ARRANGING THICKNESSES AND SPANS OF ORTHOTROPIC DECK FOR DESIRED FATIGUE LIFE AND DESIGN CATEGORY

Abdullah Fettahoglu
Department of Civil Engineering, Yildiz Technical University, Istanbul, Turkey

ABSTRACT

Orthotropic steel highway bridges are subject to variable traffic loads, which differ in type and magnitude. Most of these bridges were built in 1960's under design traffic load, which reflects the traffic conditions of those times. However, the number and weight of vehicles in traffic have increased since then too much in comparison to today. As a result these bridges are loaded more than their designed traffic loads and hence bridges' fatigue lives are shorten. As a remedy for this issue, thicknesses of fatigue sensitive structural parts of bridge shall be determined under today' s valid wheel loads and design category for desired fatigue life. In the scope of this study the traditional steel orthotropic highway bridge is analyzed using a FE- model, which encompasses bridge' s entire geometry. The traffic load is selected so conservatively, that it is composed of static wheel loads and wheel load area, which comply with the wheels of vehicles used today in traffic. Subsequently, fatigue lives of four fatigue sensitive structural parts of bridge are calculated. These are critical section in web of cross girder due to cut outs, weld connecting deck plate to trapezoidal rib, continuous longitudinal stringer and deck plate. Finally, required thicknesses and spans of these structural parts depending on their fatigue lives and design categories are given.

KEYWORDS: *Deck plate, Longitudinal stringer, Cross beam, Fatigue, FEM, Steel Bridge*

I. INTRODUCTION

Construction of orthotropic decks with deck plate, cross- beams and trapezoidal ribs going through the cut- outs in cross beam webs started approximately in 1965 and is still used widely in industry [1]. 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 [2], Trans-Tokyo Bay Crossing steel box-girder bridge [3], 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 using trapezoidal stiffeners [4]. In England, Germany and Netherlands there are a lot of steel highway bridges having orthotropic decks [1]. The traditional orthotropic deck is composed of deck plate, longitudinal stringer and cross beams. The distance between longitudinal stringers and between cross beams are in general 300 mm and 3 m to 5 m respectively. In addition to deck structure, wearing course lying on deck plate and main girders transmitting load to supports are two important components of orthotropic bridges. While wearing course might be of asphalt or concrete, main girder might be of a girder, a truss, a cable stayed or a tied arch system. Wheel loads are first dispersed by wearing course and introduced in deck plate. Then longitudinal stringers transmit wheel loads to cross beams. Finally wheel loads are transferred from cross beams over main girders to the bridge's supports. When the orthotropic deck structure design was developed in 1960s, fatigue calculations were not considered in design principles [5]. In addition, fatigue strengths of structural parts forming orthotropic deck were also not known at that time. In time cracks have appeared and developed continuously in orthotropic decks, which shall not have been emerged in orthotropic deck according to design principles foreseen at that time. These cracks and improved fatigue theory of fluctuating variable loads reminded to calculate orthotropic decks as per fatigue strengths [6]. Afterwards, several research facilities have been started to obtain fatigue strengths of structural details of orthotropic decks [7,8 ,9 10, 11]. Finally, prEN 1993- 1- 9 [12] collects fatigue strengths of orthotropic deck details and is used throughout this study for the fatigue calculations. To simulate vehicle loads in traffic, static wheel loads and wheel load area, which comply with the wheels of vehicles used today in traffic, are selected. To calculate the stresses developed under wheel loads, a FE- model of traditional steel orthotropic highway bridge is established using ANSYS [13]. This FE- model encompasses bridge' s entire geometry, which conforms to recommendations of DIN FB 103 [14] and presented in the next section. Since the number and weight of vehicles in traffic are increased in time since 1960's up to now, the frequency of wheel loads on the bridges, hence the design traffic category of many existing bridges changed. As a solution to this problem, thicknesses and spans of fatigue sensitive structural parts of bridge are determined under today' s valid wheel loads and design traffic category for desired fatigue life in section III. These fatigue sensitive structural parts are the critical section in web of cross girder due to cut outs, weld connecting deck plate to trapezoidal rib, continuous longitudinal stringer and deck plate. Subsequently, results are assessed in section IV and finally, conclusions of this study together with future work are given in section V.

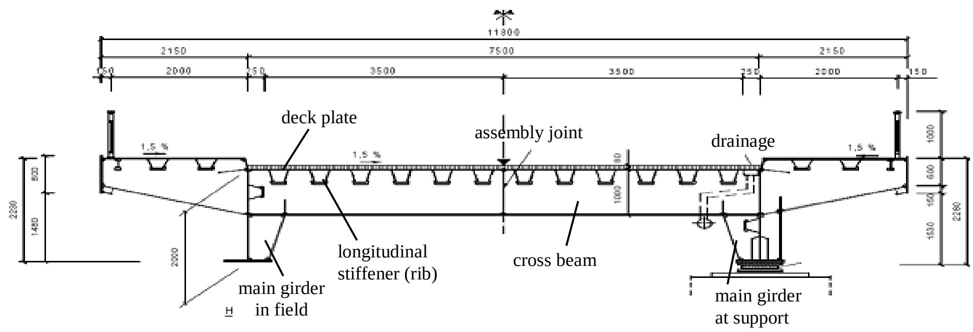
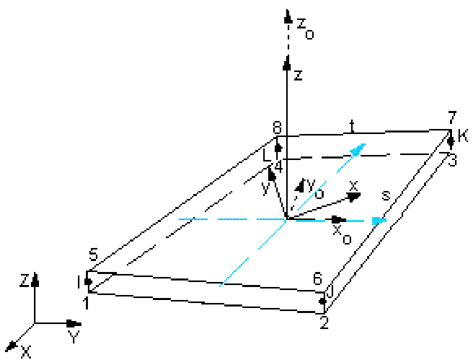


Figure 1. Traditional load bearing parts of steel orthotropic bridge.

