Fatigue durability of trough rib to deck plate welded detail of some orthotropic steel decks

Ya Samol*, Kentaro Yamada**, Toshiyuki Ishikawa***

* Dr. of Eng., Dept. of Eng., Takigami Steel Construction, 1-1, Shinmei-cho, Handa-shi, Aichi-Ken, 475-0826 ** Ph.D., Professor, Dept. of Environmental Eng. and Architecture, Nagoya University, Furo-cho, Chikusa-ku, Nagoya 464-8603 *** Dr. of Eng., Assistant Professor, Dept. of Environmental Eng. and Architecture, Nagoya University, Furo-cho, Chikusa-ku, Nagoya 464-8603

In this study, a number of orthotropic steel deck models are studied on their trough rib to deck welded joint at span center with regard to root-deck and root-weld fatigue cracks. It is found that increase in deck plate thickness can prolong fatigue lives of the joints regard to both fatigue cracks. Wheel position has also large influence on fatigue life of the joint. The wheel located between the trough ribs gives less damage to the joint. The longer span of trough rib tends to be more venerable to fatigue than the shorter one for root weld crack, but asphalt pavement of 75mm or 100mm thick makes not much difference in increasing in fatigue lives of the joint when load distribution is considered for half period of the year. *Keywords: Orthotropic steel deck, fatigue, durability, fillet weld, trough to deck welded detail*

1. INTRODUCTION

Most orthotropic steel decks (OSDs) with closed ribs in Japan consist of a deck plate of 12 mm thick and trough ribs of 6 or 8 mm thick with asphalt pavement of 60-80 mm thick. After some years in service fatigue cracks have been reported in the welded connections between trough ribs and deck plates (called herein as rib-to-deck joints)^{1),2)}, as shown in Fig.1. These fatigue cracks have received much attention because they have initiated from the hidden location, i.e. weld root, and propagated through deck plate (root-deck crack) or weld metal (root-weld crack) which directly support the traffic. Many studies have been carried so far to investigate the causes of fatigue cracks³), inspection methods and their applicability in the field⁴), repair/retrofit methods for existing OSDs. Re-decking with steel fiber reinforced concrete (SFRC)⁵⁾ and using large-sized trough ribs and thicker deck plates⁶⁾ are two possible solutions to be proposed.

Although quite a number fatigue tests on large-scale specimens were carried out at various organizations⁷⁾⁻¹², there have been few researches on fatigue durability evaluation by using fatigue test results with small-scale specimens in a combination with stress analyses. This study addresses this issue and also examines some OSD models which may have higher fatigue performances. Note that in the fatigue design

recommendation by Japan Road Association $(JRA)^{13}$, the fatigue evaluation is made with the criterion of structural detailing. That is the fatigue evaluation is not necessary once ODS details meet the required provisions.

Large number of heavy trucks including the overloaded ones is often cited as the main cause of fatigue crack. However, small plate bending rigidity of deck plate is also a main factor too³⁾. Recently, there has been a tendency to use thicker plates for new OSDs as preventive measures to avoid fatigue problem. Note that a deck plate of at least 14 mm thick is often used for OSD in common practice in the U.S.A¹⁴⁾. In an analytical study on fatigue durability of rib-to-deck joints with regards to root-deck crack¹⁵⁾, in which the fatigue strengths were obtained with one-millimeter stress method¹⁶⁾ and stress ranges were obtained with finite element analyses (FEA), we found that the use of thicker plate (say 16 mm) leads to decrease in stress range and contributes to prolong the fatigue life of rib-to-deck joint



Fig.1 Fatigue cracks in trough to deck welded detail

although it may contribute to a decrease in the fatigue strength of the joint. Experimental studies were also conducted to investigate the fatigue strengths of rib-to-deck welded details with regard to root-deck crack¹⁷⁾⁻¹⁹ and root-weld crack²⁰. Such fatigue strengths are basic for fatigue durability evaluation and they will be used in this study to evaluate the fatigue durability of some OSD models, in which stress ranges are obtained by finite element analyses (FEA). Fatigue cracks may occur at span center of trough ribs or at intersections between deck plate and crossbeam, but the study is focused only at span center of trough rib because the fatigue life of the joint at both locations may not be much different as shown in our previous study¹⁵.

2. FATIGUE STRENGTHS OF RIB-TO-DECK WELDED DETAILS

Fatigue strengths (S-N curves) of rib-to-deck welded joints are not available in the current fatigue design recommendations by JRA¹³ and Japanese Society of Steel Construction (JSSC)²¹). To obtain fatigue strengths, analytical methods such as one-millimeter stress method¹⁶ may be used, as presented in previous study¹⁵. These analytical estimations should be verified with fatigue test results if the fatigue test data are available. Experimental studies such as plate fatigue tests with small-scale specimens¹⁷⁾⁻¹⁹ can also be used to obtain the basic fatigue test data. With plate bending fatigue testing machines²², a total of 80 fatigue tests of that kind²³ were conducted at Nagoya University until early 2009.



2.1 S-N curves for root-deck crack

Instead of testing the full-scale specimens, simplified small-scale specimens representing the welded connection between trough ribs to deck plate of OSD were fatigue-tested. With a width of 300 mm, the simplified specimens consisted of a main plate (deck plate) and an attachment. The attachment was a rib plate in all test specimens except four specimens which were with a full-sized trough rib. The thickness of deck plate (t_d) was 12, 14 or 16 mm. The thickness of attachment (t_r) was 6 or 8 mm thick. The more details of test specimens, their consideration with regard to actual OSD and fatigue set-up can be found elsewhere^{15,17)-19}.

