 \) mm and \( e = 375 \) mm.
5. Rib spacing to deck plate thickness ratio

Tables 3 and 4 show the stresses developed in deck plate, cross-beam and rib separately for different rib spacing to deck plate thickness ratios. In the FE-analyses of this parameter study rib width and height are always kept constant as 300 and 275 mm, respectively, then variation of rib spacing to deck plate thickness ratio is evaluated separately. Initial values of rib spacing and deck plate...
thickness are 300 and 12 mm, respectively and results of these FE-analyses for initial dimensional values are given in Figures 30 and 31. Subsequently, four different FE-analyses are performed, when rib spacing equals 150, 225, 375, and 450 mm, while other dimensions of bridge are kept constant. Displacements of steel highway bridge depending on rib spacing to deck plate thickness ratio is assessed using contour graphics, which depict the change in max. displacement vector sum. Figures 30, 32, 34, 36, 38, and 48 illustrate the variation of max. displacement vector sum on the whole bridge depending on rib spacing. As a result, max. displacement vector sum develops always in deck
plate and increases proportional to rib spacing. In Figure 48, it is observed that the value at the rib span of 300 mm is out of this trend. The reason of deviation from trend line at this point is the place of wheel loads, which rest on deck plate and rib web at the rib spacing of 300 mm, while wheel loads rest solely on deck plate at the FE-analyses of other rib spacings. While the distance between wheel loads (axle distance of the vehicle) remains always same, the width of bridge changes in transversal direction as rib spacing changes. If wheel loads rested always on rib web and deck plate or only on deck plate, the slope of curve given in Figure 48 would always have a positive slope.
Figure 52. Variation of max. stress values in deck plate as to rib spacing.

Figure 53. Variation of max. stress values in deck plate as to thickness of deck plate.

Figure 54. Variation of max. stress values in cross-beam as to rib spacing.
In the second set of FE-analyses instead of rib spacing, thickness of deck plate is changed to investigate the variations of displacement and stresses. Initial values of a, b, e and t (see Figure 14) are same as 300, 275, 300, and 12 mm, respectively. Afterwards, four different new FE-analyses are done for $t = 8, 10, 14,$ and $16$ mm, while other dimensions of bridge are kept constant. Results of these FE-analyses are given below.

From the assessment of Figures 30, 40, 42, 44, 46, and 49 max. displacement vector sum is inversely proportional to the thickness of deck plate and decreases as deck plate thickness increases. Eurocode 3 Part 2 (2006) Annex C1.2.2 recommends $e/t \leq 25$ and $e \leq 300$ mm for design of orthotropic deck. According to this recommendation $e = 300$ mm and $t = 12$ mm provide min. conditions to design orthotropic decks. Figure 49 shows that deviations from this min. condition by decreasing deck plate thickness results in localized and rapidly increased displacements in deck plate. This
result is also valid if deviations from min. condition occur by increasing rib spacing as proven in Figure 48. Finally the numerical results obtained under wheel loads used in this study support recommendations of Eurocode 3 Part 2 (2006) regarding rib spacing to deck plate thickness ratio based on the displacements developed in deck plate in terms of deformation criteria.

Increasing rib spacing yields in increase in von Mises stress as seen in Figure 50. The slope of the curve given in Figure 50 states that max. von Mises stress increases rapidly between rib spacing of 150 and 300 mm, nevertheless it increases with a decreasing slope after rib spacing of 300 mm, which is a point of inflection, where the sign of curve’s curvature changes. The cause of this “S” shape of curve is the changing of stress behavior between rib spacings of 150 and 300 mm. At the rib spacings of 150 and 225 mm max. von Mises stress rises in deck plate and in cross-beam, respectively as given in Figures 33 and 35. Max. von Mises stress develops always in the connection of rib and cross-beam webs as given in Figures 31, 37, and 39 during and after rib span of 300 mm. Accordingly, Figure 50 shall be seen as two curves before and after rib spacing of 300 mm, which is the min. recommendation of Eurocode 3 Part 2 (2006) and satisfies the ratio, e/t = 25. Figure 51 reveals that increasing of deck plate thickness causes decreasing of max. von Mises stress. At the deck
plate thickness of 8 mm, max. von Mises stress localized direct under wheel loads takes a very high value close to yield stress (see Figure 41). Therefore, deck plate thickness under 10 mm shall not be allowed in orthotropic decks under wheel loads used in this study. Max. von Mises stress develops always in the connection between cross-beam and rib webs as given in Figures 31, 43, 45, and 47 in deck plates having thickness higher than 8 mm. The magnitudes of shear stresses in comparison with normal stresses developed in orthotropic deck are so less, that they are not involved in the assessment of results. The stress components, which vary essentially depending on rib spacing to deck plate thickness ratio are given in Figures 52-57.