II. FE- MODEL OF THE BRIDGE

So as to compare the stresses developed for different structural thicknesses and spans, all dimensions of the bridge shall be defined as variables in ANSYS [13]. Therefore an algorithm to provide this condition is written by means of APDL (Ansys Parametric Design Language). Afterwards thicknesses and spans of structural parts, which are of interest, are entered in ANSYS using this algorithm. Stresses developed for different thicknesses and spans are given in the next section. The FE- model of the bridge is generated using SHELL 181, which is given in Figure 2.



x_o = Element x- axis , if x is not defined by user.
x = Element x-axis, which is defined by user.

Figure 2. Shell 181 finite element, which is used in this study [13].

The FE model of Huurman et al. [15] inspired the researchers to create FE- model of the bridge used in this research [16, 17]. However, in the FE- model, which is generated using ANSYS [13] 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. [15] (See Figure 3). Because of the excessive number of nodal unknowns, dimensions of the bridge used in this research are chosen as short as possible. To decrease further the number of nodal unknowns solely the quarter of the bridge shown in Figure 4 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. However, element and node numbers vary slightly, when cross- beam span and / or rib span is / are changed, which is the situation handled in section 3.5. of this article. Actually cross- beam span is always taken as 3 m in all FE- analyses except the FE- analysis named as Redesign- 2. Width of pedestrian road and deck plate in transverse direction are 1.1 m and 6.3 m respectively, while width of deck plate changes, when rip span changes. Nevertheless, length of deck plate is always equal to 6 m (bridge span distance), when cross- beam span changes. That is, number of cross beams differs for cross beam span of 3 m (3 cross- beams) and 2 m (4 cross- beams).

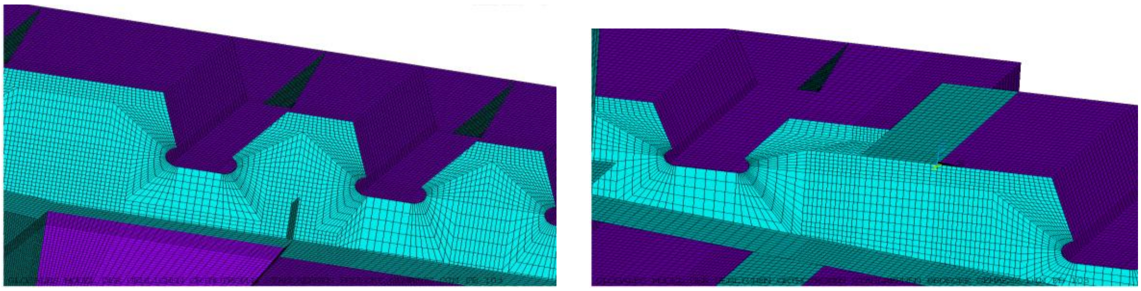


Figure 3. Left, connection of cross-beam to main girder. Right, connection of deck plate to pedestrian road.

The bridge analyzed spans 6 m in longitudinal direction and has stiffened main girders at supports, normal main girders at field (outside support areas), 2 exterior- 5 interior ribs, 1 rib in main girder and 1 rib in pedestrian road. The initial height, width and span of the ribs used in orthotropic deck are 275 mm, 300 mm and 300 mm respectively. However, rip span changes in some FE- analyses to evaluate its effect on results, while number of ribs and other dimensions are kept constant.

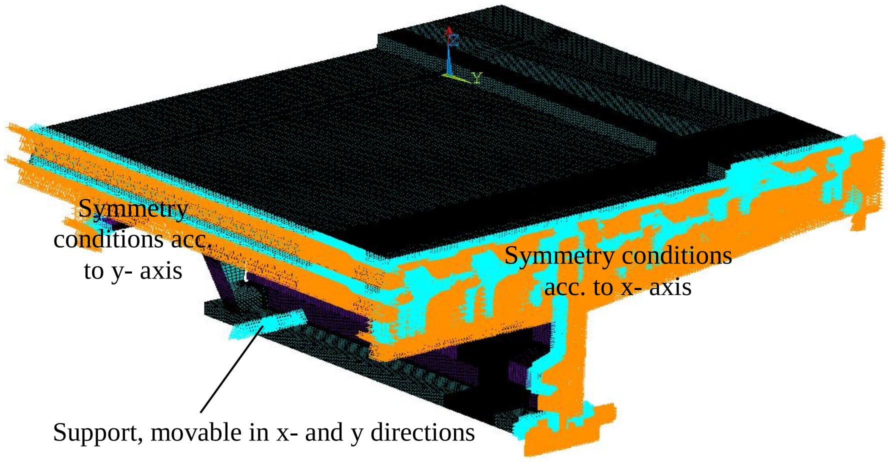


Figure 4. FE- model and boundary conditions of bridge quarter.

According to Capital II of DIN FB 103 [14] the yield stress and strength values of the selected steel material (S 355) are given in Table 1.

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

The wheel loads and wheel areas on FE- model of the bridge are given in Figure 5.