An example of test specimen and test set-up is shown in Fig.2. In the test-set up, the main plate was loaded with a fluctuating plate bending stress and the rib carried no load. This condition may be different from the actual structure of OSD of which the rib has closed section then the rib plate carries load. With analytical approach, it was shown that the bending stress in rib plate contributes a small damaging effect to the root-deck crack¹⁵, and with experimental approach it was shown that the tests specimens with attachment as a rib plate and those with attachment as a full-sized trough rib had similar fatigue lives¹⁸. From both points of view, the force in rib plate may be simplified with regard to root-deck crack, and thus the rib is free in the test set-up.

The fatigue test data of the specimens which failed by fatigue tests are plotted in Fig.3. The vertical axis is the stress range at weld root, determined by interpolation/extrapolation of the stress ranges measured with uniaxial electrical strain gages attached near weld lines. The horizontal axis is the number of cycles to failure. In this study, it is taken at the termination of fatigue test when fatigue crack has penetrated the deck plate.

By regression analysis assuming the slope of S-N diagram as 3, the fatigue strengths at 2×10^6 cycles were determined and are listed in Table 1. Fatigue test results showed that fatigue strengths of the rib-to-deck joints of OSD tend to decrease with an increase in the thickness of deck plate. Previous study¹⁵ with one-millimeter stress method also showed similar tendency. Comparison between test results and analytical method shows the decrease in fatigue strength obtained by fatigue tests and that obtained by analytical method is different; the decrease by fatigue test results is larger²³. The difference is not clearly known and it may be necessary to conduct more study on this subject in the future. In this study, the fatigue strengths obtained by fatigue tests are used for fatigue durability evaluations.

Table 1 Fatigue strengths at 2×10^6 cycles for root-deck crack (in MPa)

		(
	Welded details (thickness in mm)			
	$t_d = 12$, $t_r = 6$ or 8	$t_d = 14, t_r = 8$	$t_d = 16, t_r = 8$	
Mean-2s	132	120	93	
Mean	185	153	111	
Mean+2s	258	194	133	

2.2 S-N curves for root-weld crack

To study the fatigue strength of root-weld crack, which initiated at weld root and propagated through weld throat, a number of simplified test specimens as shown in Fig.4 were fatigue-tested using plate bending machine²⁰⁾. With a width of 500 mm, each test specimen consisted of a main plate (deck plate) and an attachment (a rib). The thickness of deck plate (t_d) was 12, 14 and 16 mm. The thickness of rib (t_r) was 6 or 8 mm. The weld penetration is 30%, 75%, 80% of rib thickness and weld melt-through (WMT).



Fig.4 Fatigue test specimen for root-weld crack



Fig.5 Fatigue test data for root-weld crack

 Table 2 Fatigue strengths of the rib-to-deck welded joint with regard to root-weld crack (unit: MPa)

	Mean-2s	Mean	Mean+2s				
Fatigue strength	93	146	231				

In the test set-up as shown in Fig.4, the deck plate was fixed to a test frame and the rib was loaded with a fluctuating plate bending stress in such a way to create a fully tensile fluctuating stress condition at weld root. Some of fatigue tests were continued over 2×10^7 cycles, which is much longer than some fatigue tests which had been carried in the past²⁴⁾⁻²⁶⁾.

The extent of weld penetration and rib thickness are of no significance once the test data are plotted in term of stress ranges in weld throat²⁰⁾. Thus, it is not necessary to separate the group of test data. For easy in identifying the specimens, the test data are grouped based the thickness of deck plate and rib plate in S-N diagram as shown in Fig.5. The vertical axis is the stress range in weld throat which are computed from the stress ranges measured with uniaxial electrical resistance strain gages attached on the rib and measured average thickness of weld throat. The horizontal axis is the number of cycle when fatigue crack has penetrated the weld metal.

The fatigue strength at 2×10^6 cycles in term of stress range in weld throat was determined with the slope of *m*=3 for S-N are listed in Table 2.

3. STRESS RANGES COMPUTED BY FEA

3.1 FEM models of OSD

The geometries and the FEM models of OSD are shown in Fig.6. Two types of OSD models were studied, denoted as standard-deck models with rib spans of 2.24 m and modified standard-deck models with rib spans twice longer (4.48 m). In Table 3, which gives more details on FEM, the letter "L" is referred to standard-deck model and "2L" is referred to modified standard-deck model. The numbers following "L" or "2L" are the thickness of deck plate. The span of trough rib is around 2.5 m for most OSD bridges; however, there are some OSD bridges which were fabricated with rib spans around 4 m or over. For easy in studying, the difference in span length is taken as twice.

In the fatigue design recommendation by JRA which was published in 2002¹³, the span of trough rib is set below 2.5 m for fatigue evaluation by structural detailing.

The FEM models were created with shell elements (SHELL4) with membrane and bending capacities of the software package COSMOS/M 2.9^{27} . The minimum mesh sizes of 10 mm × 10 mm were used near the objective joint. The fillet weld geometries were not modeled. The Young's modulus was 2×10^5 MPa and the Poisson's ratio was 0.3.

As for boundary condition for standard-deck model, as shown in Fig.6(b), the displacements in vertical direction were constrained along the edges and lower ends crossbeams. The nodes at the intersections between deck plate and crossbeam were constrained in all directions as these locations are assumed to have large rigidity. For modified standard-deck model, as shown in Fig.6(c), only half side was modeled by taking symmetry above crossbeam. The displacements in vertical direction were constrained along the edges. The displacements in all directions were constrained at the intersections between deck plate and crossbeam as these points are assumed to have large rigidity, and at the lower ends of crossbeams as these points appear to have large rigidity due to higher crossbeams.