In Table 3,

- In deck plate,
  - For $e = 150, 225$ and $300$ mm $\sigma_x$ develops direct under wheel load, due to bending of deck plate. Max. comp. $\sigma_x$ develops at top, while max. tens. $\sigma_x$ develops at bottom of deck plate. For $e = 375$ mm max. tens. $\sigma_x$ develops at the bottom of deck plate under wheel load closer to normal cross-beam. Max. comp. $\sigma_x$ develops at the top of deck plate under wheel load closer to stiffened cross-beam. For $e = 450$ mm max. tens. $\sigma_x$ develops at the bottom of deck plate under wheel load closer to stiffened cross-beam. Max. comp. $\sigma_x$ develops at the top of deck plate under wheel load closer to normal cross-beam.
  - For $e = 150, 225$ and $300$ mm $\sigma_y$ develops under wheel load due to bending of deck plate. Max. tens. $\sigma_y$ develops at the bottom of deck plate under wheel load closer to normal cross-beam. For $e = 375$ and $450$ mm max. comp. $\sigma_y$ develops at the top of deck plate under wheel load closer to stiffened cross-beam.
  - $\sigma_z$ is always zero in deck plate.

- In cross-beam at field,
  - For $e = 150, 225$ and $300$ $\sigma_x$ develops at normal cross-beam cope hole rounds. For $e = 375$ and $450$ max. tens. and comp. $\sigma_x$ arise at the rib web, normal cross beam and cope hole connection points at opposite sides of cope hole.
  - $\sigma_x$ develops at the bottom flange of normal cross-beam. Max. comp. $\sigma_x$ develops at the top of flange plate, while max. tens. $\sigma_x$ develops at the bottom of flange plate.
  - Max. tens. and comp. $\sigma_y$ stresses develop at the rounds of cope hole. This cope hole is located in the interior side next to main girder.

- In ribs,
  - For $e = 150$ mm both $\sigma_x$ stresses develop at the bottom flange of rib adjacent to main girder. They arise in the longitudinal mid of bridge. For $e = 225$ and $300$ mm $\sigma_x$ develops at the bottom flange of rib, where rib web connects to stiffened cross-beam. For $e = 375$ mm max. tens. $\sigma_x$ arises at bottom flange of rib, located longitudinally closer to stiffened cross-beam. Max. comp. $\sigma_x$ arises at bottom flange of rib, located longitudinally closer to normal crossbeam. For $e = 450$ mm both $\sigma_x$ stresses develop at the bottom flange of rib adjacent to main girder. They arise in the longitudinal mid of bridge.
  - For $e = 150$ mm max. comp. $\sigma_y$ develops at the bottom flange of rib adjacent to main girder. They arise in the longitudinal mid of bridge. Max. tens. $\sigma_y$ develops at the bottom flange of rib adjacent to main girder, closer to stiffened crossbeam. For $e = 225$ mm max. comp. $\sigma_y$ develops at the bottom flange of rib adjacent to main girder located longitudinally at normal crossbeam location. Max. tens. $\sigma_y$ develops at the rib web longitudinally closer to stiffened crossbeam. For $e = 300$ mm max. tens. $\sigma_y$ develops at the rib web closer to stiffened crossbeam. Max. comp. $\sigma_y$ develops at the rib web to normal cross-beam connection. This rib is located next to main-girder. For $e = 375$ mm max. tens. $\sigma_y$ arises at the rib web, located longitudinally at stiffened cross-beam position. Max. comp. $\sigma_y$ arises at the rib web, located
longitudinally at normal cross-beam position. For $e = 450 \text{ mm}$ both $\sigma_y$ develop at the opposite rib webs of the rib adjacent to main girder. Their position is longitudinally at the normal cross-beam location.

- For $e = 150 \text{ mm}$ max. tens. $\sigma_y$ develops at the rib web adjacent to main girder. Max. comp. $\sigma_z$ develops at the rib web to deck plate connection, longitudinally closer to stiffened cross-beam. For $e = 225 \text{ mm}$ max. tens. $\sigma_y$ develops at the rib web very close to stiffened cross-beam. Max. comp. $\sigma_z$ develops at the rib to deck plate connection longitudinally almost between normal and stiffened cross-beams. For $e = 300 \text{ mm}$ max. tens. $\sigma_y$ develops at the rib web to normal cross-beam connection, while max. comp. $\sigma_z$ develops at the rib web to stiffened cross-beam connection. This rib is located next to main girder. For $e = 375 \text{ mm}$ max. comp. $\sigma_y$ develops at the rib web, located longitudinally at stiffened cross-beam position. Max. tens. $\sigma_y$ develops at the rib web, located longitudinally at normal cross-beam position. For $e = 450 \text{ mm}$ both $\sigma_y$ develop at the opposite rib webs of the rib adjacent to main girder. Their position is longitudinally at the normal cross-beam location.