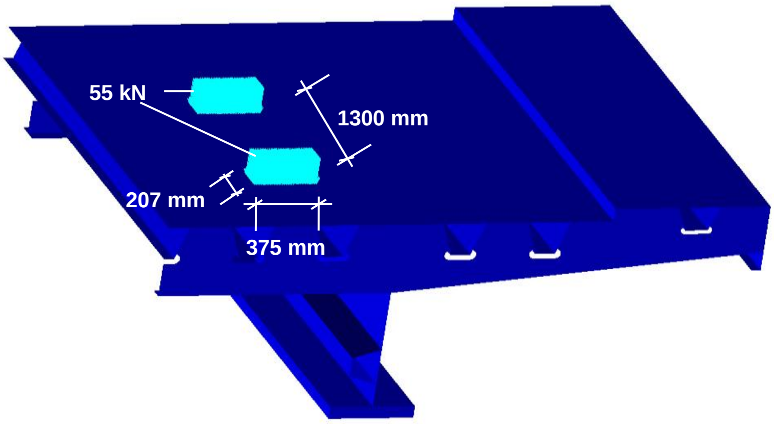


Figure 5. wheel loads and wheel areas on FE- model of the bridge.

III. Results & Discussion

3.1. Critical Section in Web of Cross Girder due to Cut Outs

In this structural part stresses because of Vierendeel effect play the distinctive role, when designing for desired fatigue life. Stress range at this part is calculated by means of the stress results of FE-analysis. Fatigue strength of this part is given in detail category 71 as per prEN 1993- 1- 9: 2003 (D) Table 8.8 [12]. Stress distribution in this critical part is depicted in Figure 6, in which web tickness of the cross girder is determined by stresses due to Vierendeel effect. Because entire bridge geometry is incorporated in the FE- model, stress concenrations develop at the edge of cut outs, even though the material does not yield. If the concentration of stresses at cut out edge is ignored and the distribution of stresses are assumed linear, the traditional stresses due to Vierendeel effect result in the values given in Figure 7. Since stress ranges given in S- N curves are based on stresses calculated according to linear elasticity theory, the linear distributed stresses in Figure 7 are taken into account for fatigue calculations.

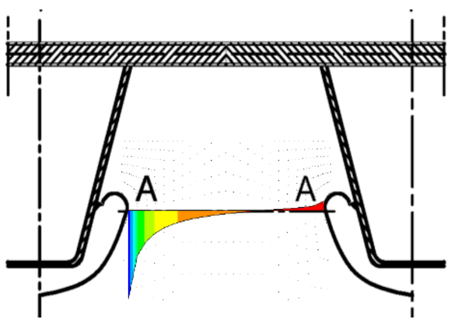


Figure 6. Stress distribution in cross girder in vicinity of cut outs. Simulation "BB".

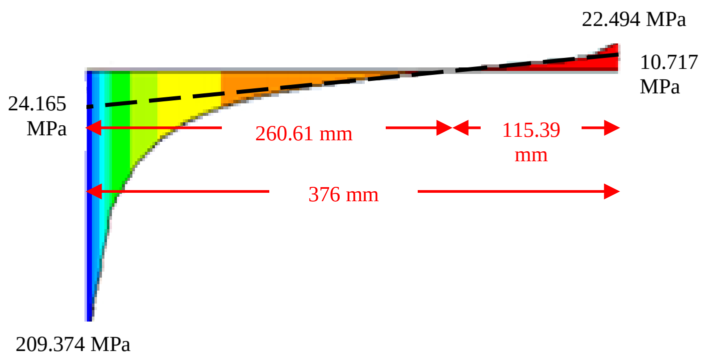


Figure 7. Stresses due to Vierendeel effect with and without concentration at edges. Simulation "BB"

First, the stress range of this structural part for its fatigue calculation is simply the double of max. absolute stress at the edge of cut out. Second, the endurance of this structural part is calculated using fatigue strength (S- N curve), which is fatigue detail category 71 given in Figure 8. Third, the fatigue life of structural part is calculated using Table 4.5 in ENV 1991- 3: 1995 [18] as per different web thicknesses of cross girder. Table 2 shows Table 4.5 in ENV 1991- 3: 1995 [18], which shows the occurance of stress range in a year according to bridge design traffic category.

Table 2. Number of vehicle passing per year for one traffic lane.

Design Traffic Category	Occurance of stress range per year for one traffic lane
1: Motorways and streets with 2 or more traffic lanes in every traffic direction and with high passing of trucks	$2x10^6$
2: Motorways and streets with average passing of trucks	$0.5x10^6$
3: Main streets with low passing of trucks	$0.125x10^6$
4: Local streets with low passing of trucks	$0.05x10^6$

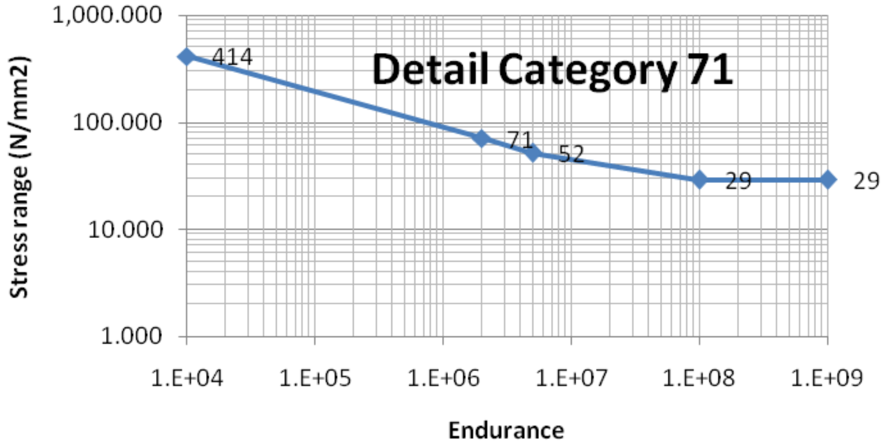


Figure 8. Fatigue strength curve 71.