(a) Geometries of the FEM models



(c) Modified standard-deck model Fig.6 FEM models of orthotropic steel decks

Table 3 Details of FEM mo	dels ((in mm)	
---------------------------	--------	---------	--

Model	+	+	Asphalt	Rib	Fig6
Designation	ι_d	ι_r	pavement	span	rig.0
L12	12	8	0,75,100	2240	(b)
L14	14	8	0,75,100	2240	(b)
L16	16	8	0,75,100	2240	(b)
2L12	12	8	0,75,100	4480	(c)
2L14	14	8	0,40,75,100	4480	(c)
2L16	16	8	0,40,75,100	4480	(c)

In real situation, the wheels of vehicles run on the asphalt pavement at different positions in the transverse direction of the OSD bridges. The distribution of position of wheel is assumed to be a normal distribution with a mean position and a standard deviation of 350 mm²⁸⁾. Five positions of wheel, as shown in Fig.7, were applied to deck plate in the transverse direction to study its effect on the rib-to-deck joint. Three wheel positions were applied near the center of rib span which is the point of interest in this study in order to establish an influence line for computing stress range.

In the analysis, the wheel load was applied to the deck plate in form of uniform pressure which is equal to the unit load divided by a contact area. The size of contact area varies with whether the load distribution due to the rigidity of asphalt pavement is considered or not. If load distribution is not considered (designated as no-load-distribution case), the size of contact area is 200 mm \times 500 mm. Such a size of contact area is also found elsewhere²⁹⁾. Note that the size of contract area may be larger than this value as it varies with many parameters such as the air pressure of the wheel, the load the wheel bears³³⁾.



Fig.8 Nominal stress for fatigue evaluation

If load distribution is considered (designated as with-load-distribution case), the size of contact area is assumed to increase by spreading 45° in all directions from the surface of asphalt pavement toward the surface of deck plate³⁰). By this way, a thicker asphalt pavement leads to more spread of contract pressure and is expected to contribute to smaller stress and stress range. The case where there is no asphalt pavement may be treated as the case of no load distribution. In summer when the temperature of asphalt pavement becomes high (say 40°C or more), the rigidity of asphalt pavement looses and then load distribution vanishes; this case is also treated as the case of no load distribution due to the rigidity of asphalt pavement is not considered in the design²⁹.

By using the concept of load distribution, it is apparent that the effect of the thickness of asphalt pavement on fatigue durability of the rib-to-deck joint can be studied by just changing the size of contract area. For this purpose, the thickness of asphalt pavement was selected as 75 and 100 mm in all OSD models except some modified standard-deck models which were with 40 mm.

3.2 Stresses computed by FEA

(1) Definition of nominal stress for root-deck crack

The nominal stress is defined as the stress at 10 mm away from the intersection and inside the deck plate¹⁵, as shown in Fig.8.

The computed stress ranges are plotted in Fig.9. The maximum stress ranges were obtained with Pass-3. When the

wheel load is far from the objective joint (i.e. Pass-1' or 2'), the stress ranges become smaller, thus such positions have smaller damaging effects on the objective joint.



Fig.9 Stress ranges under different load position



Fig.10 Stress ranges when changing rib span



Fig.11 Stress ranges under Pass-3

A slight change in stress range is observed with a change in the rib span, as shown in Fig.10. It is because the direction of the stresses of interest is perpendicular to the direction of trough ribs

As the thickness of deck plate increases, the stress range decreases due to the increase in plate rigidity of deck plate, as shown in Fig.11.

(2) Stress range for root-weld crack

1) Stress in rib directly obtained by FEA

The stress range which is most related to the root-weld crack

is the stress range in weld throat and at the weld root because the fatigue crack initiated from weld root and propagated along weld throat.

Because shell elements are used for creating FEM models of OSD, the weld geometries or weld penetration are not considered. Thus, it is necessary to find a way for computing stress range in weld throat from the stress directly obtained with FEA with shell models.

The stress at the node 6 mm away from the intersection (σ_{r_in}) , as shown in Fig.12, was used to compute the stress range in weld throat. The position of node was selected by assuming that weld root is located around 6 mm away from the intersection.

Under wheel load, bending stress is dominant compared to membrane stress in the deck plate¹⁵⁾. But, bending stress is not clearly dominant in the rib plate because the membrane stress also has its magnitude, as shown in Fig.13.



Fig.12 The stress used for compute stress in weld throat



Fig.13 Distribution of stresses in the transverse direction



Fig.14 Distribution of $\sigma_{r in}$ in the transverse direction

The distribution of the stresses (σ_{r_in}) in the transverse direction of the deck is shown in Fig.14. With Pass-3 where wheel load locates directly above the objective joint, the membrane stresses (σ_m) are maximum because the rib plate acts as a support. If the wheel load is applied on the joint next to the

objective joint, the membrane stress are nearly zero. The bending stresses (σ_{rb}) are nearly zero when the wheel load is applied on the objective joint. They are apparently maximum under Pass-2 or minimum under Pass-1 (in no-load-distribution case).

2) Stresses in weld thraot

Two methods are considered in computing the stress range in weld throat from the stress directly obtained by FEA ($\sigma_{r_{in}}$), as schematically shown in Fig.15.