In Table 4,

- In deck plate,
  - For $t = 8$ and $10 \text{ mm}$ both $\sigma_y$ arise at deck plate under wheel load, longitudinally closer to normal cross-beam. Max. tens. $\sigma_y$ arises at top, while max. comp. $\sigma_y$ arises at the bottom of deck plate. For $t = 12 \text{ mm}$ $\sigma_y$ develops directly under wheel load, due to bending of deck plate. Max. comp. $\sigma_y$ develops at top, while max. tens. $\sigma_y$ develops at the bottom of deck plate. For $t = 14 \text{ mm}$ both $\sigma_y$ arise at deck plate under wheel load, longitudinally closer to normal cross-beam. Max. tens. $\sigma_y$ arises at top, while max. comp. $\sigma_y$ arises at the bottom of deck plate. For $t = 16 \text{ mm}$ max. tens. $\sigma_y$ arises at the bottom of deck plate, under wheel load, longitudinally closer to stiffened cross-beam, max. comp. $\sigma_y$ arises at bottom of deck plate, where deck plate connects to normal cross-beam.
  - For $t = 8$ and $10 \text{ mm}$ max. tens. $\sigma_y$ develops at bottom of deck plate, closer to normal cross-beam, max. comp. $\sigma_y$ develops at the top of deck plate, closer to stiffened cross-beam. For $t = 12 \text{ mm}$ $\sigma_y$ develops under wheel load due to bending of deck plate. For $14 \text{ mm}$ max. comp. $\sigma_y$ arises at the top of deck plate under wheel load closer to stiffened cross-beam. Max. tens. $\sigma_y$ arises at the top of deck plate, and deck plate to normal cross-beam connection. For $16 \text{ mm}$ max. tens. $\sigma_y$ arises at the bottom of deck plate, under wheel load, longitudinally closer to stiffened cross-beam, Max. comp. $\sigma_y$ arises at the bottom of deck plate, where deck plate connects to normal cross-beam.
  - $\sigma_y$ is zero in all points of deck plate.

- In cross-beam at field,
  - For $t = 8$ and $10 \text{ mm}$ tens. $\sigma_y$ arises at rib, normal cross-beam and cope hole connection. Max. comp. $\sigma_y$ arises at round of normal cross-beam’s cope hole. For $t = 12 \text{ mm}$ $\sigma_y$ develops at normal cross-beam cope hole rounds. For $t = 14$ and $16 \text{ mm}$ both $\sigma_y$ stresses develop in the round of normal cross-beam’s cope hole.
  - For $t = 8$ and $10 \text{ mm}$ max. tens. $\sigma_y$ develops at the top of normal cross-beam’s bottom flange, while max. comp. $\sigma_y$ develops at bottom of flange. For $t = 12 \text{ mm}$ $\sigma_y$ develops at the bottom flange of normal cross-beam. Max. comp. $\sigma_y$ develops at the top of flange plate, while max. tens. $\sigma_y$ develops at the bottom of flange plate. For $t = 14$ and $16 \text{ mm}$ max. tens. $\sigma_y$ develops at the top of normal cross-beam’s bottom flange, while max. comp. $\sigma_y$ develops at bottom of flange.
  - For $t = 8$ and $10 \text{ mm}$ $\sigma_y$ stresses develop at the round of normal cross-beam cope hole. For $t = 12 \text{ mm}$ max. tens. and comp. $\sigma_y$ stresses develop at the rounds of cope hole. This cope hole is located in the interior side next to main girder. For $t = 14$ and $16 \text{ mm}$ max. $\sigma_y$ stresses develop at the round of normal cross-beam cope hole.
• In ribs,
  - For $t = 8$ and $10$ mm both $\sigma_x$ stresses arise at bottom flange of rib adjacent to main girder and longitudinally at the stiffened cross-beam location. For $t = 12$ mm $\sigma_x$ develops at the bottom flange of rib, where rib web connects to stiffened cross-beam. For $t = 14$ and $16$ mm both $\sigma_x$ stresses arise at bottom flange of rib adjacent to main girder and longitudinally at the stiffened cross-beam location.
  - For $t = 8$ and $10$ mm max. tens. $\sigma_y$ develops at rib web adjacent to main girder, and longitudinally at the stiffened cross-beam location. Max. comp. $\sigma_y$ develops at the rib web adjacent to main girder and longitudinally at the normal cross-beam location.
  - For $t = 8$ and $10$ mm max. tens. $\sigma_z$ arises at the rib web adjacent to main girder and longitudinally at the normal cross-beam location. Max. comp. $\sigma_z$ arises at the rib web adjacent to main girder and longitudinally at the normal cross-beam location.