$\Delta\ \sigma_{E,2}$ = 2x24.165 = 48.33 MPa
γ_{Ff} = 1.00
γ_{Mf} = 1.15 (According to prEN 1993- 1- 9: 2003 (D) Table 3.1: Damage tolerant assessment method with high consequence of failure [12])
γ_{Ff} x $\Delta\sigma_{E,2}$ = 48.33 < 61.74 = 71 / 1.15 = $\Delta\sigma_C$ / γ_{Mf}

The fatigue calculation done above is repeated for increased web thicknesses of cross girder. There are totally 4 conditions; web thickness of 14 mm (simulation BB), 16 mm (simulation QSD16), 18 mm (simulation QSD18) and 20 mm (simulation QSD20). Table 3 summarizes the results of fatigue calculation, namely fatigue lives as per chosen cross- beam web thickness and design traffic category.

Table 3. Fatigue life calculation of web of cross girder as per its thickness

Simulation's name	$\Delta\ \sigma_{E,2}$ (MPa)	N_i	Fatigue life (year) as per			
			Design Traffic Category 1	Design Traffic Category 2	Design Traffic Category 3	Design Traffic Category 4
BB	48.330	4 094 832	2.05	8.19	32.76	81.90
QSD16	42.808	6 574 653	3.29	13.15	52.60	131.49
QSD18	38.390	11 334 620	5.67	22.67	90.68	226.69
QSD20	34.770	18 598 335	9.30	37.20	148.79	371.97

According to Capital 4.6.1 General Issues 2(c) of DIN ENV- 1991- 3 [18] service life of bridges shall be 100 years for all design traffic categories. Fatigue lives depending on design traffic categories are calculated for cross- beam web thicknesses of 14 mm, 16 mm, 18 mm and 20 mm respectively. Figure 9 shows that increasing cross- beam web thickness leads increasing of its fatigue life. As design category number of bridge increases, slope of the curve given in Figure 16 increases also. However, required service life of bridge (100 years) is obtained only for design traffic categories of 3 and 4. As a result, recommended cross- beam web thicknesses for bridges of design traffic category 3 and 4 are 20 mm and 16 mm respectively.

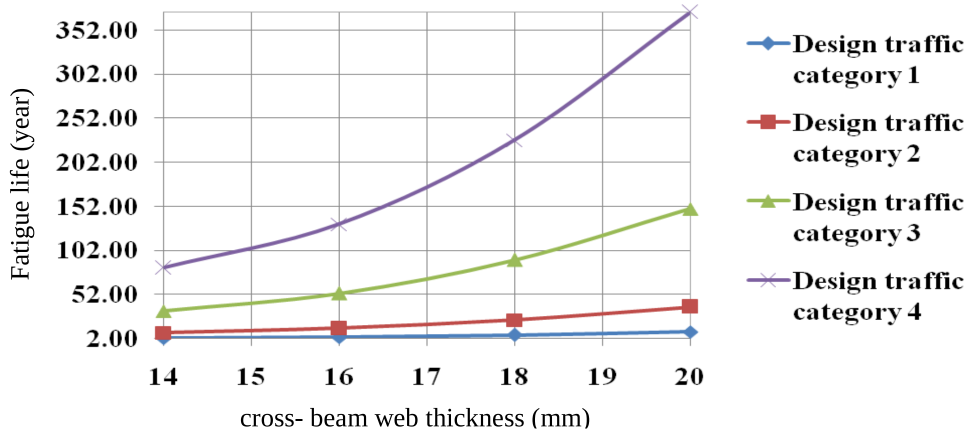


Figure 9. Fatigue lives of cross- beam web as to thickness and design traffic category.

3.2. Continuous Longitudinal Stringer, with Additional Cut-out in Cross Girder

The fatigue assessment of this structural part is based on the direct stress in the longitudinal stringer (see Figure 10). The fatigue strength of this structural part is determined by Fatigue Strength Curve 71 according to prEN 1993- 1- 9: 2003 (D) Table 8.8 [12]. Here Fatigue Strength Curve 71 is selected instead of Curve 80, since the thickness of cross beam web is 14 mm (higher than 12 mm).

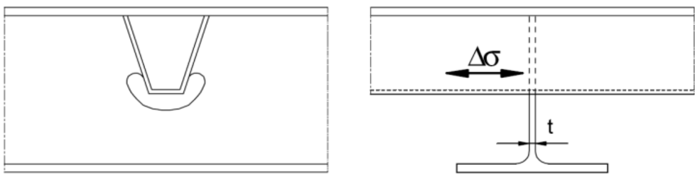


Figure 10. Fatigue sensitive structural part.

$\Delta\ \sigma_{E,2}$ = 2x82.162 = 164.324 MPa (See Figure 11)

γ_{Ff} = 1.00

γ_{Mf} = 1.00 (According to prEN 1993- 1- 9: 2003 (D) Table 3.1: Damage tolerant assessment method with low consequence of failure [12]).

$\gamma_{Ff} \times \Delta\sigma_{E,2} = \quad 164.324 > 71 = \Delta\sigma_C / \gamma_{Mf}$

The fatigue life of this structural part is very short even for design traffic category 4. Now 4 scenarios will be assessed, in which rib thicknesses are increased. These are the results of simulations having names of RD8NRV1, RD8NRV2, RD10NRV1 and RD10NRV2. In RD8NRV1, the thickness of one rib at both sides of main girder is increased from 6 mm to 8 mm. . In RD8NRV2, the thickness of two ribs at both sides of main girder is increased from 6 mm to 8 mm. In RD10NRV1, the thickness of one rib at both sides of main girder is increased from 6 mm to 10 mm. In RD10NRV2, the thickness of two ribs at both sides of main girder is increased from 6 mm to 10 mm. Increasing rib thickness leads lengthening of structure' s fatigue life as seen in Table 4, nevertheless none of the fatigue lives provides a meaningful lifespan for structure.