Method 1: The stress located on the interior face of rib plate is assumed as if it were bending stress, then is further converted to bending moment, M'; that is $\sigma_{r_in}=6M'/t_r^2$ for a unit of width. The stress range in throat is given by Eq.(1).

$$\Delta \sigma_{throat} = \Delta \sigma_{r in} \times (t_r / t_{throat})^2 \tag{1}$$

where t_r and t_{dwoat} are the thickness of rib and weld throat, respectively.

Method 2: The bending stress (σ_{tb}) is converted to bending moment, *M*, and the membrane stress (σ_{tm}) is converted to axial force, *P*, acting on the cross-section of rib plate. Once the weld geometries are assumed, both *M* and *P* are applied to the cross-section of weld throat. For a unit of width, the stress range in rib is given by Eq.(2).

$$\Delta \sigma_{r \ in} = \Delta (\sigma_{rb} + \sigma_{rm}) \tag{2}$$

where $\sigma_{rb}=6M/t_r^2$ and $\sigma_{rm}=P/t_r$; t_r is thickness of rib plate.

For a unit of width, the stress range in weld throat is given by Eq.(3)

$$\Delta \sigma_{throat} = \Delta \left[\sigma_{rb} \left(\frac{t_r}{t_{throat}} \right)^2 + \sigma_{rm} \left(\frac{t_r}{t_{throat}} \right) \right]$$
(3)

Method 1 is rather simplified and it was used a previous study²⁰⁾. Method 2 is more complicated and is expected to give a closer stress range in weld throat because it accounts for separation of bending stress and membrane stress in weld throat. Note that Method 2 is more simplified than the method presented by Maddox for computing stress range in weld throat for test specimens²⁴⁾.

The stress ranges in weld throat from both proposed methods are compared for their agreement as shown in Fig.16 with a thickness of weld throat of 7.1 mm which was obtained by assuming that weld leg was 6 mm and weld penetration was 75% of rib thickness. As shown in Fig.16, both methods give almost similar results, thus they agree with each other well. Any of them may be used, but because of simplicity, Method 1 is more appropriate in practice and then is selected in this study.

Pass-3 which is a critical position as it induces maximum stress range in the objective joint for the case of root-deck (see Figs.9 and 10). However, this position of wheel is apparently not a critical position for the case of root-weld crack as it induces relatively smaller stress range compared to other positions of wheel, as shown in Fig.16. The stress ranges due to the positions of wheel on the joints adjacent to the objective joint (i.e. Pass-1' or -2') are larger.

The stress ranges in weld throat due to Pass-1 are maximum in the no-load-distribution case, but those due to Pass-1' are maximum for in the with-load-distribution case.

As shown in Fig.17, similar to the case of root-deck fatigue crack, the stress range in weld throat becomes smaller with an increase in deck plate thickness. This may be that thicker deck plate leads to larger rigidity of deck plate, and in turn resulting in small deformation in deck plate and rib.





Fig.16 Comparison stress range in weld throat by Method 1 and Method 2 (L12 model)



Fig.17 Stress range in weld throat under Pass-1

4. FATIGUE DURABILITY OF TROUGH RIB TO DECK WELDED JOINTS

4.1 Load conditions and assumptions

In this study, it is assumed that 1,000 double wheel load of 55 kN run daily on the OSD models of interest. Such a traffic condition corresponds to one of heavily loaded national highways in Japan³¹⁾.

The fatigue durability evaluation is carried out on the rib-to-deck joint with weld penetration of 75% of rib thickness, being the same as that specified by JRA¹³.

Note that in a usual fatigue consideration¹³, the design axle load of 100kN may be used and the number of cycles is obtained from the expected traffic of the OSD bridge of interest. In the fatigue evaluation, a number of correction factors are applied, of which the correction factor accounting for frequency of such load is small, i.e. 3% of the expected number of traffic.

4.2 Fatigue life under double wheel with no consideration of effect of transverse distribution of position of wheel

In this section, the fatigue lives of the joint are computed with no consideration of transverse position of wheel, that is wheel is assumed to always run at Pass-1, -2 or -3, separately. Such a condition may represent the case where the vehicles travel in the roads of narrow lanes or the case of large-scale fatigue tests with running wheel in laboratory.

(1) For root-deck fatigue crack

The computed fatigue lives of the rib-to-deck joint are plotted in Fig.18. Increase in span length of trough rib has less effect on the fatigue lives of the joint with regard to root-deck crack. That is, the fatigue lives are similar in both cases as the stress ranges do not change much, as shown in Fig.10.

In the no-load-distribution case, the fatigue lives of the joint are short. If the thickness of deck plate increases from 14 to 16 mm, the fatigue lives also increase but very small. In fact, the increase in thickness of deck plate contributes to a decrease in stress range which is a favorable factor, but at the same time it also contributes to a decrease in fatigue strength which is an unfavorable factor. But the decrease in stress range is more than the decrease in fatigue strength; thus the fatigue life is longer with the use of thicker plate. For example, from Table 1, for t_d from 12 to 14 mm, fatigue strength decreases from 132 to 120 MPa (around 10%). From Fig.11, comparing L12 to L14 models shows stress range decreases from 141 to 101 MPa (around 29%). As a result, fatigue life increases around twice $(=(0.9/0.71)^3 = 2.0)$.



Fig.18 Fatigue lives of the joint regarding to root-deck crack

The fatigue lives of the joint in the with-load-distribution case are longer than those in the no-load-distribution case, as expected from previous study¹⁹⁾. Thicker pavement tends to give longer fatigue life as more spread of contract area is obtained. The use of thick pavement in a combination with the use of thicker plates such as 14 or 16 mm can contribute to future prolong fatigue life of the rib-to-deck joint of OSD.