As seen in Figure 52 and in Figure 53 max. transversal and longitudinal stresses in deck plate increases with the increase of rib spacing and decrease of deck plate thickness.

Variation of deck plate thickness results in slight changes in max. transversal in-plane stresses developed in cross-beam and is inversely proportional with transversal in-plane stresses as given in Figure 55. However increase of rib spacing causes rapidly higher transversal and vertical in-plane stresses in cross beam (see Figure 54).

All stress components developed in ribs are inverse linear proportional to deck plate thickness as given in Figure 57. According to Figure 56, transversal stresses in ribs almost do not change between rib spacings of 150 and 225 mm, however they increase rapidly proportional to rib spacing between rib spacings of 225 and 450 mm. Max. vertical stresses in ribs increase very slightly from rib spacings of 150–225 mm and from rib spacings of 375–450 mm. However vertical stresses in ribs increase enormously from rib spacings of 225 mm to rib spacings of 375 mm. Longitudinal stresses developed in ribs increase also proportional to rib spacing as seen in Figure 56.

There is an exception, max. longitudinal tension stress decreases from rib spacings of 375 mm to rib spacing of 450 mm as seen in Figure 56, since the point and phenomenon of stress behaviors are different at rib spacing of 375 and 450 mm. The cause of this exception is explained in Figure 58. While max. tension and compression longitudinal stresses rise in the rib next to main girder at rib spacing of 375 mm, they develop at the second rib away from the main girder at rib spacing of 450 mm. The main reason of this issue is actually increasing the width of deck plate in transversal direction proportional to rib spacing, as the axle distance between wheel loads as to bridge’s mid-point (symmetry center) remains same (see Figure 59).

6. Conclusions
In this study a FE-model established by researcher is used to evaluate the stresses developing in orthotropic decks. It is investigated the influence of dimensional parametric ratios on the deformations and stresses developed in orthotropic deck. These dimensional parametric ratios are the rib
width to height ratio and the rib spacing to deck plate thickness ratio under used wheel loads employed in this study. The results summarized below are the findings of this study,

- Numerical results obtained in this study support recommendations of Eurocode 3 Part 2 (2006) regarding rib spacing to deck plate thickness ratio as $e/t \leq 25$ and $e \leq 300$ mm under wheel loads used in this study. Since same results with Eurocode 3 Part 2 (2006) are obtained in this study, numerical model used is considered satisfactory.

- Another result obtained in this study is deck plate thickness less than 10 mm leads to localized high stress concentrations direct under wheel loads and should not be allowed to be used for designing of orthotropic decks.

- It is concluded from the overall assessment of numerical results that height of trapezoidal rib should be equal to its width with respect to used wheel loads, deck dimensions used in this study, deformations and stresses developed in orthotropic decks. Although it would be appropriate to compare the numerical results with experimental test, author considers the affinity of results between this study and Eurocode 3 Part 2 (2006) can be seen as benchmark of his FEA-model. However, a new study of the author, which will take one or two years including comparison of numerical results with experimental tests and which is funded by the government is now in progress.

- To avoid high to moderate stresses or stress concentrations in cross beam or in ribs, dimensional parametric ratios should be as $e/t \leq 225/12 = 18.75$ and $a/e \geq 300/225 = 1.33$ under wheel loads used in this study.

- To allow moderate stress values in orthotropic deck structure dimensional parametric ratios should be as $25 > e/t \geq 18.75$ and $1.33 > a/e \geq 1$.

- If $e/t >> 25$ and $a/e << 1$, stress concentrations at the intersection between rib and cross-beam webs together with yielding of material can occur under wheel loads used in this study.

- Of course, dimensions of orthotropic deck should be determined as per fatigue strength, however instead of repeating fatigue calculations after every FE-analysis, it is preferred in this study using von Mises stress, which is a function of stress phenomena at every structural point and a scalar value. As a result, von Mises stress is used for comparison of results of FE-analyses with each other. Consequently, dimensional parametric ratios investigated in this research are of great importance to define the places and values of max. stresses and so control the stresses developing in orthotropic deck. Author will focus on the effect of different loading schemes such as quasi- static and dynamic loads in his future studies.
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