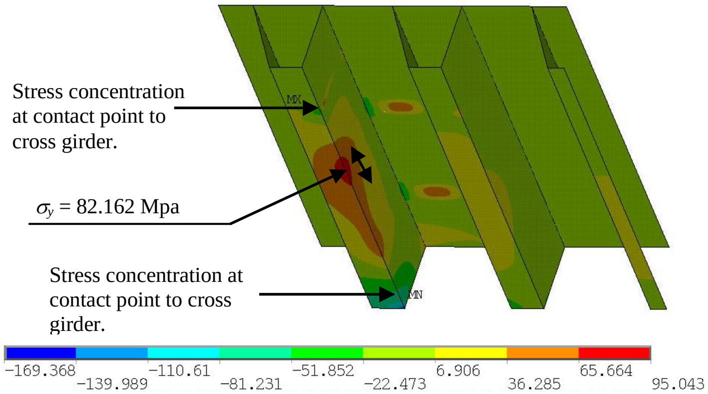


Figure 11. Extreme values of direct stresses in ribs. Simulation "BB"

Table 4. Fatigue life calculation of rib as per its thickness.

Simulation's name	$\Delta\ \sigma_{E,2}$ (Mpa)	N_i	Fatigue life (year) as per			
			Design Traffic Category 1	Design Traffic Category 2	Design Traffic Category 3	Design Traffic Category 4
BB	164.324	158 444	0.08	0.32	1.27	3.17
RD8NRV1	122.860	379 095	0.19	0.76	3.03	7.58
RD8NRV2	120.638	400 431	0.20	0.80	3.20	8.01
RD10NRV1	97.620	755 725	0.38	1.51	6.05	15.11
RD10NRV2	94.682	828 281	0.41	1.66	6.63	16.57

Figure 12 illustrates the variation of fatigue life of continuous longitudinal stringer. Neither traditional rib web thickness of 6 mm nor increased rib web thickness values of 8 mm and 10 mm provides required bridge' s service life, 100 years. The max. fatigue life of this structural part even for design traffic category 4 and using 10 mm rib web thickness is very low, 16.57 years. So, not increasing rib web thickness, but may be decreasing rib span and / or cross- beam span can supply required fatigue stiffness to this structural detail.

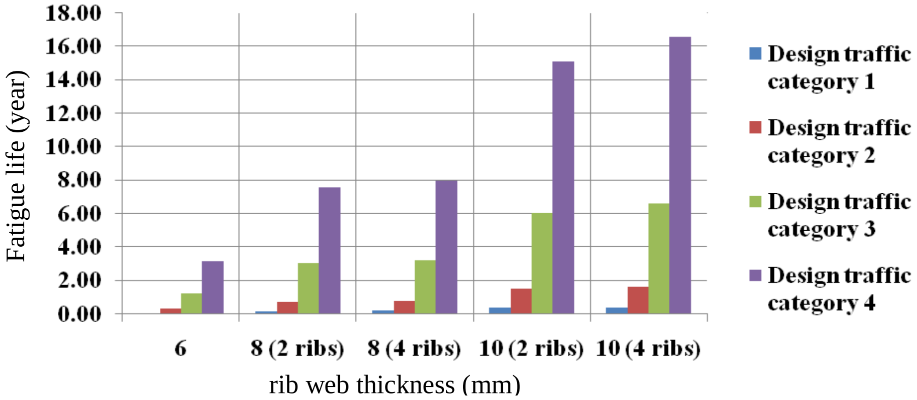


Figure 12. Fatigue lives of rib web as to thickness and design traffic category.

3.3. Weld connecting deck plate to rib

Weld connecting deck plate to rib is of importance as per fatigue strength on the basis of practical findings. Calculation of stress range depends on the moment in rib web (see Figure 13).

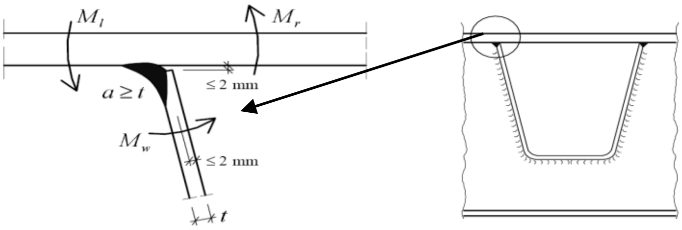


Figure 13. Weld connecting deck plate to rib.

According to prEN 1993- 1- 9: 2003 (E) Table 8.8 [12] assessment of fatigue life is based on direct stresses from bending of rib web. This direct stress is taken from the results of FE- analysis given in Table 5 (See also Figure 14).

$\Delta\ \sigma_{E,2}$ = 2x67.469 = 134.938 MPa
γ_{Ff} = 1.00
γ_{Mf} = 1.15 (According to prEN 1993- 1- 9: 2003 (D) Table 3.1: Damage tolerant assessment method with high consequence of failure [12])
$\gamma_{Ff} \times \Delta\sigma_{E,2} = 134.938 > 61.74 = \Delta\sigma_C / \gamma_{Mf}$

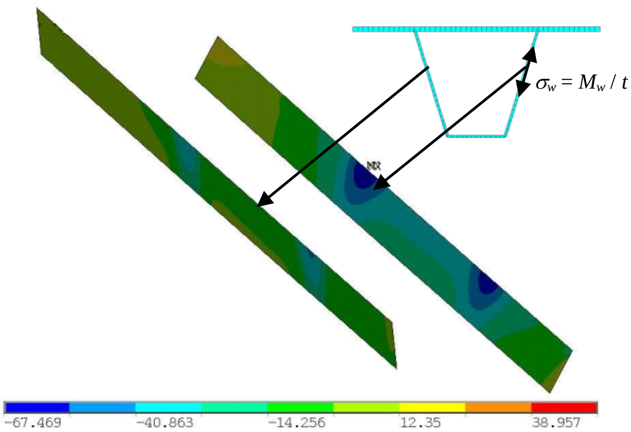


Figure 14. Direct stress values for fatigue assessment of weld. Simulation "BB"

If it is assumed that the contact area between deck plate and rib carries the traffic load in case of pressure stresses, the max. tension stress is equal to the stress range for the fatigue calculation of weld.
$\gamma_{Ff} \times \Delta\sigma_{E,2} = 52.26 < 61.74 = \Delta\sigma_C / \gamma_{Mf}$
The fatigue lives of weld are given in Table 5. Unfortunately, according to the results given in Table 5 any increase of rib web has almost no effect on the fatigue life of weld.