The fatigue lives with Pass-1 or -2 are longer than those with Pass-3. This is because the stress ranges due to Pass-1 or -2 are smaller than those due to Pass-3.

(2) Root-weld fatigue crack

As shown in Fig.19(a), the computed fatigue lives of the rib-to-deck joint with Pass-3 are relatively high because stress ranges due to this position of wheel are small (see Fig. 16). It seems that the rib-to-deck joint with regard to root-weld crack is less affected by Pass-3. However, if with regard to root-deck crack, the rib-to-deck joint is highly affected by Pass-3 as discussed earlier.

The computed fatigue lives of the objective joint with Pass-1' are relatively short as shown in Fig.19(b) because the stress ranges under this position are high (see Fig.16). By symmetry, if the position of wheel is located at Pass-3, the joint which would be highly affected by this position of wheel and should receive more attention is the joint adjacent to the objective joint as shown in Fig.20. It is more explanative by viewing deformed shapes under both positions of wheel as shown in Fig.21.



Fig.19 Fatigue lives of the joint failed in weld throat



Fig.20 Joint to be paid attention to when position of wheel is at Pass-3

The computed fatigue lives with Pass-1 are plotted in Fig.22. Comparison this figure with Fig.19(a) shows the fatigue lives with Pass-1 are shorter than those under Pass-3. Thus, it is said that Pass-1 is more affecting position of wheel in the case of root-weld crack.

The fatigue lives of rib-to-deck joints of the modified standard-deck models (2L12, 2L14 and 2L16 models) are shorter than those of the joints of the standard-deck models (L12, L14 and L16 models). This is because the stress ranges of the OSD models of longer ribs are higher as longer ribs would result in more torsional deformations in ribs. Thus, attention should be paid to root-weld crack when trough ribs are to be increased in span length. Note that increase in span length of the trough ribs has less influence on root-deck crack as discussed earlier.



(b) With Pass-1', rib has larger deformation Fig.21 Deformation of rib at the objective joint

Similar to the case of root-deck crack, an increase in deck plate thickness can also result in an increase in fatigue life of the joint with regard to root-weld crack, see Figs.18 and 19.

The amount of increase in fatigue life of the rib-to-deck joint with regard to both kinds of fatigue crack is nearly the same, and is around twice for an increase in thickness of deck plate 12 to 14 mm (L12, L14 models). In fact, a decrease in stress range for root-weld crack is smaller than a decrease in stress range for root-deck crack. For example, for a increase in thickness of deck plate from 12 to 14 mm, the stress range for root-deck crack

drops by 20% (with Pass-3), and the stress range for root-weld crack drops by 29% (with Pass-1). With the same increase in thickness of deck plate, the fatigue strength of the rib-to-deck detail drops by 10% for root-deck crack but it does not drop for root-weld crack. As a result, the fatigue durability increases roughly by double for an increase in thickness of deck plate from 12 to 14 mm.



Fig.22 Fatigue lives under Pass-1



Fig.23 Example of correction factors α_L

In the with-load-distribution case, the fatigue lives of the welded joints also increase because stress ranges also drop when the rigidity of asphalt pavement is considered by load distribution.

4.3 Fatigue lives with effect of transverse distribution of position of wheel

The way of consideration of the effect of distribution of position of wheel in transverse direction of OSD is the same as the way presented in previous study¹⁵⁾. The correction factor accounting for distribution of position of wheel, α_L , can be computed using Eq.(4).

$$\alpha_L = \sqrt[3]{\Sigma(\lambda_i^3 \times n_i) / \Sigma n_i} \tag{4}$$

where n_i is a frequency of occurrence corresponding to a transverse load distribution factor λ_i . The factor λ_i is a ratio of the stress range due to any position of wheel load to the stress range due to the mean position of wheel load. Both factors were taken the same as those presented the previous report¹⁵⁾. The distribution of position of wheel with a standard deviation of 350 mm²⁸⁾ is assumed and the mean position of wheel is at Pass-1, -2 or -3. Note that Eq.(4) can be found elsewhere³²⁾.

(1) For root-deck fatigue crack

An example of computed correction factors α_L is plotted in Fig.23. The values of α_L with Pass-3 are around 0.8 and those with Pass-1 or -2 are larger than unity. This is because the stress ranges with Pass-3 are large; when they are considered with smaller stress ranges, the damaging effects become smaller and then the correction factors α_L are less than unity. Different from Pass-3, Pass-1 and -2 give the values of α_L larger than unity.



Fig.25 Fatigue lives with mean position of wheel at Pass-3

The stress ranges corrected with α_L are plotted in Fig.24. Pass-1 or Pass-2 gives approximately equal values of stress ranges, but they are smaller than those due to Pass-3.

The fatigue lives of the rib-to-deck welded joint with the correction factors α_L assuming the mean position of wheel at

Pass-3 are plotted in Fig.25.

In the no-load-distribution case, the fatigue lives of the joint with α_L are longer than those without α_L , around twice, i.e. 8~23 year compared to 4~12 years for L12 model. Increase in thickness of deck plate to 14 mm can result in double increase in fatigue life of the joint to 17~35 years.

In the with-load-distribution case, fatigue life of the joint may reach 100 years with deck plate of 14 mm thick. This infers that the use of deck plate of 14 mm thick with permanent load distribution due to pavement of 100 mm thick may satisfy the current fatigue design life of 100 years under the assumed traffic condition which corresponds to one of heavily loaded national highways in Japan. With a deck plate 16 mm thick with permanent load distribution due to pavement of 75 mm thick, the rib-to-deck joint has fatigue life at least 85 years.