Table 5. Fatigue life calculation of weld between rib and deck plate as per rib thickness

Simulation's name	$\Delta\ \sigma_{E,2}$ (MPa)	N_i	Fatigue life (year) as per			
			Design Traffic Category 1	**Design Traffic Category 2**	**Design Traffic Category 3**	**Design Traffic Category 4**
BB	52.260	3 238 757	1.62	6.48	25.91	64.78
RD8NRV1	52.732	3 152 563	1.58	6.31	25.22	63.05
RD8NRV2	52.597	3 176 900	1.59	6.35	25.42	63.54
RD10NRV1	51.444	3 395 333	1.70	6.79	27.16	67.91
RD10NRV2	51.605	3 363 653	1.68	6.73	26.91	67.27

As given in Table 5 weld connecting deck plate to rib has not a fatigue life equals or higher than 100 years, when the thickness of rib 6 mm, 8 mm or 10 mm is. In addition, changing rib web thickness has almost no influence on fatigue life of weld connecting deck plate to rib.

3.4. Deck plate

The fatigue calculation of deck plate is done as per Fatigue Strength Curve 160, which is given in Figure 15. Therefore sharp edges, surface and rolling flaws on deck plate shall be improved by grinding, until smooth transition is achieved.

3.4.1. Deck plate part, which is on the rib web

$\Delta\ \sigma_{E,2}$ = 2x134.291 = 268.582 MPa (See Figure 16)

γ_{Ff} = 1.00

γ_{Mf} = 1.00 (According to prEN 1993- 1- 9: 2003 (D) Table 3.1: Damage tolerant assessment method with low consequence of failure [6])

γ_{Ff} x $\Delta\sigma_{E,2}$ = 268.582 > 160 = $\Delta\sigma_C$ / γ_{Mf}

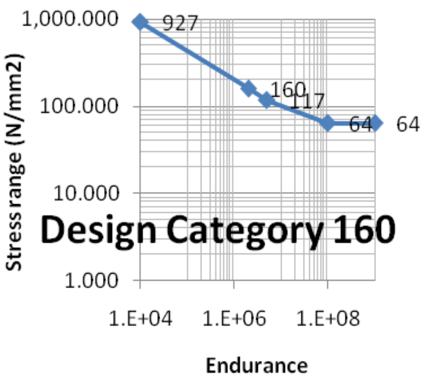


Figure 15. Fatigue Strength Curve 160

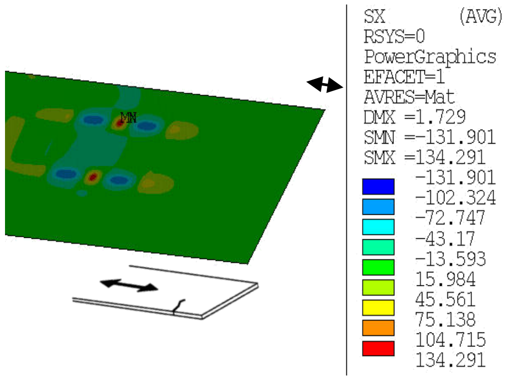


Figure 16. Direct stresses in deck plate for fatigue calculation. Simulation "BB"

Results tabulated in Table 6 indicate that increasing deck plate thickness has a positive effect on increasing fatigue life, however this does not provide satisfied values. In Table 6 deck plate thicknesses in simulations BB, DBD14 and DBD16 are 12 mm, 14 mm and 16 mm respectively.

Table 6. Fatigue life calculation of deck plate on rib web as per deck plate thickness.

Simulation's name	$\Delta\ \sigma_{E,2}$ (Mpa)	N_i	Fatigue life (year) as per			
			Design Traffic Category 1	Design Traffic Category 2	Design Traffic Category 3	Design Traffic Category 4
BB	268.582	413 330	0.21	0.83	3.31	8.27
DBD14	179.170	1 392 296	0.70	2.78	11.14	27.85
DBD16	121.240	4 493 552	2.25	8.99	35.95	89.87

3.4.2. Deck plate part between rib webs

γ_{Ff} x $\Delta\sigma_{E,2}$ = 107.342 + 102.377 = 209.719 > 160 = $\Delta\sigma_C$ / γ_{Mf} (See Figure 17)

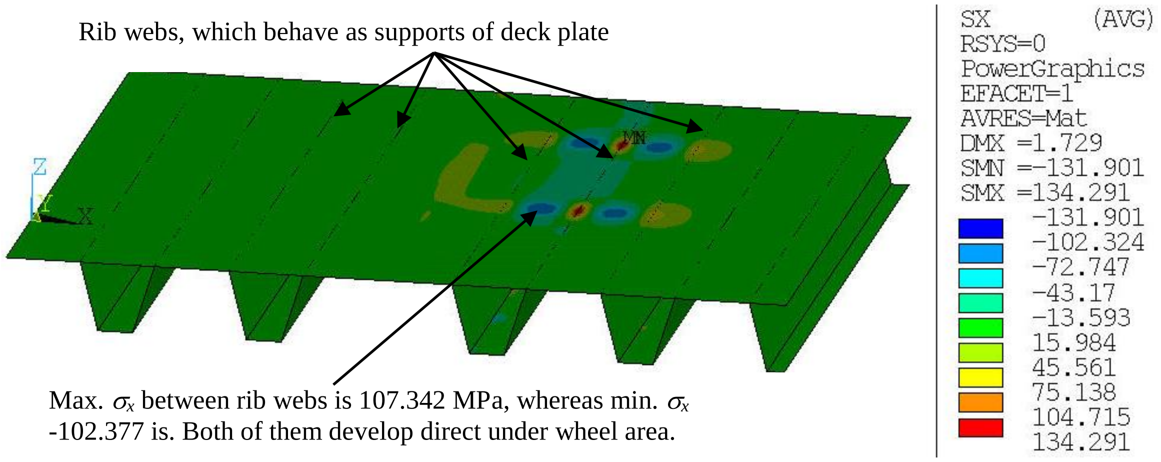


Figure 17. Stresses considered for fatigue calculation of deck plate between rib webs.

Table 7 tabulates the fatigue results , when deck plate thickness 12 mm (BB), 14 mm (DBD14) and 16 mm (DBD16) is.

Table 7. Fatigue lives of deck plate between rib webs as per deck plate thickness.

Simulation's name	Max. Stress	Min. Stress	Δ $\sigma_{E,2}$ (MPa)	N_i	Fatigue life (year) as per			
					Design Traffic Category 1	Design Traffic Category 2	Design Traffic Category 3	Design Traffic Category 4
BB	107.342	102.377	209.719	868 189	0.43	1.74	6.95	17.36
DBD14	84.322	80.74	165.062	1 780 682	0.89	3.56	14.25	35.61
DBD16	69.226	66.492	135.718	3 203 430	1.60	6.41	25.63	64.07

It is seen from Figure 18 and Figure 19 that increasing deck plate thickness results in higher fatigue lives of this structural part, nevertheless never supplies the required service life, 100 years. Reducing rib span shall be a remedy for supplying the required service life of 100 years.

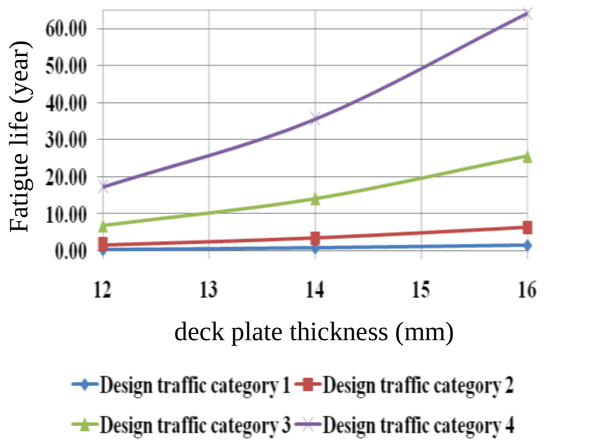


Figure 18. Fatigue lives of deck plate part between rib webs as to thickness and design traffic category.

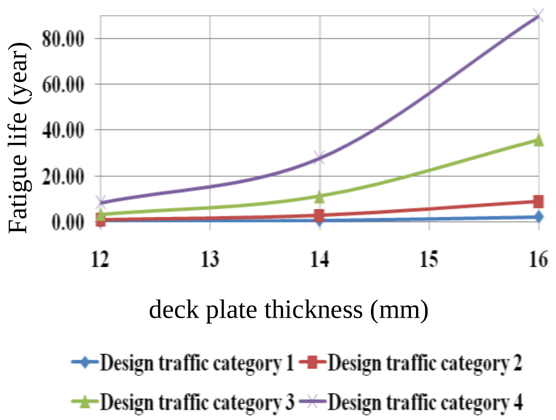


Figure 19. Fatigue lives of deck plate part on rib web as to thickness and design traffic category.

3.5. Redesign of Bridge Providing Necessary Fatigue Life

Because all of the structural parts of bridge do not simultaneously provide 100 years even for design traffic category 4, new designs of orthotropic bridge are required to be evaluated. Subsequently, two redesigns are considered according to the dimensions given in Table 8. Fatigue calculations of redesigned orthotropic deck structure are given in Table 9.

Table 8. Changed dimensions in FE- analyses, Redesign- 1 and -2.

Dimension definition	Redesign- 1	Redesign- 2
Cross- beam span (m)	3	2
Rib span (mm)	100	150
Deck plate thickness (mm)	18	18
Cross- beam web thickness (mm)	16	16

Table 9. Fatigue lives of redesigned bridge structures.

Simulation Name	Structural Part	$\Delta\ \sigma_{E,2}$ (MPa)	N_i	Fatigue Life (year) as per			
				Design Traffic Category 1	Design Traffic Category 2	Design Traffic Category 3	Design Traffic Category 4
Redesign 1	Cross- beam	10.774	∞				
	Rib	16.616	∞				
	Weld	9.251	∞				
	Deck plate between rib webs	34.225	∞				
	Deck plate on rib webs	77.04	40 388 858	20.19	80.78	**323.11**	**807.78**
Redesign 2	Cross- beam	6.951	∞				
	Rib	51.162	5 423 119	2.71	10.85	43.38	**108.46**
	Weld	16.373	∞				
	Deck plate between rib webs	105.94	8 214 847	4.11	16.43	65.72	**164.30**
	Deck plate on rib webs	65.492	90 982 538	45.49	**181.97**	**727.86**	**1 819.65**

In the first redesign of the bridge stress ranges appeared in cross- beam web, longitudinal stringer and weld connecting deck plate to rib are below cut- off limit. However stress range developed in deck plate part resting on rib web enforced bridge to be classified in design traffic category 3. If deck plate thickness is increased, design traffic category of bridge can be increased to 2 and 1 without changing other dimensions of bridge. Second redesign of bridge has a fatigue life of 108.46 years, when it is used as to design traffic category 4. The comparison of dimensions and fatigue lives of separate structural parts between Redesign- 1 and -2 indicates that decreasing rib span is more effective than decreasing cross- beam span to increase fatigue life of structure.