As shown in Fig.26, the fatigue lives with Pass-1 or -2 are similar and slightly more than those with Pass-3.

With load distribution and mean position of wheel at Pass-1, the fatigue lives of the joints of L12_75mm, 2L14_40mm, 2L16_40mm models are 54, 63 and 73 years, respectively. With Pass-2, the respective fatigue lives of the joints are 47, 51 and 57 years. This suggests that the modified standard-deck models be slightly better than the standard-deck model (L12_75mm model) with the mean position of wheel at Pass-1 or Pass-2.



Fig.26 Fatigue lives with the mean position at Pass-1 and -2



Fig.29 Fatigue lives with mean position of wheel at Pass-1

(2) Root-weld fatigue crack

With the same consideration as the case of root-deck crack, correction factors α_L in the case of root-weld cracks are plotted in Fig.27 and the corrected stress ranges are plotted in Fig.28. After being corrected with α_{L_2} the stress ranges with the mean position of wheel at Pass-1 are maximum; it thus appears that the mean position at Pass-1 is the most unfavorable position

because α_L accounts for the stress ranges due to Pass-1' which are also high. The corrected stress ranges due to Pass-3 become higher as the correction factor accounts for higher stress ranges due to Pass-1 and -2.



(b) Pass-2 Fig.30 Fatigue lives of the joints failed by root-weld crack as mean position of wheel at Pass-2 and -3

The fatigue lives of the rib-to-deck joint with the mean position of wheel at Pass-1 are plotted in Fig.29. Similar to the case of no consideration of α_L , an increase in thickness of deck plate contributes to increase in fatigue durability of the joint. For example, comparison between L12 0mm model and L14 0mm model reveals that an increase in thickness of thickness results in increase in fatigue life around twice from 10 to 20 years. Comparison between L14 0mm model and L14 75mm model shows a difference in fatigue life around three times, from 20 to 70 years, referring that fatigue life of the joint also increases with the with-load-distribution case. Comparison between L12 0mm and L14 75mm shows fatigue life of the rib-to-deck joint increases 6 times (from 10 to 60 years). Similar to the case of root-deck crack, in the case of root-weld crack the use of thick deck plate for OSD along with the pavement allowing load distribution contributes to prolong fatigue life.

Also shown in Fig.29, the modified standard-deck models

of which the rib spans are longer have shorter fatigue lives than the standard-deck models. Thus, attention should be paid to the root-weld crack for OSD decks with long spans.

The rib-to-deck joint has no problem with root-weld crack under the condition that the wheel always travels at Pass-3, attention should be paid to when the position of wheel is distributed in transverse direction of deck with the mean position at Pass-3, as show in Fig.30(a). Comparison between Fig.19(a) and Fig.30(a) shows that for 2L12 mode the fatigue life of the joint drops considerably from 80 years to 9 years in the no-load-distribution case. If the mean position of wheel is assumed at Pass-2, the fatigue lives of the joint are higher, the minimum is 35 years, see Fig.30(b). From these results, it seems that mean position of wheel at Pass-2 contributes less damage to the joint with regard to root-weld crack.



Fig.31 Yearly equivalent stress range corrected with α_L



Fig.32 Fatigue lives with mean position at Pass-3 and effect of temperature on asphalt pavement

4.4 Fatigue lives with effect of transverse position of wheel and effect of temperature on asphalt pavement

In semi-tropical climate area, asphalt pavement is known to be soft in summer and stiff in winter. Thus, load distribution is assumed for six months and is not for another six months of the year. The effect of temperature on asphalt pavement may be considered with a yearly equivalent stress range $\Delta \sigma_{eq}$, which expressed by Eq.(5).

$$\Delta \sigma_{eq} = \sqrt[3]{(\sigma_{non}^3 + \sigma_{with}^3)/2}$$
 (5)

where $\Delta \sigma_{non}$ is stress range during non-load-distribution period, and $\Delta \sigma_{with}$ is stress range during period of with-load-distribution.



Fig.33 Fatigue lives with mean position at Pass-1 or Pass-2 and effect of temperature on asphalt pavement

In order to consider both effect of transverse position of wheel position in transverse direction and effect of temperature on asphalt pavement at the same time, correction factors α_L are first applied and then Eq.(5) is applied.

(1) For root-deck fatigue crack

The yearly stress ranges are plotted in Fig.31 for example with the different mean position of wheel. Even though, the stress ranges with Pass-3 are the highest, the difference in stress ranges with other mean positions becomes smaller (if compared to Figs.9 and 24). The difference in stress range due to different thickness of asphalt pavement becomes smaller too.

The evaluated fatigue lives of the joint are plotted in Fig.32, with mean position of wheel at Pass-3. The fatigue lives in the no-load-distribution case (i.e. mean, 0) are also plotted for comparison.

The difference in fatigue life between asphalt pavement of

75 and 100 mm thick is very small. It seems an increase in asphalt pavement from 75 to 100 mm thick is of less significance in the current conditions. Taking the lower-bound values, short fatigue lives are obtained, around 15 years for OSD model with deck plate of 12 mm thick. With a thicker deck plate like 14 or 16 mm, the fatigue lives increase but they are not much different (around 30 years). Comparison among the three models of OSD, i.e. L12_75mm, 2L14_40mm and 2L16_40mm, the last two OSD models have longer fatigue lives. Thus, the use of thicker deck plate, longer rib span and thinner asphalt pavement for OSD seems to be more interesting than the current practice where OSD uses deck plate of 12 mm thick with asphalt pavement of 75 mm thick.