IV. CONCLUSION & FUTURE SCOPE

It is numerically proven in this study that, there exists an important dimensional change, whether a bridge is designed only as to yield stress or as to desired fatigue life under foreseen design traffic category. It is recommended to determine first rib span and then cross- beam span, when a bridge is designed for a desired fatigue life under foreseen design traffic category. Subsequently, thicknesses of deck plate and cross- beam web shall be chosen appropriately. The height of longitudinal stringer may be a parameter to reduce the stress range in ribs and weld connecting deck plate to rib. Types and thickness of the wearing courses, which disperse wheel load on deck plate, might be a solution to reduce stress ranges developed in structural parts. These will be the research subjects of the author in the future work.

REFERENCES

[1]. Jong, F.B.P. de (2007) *Renovation Techniques for Fatigue Cracked Orthotropic Steel Bridge Decks*, Dissertation, Technical University Delft.
[2]. Honshu Shikoku Bridge Authority (2005) www.hsba.go.jp, 17-01-2005
[3]. Fujino, Y. & Yoshida, Y., (2002) "Wind-Induced Vibration and Control of Trans-Tokyo Bay Crossing Bridge", *Journal of Structural Engineering*, pp1012-1025.
[4]. Virlogeux, M. (2004) "The Viaduct over the River Tarn", Conference Proceedings Steelbridge 2004 OTUA Paris, pp145-164.
[5]. Klöppel, K. & Roos, E., (1960) "Statische Versuche und Dauerversuche zur Frage der Bemessung von Flachblechen in orthotropen Platten", Der Stahlbau, Vol. 29
[6]. Troitsky, M.S. (1967) Orthotropic Bridges - Theory and Design, The James Lincoln Arc Welding Foundation, Cleveland, Ohio, United States of America.
[7]. Dowling, P.J., (1971) "The Behaviour of Orthotropic Steel Deck Bridges", Conference, University of Cardiff, United Kingdom.
[8]. Beales, C, (1979) "Severn/Wye Bridge Project - Tests on Crossbeam Joints", Working paper WP/BD 44, Transport Research Laboratory, Crowthorne, Berkshire, United Kingdom. (Confidential)
[9]. Fisher, J.W. (1977) Bridge Fatigue Guide, Design and Details, American Institute of Steel Construction Inc., New York, United States of America.
[10]. Kolstein, M.H. & Back, J.de (1989) Measurement and Interpretation of Dynamic Loads on Bridges, Phase 3, Fatigue Strength of Orthotropic Steel Decks, Part 2, Trough to Crossbeam Connections, Research sponsored by European Community of Steel and Coal, Delft University of Technology, Faculty of Civil Engineering, The Netherlands
[11]. Bruls, A., Caramelli, S., Cuninghame, J.R., Jacob, B., Kolstein, M.H., Lehrke, H.P., Pautremat, E. Le (1997) Measurement and Interpretation of Dynamic Loads on Bridges, Phase 4, Fatigue Strength of Orthotropic Steel Decks, Research sponsored by European Community of Steel and Coal, Report Delft University of Technology, Faculty of Civil Engineering, The Netherlands.
[12]. European Committee for Standardization , prEN 1993- 1- 9 Design of Steel Structures: Fatigue, Brussel, 2003
[13]. Swanson Analysis System, ANSYS: User Manuals, USA, 2010.
[14]. Deutsches Institut für Normung, DIN FB 103: Stahlbrücken, Beuth Press, Berlin, 2003.
[15]. Huurman et.al., "3D-FEM for the estimation of the behaviour of asphaltic surfacings on orthotropic steel deck bridges", 3rd International Symposium on 3D Finite Element for Pavement Analysis, Design & Research, Amsterdam, 2002.
[16]. Fettahoglu Abdullah & Bekiroglu, Serkan, "Effect of Kinematic Hardening in Stress Calculations", Advanced in Civil Engineering, Ankara, Turkey, 2012.
[17]. Fettahoglu Abdullah, "Effect of Deck Plate Thickness on the Structural Behaviour of Steel Orthotropic Highway Bridges", Advanced in Civil Engineering, Ankara, Turkey, 2012.
[18]. Deutsches Institut für Normung, ENV 1991- 3 Grundlagen der Tragwerksplanung und Einwirkungen auf Tragwerke, Teil 3: Verkehrslasten auf Brücken, Berlin, 1995.

AUTHOR

Abdullah Fettahoğlu is a Deputy Site Chief at the construction of 51 MW POLAT- 1 Coal Fired Thermal Power Plant Construction, Kutahya, Turkey. He worked in Trabzon Chamber of Commerce & Industry as a project coordinator, in ENKA and Saudi Arabian Baytur as QA/ QC engineer, in Ferchau (Airbus first supplier) as stress engineer, in Istanbul University as teaching and research assistant. He now makes his Ph.D. in Yildiz Technical University. He obtained his M.Sc. and B.Sc. from Istanbul Technical University Civil Engineering Department. His keen area is modelling structures using FEM, especially ANSYS, steel structures, quality control & assurance, tunnelling methods and construction techniques. He is a member of Turkish Civil Engineering Chamber and Turkish Tunnelling Society.