With the mean position of wheel is allocated at Pass-1 or Pass-2, the fatigue lives of the rib-to-deck joint are plotted in Fig.33. The fatigue lives in both cases are very comparable but are slightly more than those with the mean position of wheel at Pass-3. Thus, it is suggested that avoiding Pass-3 as mean position of wheel on the rib-to-deck joint of the interest may partially contributes to prolog fatigue lives of the joint. Note that Pass-3 is the position of wheel which is located directly on the joint of interest.

(2) Root-weld fatigue crack

With the mean position of wheel assumed at Pass-3, the fatigue lives are plotted in Fig.34. With asphalt pavement of 75 or 100 mm thick, the fatigues lives are approximately similar. In addition, an increase in deck plate thickness results in increase fatigue lives of the joint.



Fig.34 Fatigue lives with consideration effect of temperature on pavement and with mean position wheel at Pass-3

For standard-deck models, comparison between Fig.32 and Fig.34 shows that fatigue lives of the rib-to-deck joint with regard to root-weld crack are longer than those with regard to root-deck crack. Thus, it may be said that the likelihood of having root-weld crack in the joint is less than that of root-deck crack for OSD models. However, with modified standard-deck models which have longer rib spans, the fatigue lives with

regard to both fatigue cracks are nearly similar. This suggests that the same possibility of both cracks exist for OSD with long rib spans. Note that this result is obtained with weld penetration of 75% of rib thickness.



Fig.35 Fatigue lives with effect of temperature on pavement and with mean wheel position at Pass-1 and -2

The fatigue lives with the mean position at Pass-1 and -2 are plotted in Fig.35. With Pass-1 as mean position, fatigue lives become short for modified standard-deck models. This means attentions should be paid to when span length are to be increased. With Pass-2 as mean position, the fatigue lives of the joint are higher for both kinds of orthotropic steel deck models. When regarding to root-deck crack, fatigue lives are also longer with Pass-2 as mean position. It seems that with setting lane to have a mean running wheel position at Pass-2 may helps prolong fatigue lives of the joints regarding to both fatigue cracks.

5. SUMMARY

In this study, a number of orthotropic steel deck models are studied on their trough rib to deck welded joint at span center with regard to root-deck and root-weld fatigue cracks. It is found that increase in deck plate thickness can prolong fatigue lives of the joints regard to both fatigue cracks. Wheel position has also great effect of fatigue life of the joint. For narrow lane where the wheel may run on approximate on the same line, Pass-3 is most damaging regard to root-deck crack, and Pass-1 is for root-weld crack. The position of wheel which gives less damaging to the rib-to-deck joint is Pass-2, the one which is between the trough ribs. The longer span trough rib tends to be more venerable to fatigue than the shorter one. Consideration of load distribution of asphalt pavement for half-year for a year period, fatigue lives of the joint are longer. Between 75mm or 100mm thick, there is no much difference. This means maintaining of asphalt pavement in good condition can partially help prolong fatigue lives of the joint. If pavements like SFRC which is more rigid than asphalt pavement, which can provide load distribution in the fully whole year, fatigue lives of the rib-to-deck welded joint are comparatively high. If such pavements are used in combination with thicker deck plate like 16 mm for new orthotropic steel deck, fatigue problem may not be a problem as fatigue lives are high as shown in Fig.25 for the location at span center. However, further studies are needed to investigate the fatigue problem at the intersection between deck plate and crossbeam and trough ribs.

ACKNOWLEDGEMENT:

The first author gratefully acknowledges the scholarship granted by the Japanese Government during the period of research at Nagoya University.

References

- Yuge, T., Machida, F., Morikawa, H., Miki, T., Kamiki, T., Masui, T., Analysis of fatigue damage patterns in orthotropic steel deck of Tokyo Metropolitan Expressways, Orthotropic Bridge Conference, Sacramento, pp.531-542, CD-ROM, 2004.
- Takada Y., Hirano, T., Sakano, M., Report on conditions of fatigue damage of orthotropic steel deck of Hanshin Expressway, Proc. of 61th JSCE Annual meeting, Division 1., pp.1067-1068, CD-ROM, 2006. (in Japanese)
- Miki, C., Suganuma, H., Tomizawa, T., Machida, F., Case study of fatigue damage in orthotropic steel bridge deck, *J. of Struct. Mech. and Earthquake Eng.*, JSCE, No.801/I-73, pp.57-69, 2005. (in Japanese)
- Murakoshi, J., Arima, N., Fujiki, O., Basic investigation on applicability of the ultrasonic testing method against fatigue cracks in orthotropic steel deck, *Civil Eng. J.*, Public Works Research Institute, pp.34-39, 2004. (in Japanese)
- Ono, S., Shimozato, T., Masui, T., Machida, F., Miki, C., Retrofitting method for existing orthotropic steel deck, *J. Struct. Eng.*, JSCE, No.801/I-73, pp.213-226, 2005. (in Japanese)
- Mizuguchi, K., Yamada, K., Iwasaki, M., Inokuchi, S.: Rationalized steel deck structure and large model test for

developing new type of structure, Orthotropic Bridge Conference, Sacramento, pp.675-688, CD-ROM, 2004.

- Kagara, N., Irube, T., Hosomi, N., Nagasaki, Y., Tanaka, M., Mori, T., Proposal of reinforcing works for fatigue damage in orthotropic steel deck, Tokotsu Technical Report, No.51, pp. 6-31, 2006. (in Japanese)
- Shimozato, T., Kamiki, T., Inaba, N., Tomita, Y., Ono, S., Fatigue test with moving load on orthotropic steel deck, Proc. of 60th JSCE Annual meeting, Division 1., pp.795-796, CD-ROM, 2005. (in Japanese)
- 9) Shimozato, T., Kamiki, T., Inaba, N., Tomita, Y., Ono, S., Fatigue test on retrofitted crack in trough to deck plate of orthotropic steel deck, Proc. of 61th JSCE Annual meeting, Division 1., pp.1095-1096, CD-ROM, 2006. (in Japanese)
- 10)Inokuchi, S., Kawabata, A., Hironaka, O., Saito, S., Fatigue test by the wheel loading machine intended for weld of the trough rib to deck plate of orthotropic decks (Second report), Proc. of 61th JSCE Annual meeting, Division 1., pp.1081-1082, CD-ROM, 2006. (in Japanese)
- 11)Arima, Y., Murakoshi, J., Simulation of propagating crack in orthotropic steel deck plate by fatigue test with moving load, Proc. of 61th JSCE Annual meeting, Division 1., pp.1083-1084, CD-ROM, 2006. (in Japanese)
- 12)Oka, T., Aiba, M., Murakoshi, J., Arima, Y., Hayashi, N., Kameyama, T., Fatigue test with moving load on cracked orthotropic steel deck reinforced by stringer or by filling the trough ribs, Proc. of 61th JSCE Annual meeting, Division 1., pp.1107-1108, CD-ROM, 2006. (in Japanese)
- 13)Fatigue Design Guideline for Steel Highway Bridges, Japan Road Association, 2002. (in Japanese)
- 14)AASHTO, LRFD bride design specifications, 3rd Ed., Washington, D.C, 2004.
- 15)Ya, S. and Yamada K., Fatigue durability evaluation of trough to deck plate welded joint of orthotropic steel deck, J. JSCE, Vol.64A, No.3, pp.603-616, 2008.
- 16)Xiao, Z.G., Yamada, K., A method of determining geometric stress for fatigue strength evaluation of welded joints, *Int. J. Fatigue*, pp.1277-1293, 2004.
- 17)Yamada, K. and Ya, S., Plate bending fatigue tests for root crack of trough rib of orthotropic steel deck, *J. Struct. Eng.*, JSCE, Vol.54A, pp. 675-684, 2008. (in Japanese)
- 18)Ya, S., Yamada, K., Ishikawa, T., Bending fatigue tests on trough to deck welded details of orthotropic steel deck, Orthotropic Bridge Conference, Sacramento, pp.240-255, 2008.
- 19)Ya, S., Yamada K., Ishikawa, T., Effect of weld melt-through on fatigue strength of trough-to-deck welded joint, Proc. of 64th JSCE Annual meeting, Division 1.,

pp.275-276, CD-ROM, 2009.

- 20)Ya, S., Yamada K., Ishikawa, T., Murai, K., Fatigue evaluation of trough rib to deck plate joint failed in weld throat, *Steel Construction Engineering*, Vol.16, No.64, pp.11-20, 2009. (in Japanese)
- 21)Japanese Society of Steel Construction (JSSC). *Fatigue* Design Recommendations for Steel Structures and Commentary. JSSC Technical Rep. No.32. (in English)
- 22)Yamada, S., Watanabe, N., Yamada, K., Osonoe, T., Nakano, T., Development of simple vibration fatigue testing machine, Topy Tekko Tech. Report, No.22, 2006, pp.82-83, 2006. (in Japanese)
- 23)Ya S., Fatigue durability evaluations of trough to deck plate welded details of orthotropic steel deck, Doctoral Thesis, Nagoya University, 2009.
- 24)Maddox, S. J., The fatigue behaviors of trapezoidal stiffener to deck plate welds in orthotropic bridge decks, TRRL Supplementary Report 96 UC, Crowthorne, Berkshire, 1974.
- 25)Ushio, M., Ueda, T., and Murata, S., Fatigue strength characteristics of welded connection of Trough rib and deck plate, Kansai Road Study Association Report, pp.1-11, 1985. (in Japanese)
- 26)Kawakami, J., Itou, S., Kawabata, A., and Matsushita, H., Fatigue test of welded joint between U rib and deck plate of Orthotropic steel deck, Proc. of 60th JSCE Annual meeting, Division 1, pp.791-792, CD-ROM, 2005. (in Japanese)
- 27)Structural Research Analysis Corporation (SRAC): COSMOSM User's guide, 2004.
- 28)Shinohara H., *Fatigue of orthotropic steel bridge deck*, Steel structure series 4, 1990. (in Japanese)
- 29)Specifications for Highway Bridges and Commentary. Part I, General, and Part II, Steel Bridges, Japan Road Association (JRA), 2002. (in Japanese)
- 30)Xanthakos, P. P., Theory and Design of Bridges, Wiley-Interscience, pp.616, 1993.
- 31)Yamada, K., Some new approaches to fatigue evaluation of steel bridges, *Int. J. Steel Structures*, KSSC, Vol.6, No.4, pp.319-326, 2006.
- 32)Iwasaki, M., Nagata, K., Inada, I., Yamada, K., Practical fatigue assessment method of orthotropic steel deck with service stress, *J. Struct. Eng.*, JSCE, Vol.43A, pp.1051-1058, 1997. (in Japanese)
- 33)Okura, I., Ishikawa, T., Tsutsui, M. and Osawa, S., Influence of contact shape of truck tires on plate-bending stresses in aluminum decks, *J. JSCE*, Vol. 63A, No. 4, pp.655-666, 2007. (in Japanese)

(Received September 24, 2009)