Orthotropic Deck Renovation of the Movable Bridge Scharsterrijn

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Orthotropic steel bridges experienced early fatigue failures of several welded connections in the steel deck plate. Solutions to enlarge the fatigue life of the existing movable decks include the bonding of a second steel plate of 6 mm to the existing 12mm deck. The adhesive is a thin epoxy layer, vacuum infused between the two steel plates. This renovation technique was for the first time applied on the orthotropic deck of the movable Bridge Scharsterrijn. In this paper, the results from static measurements performed on the old and renovated deck are presented. The tests were carried out with a calibrated truck positioned on specific locations of the deck. The resulting strains in the deck plate at each location were recorded using strain gauges installed on the deck. Strain influence lines were drawn for each strain gauge position and wheel load. After analyzing the results before and after the renovation, it can be concluded that the strain values decrease considerably after renovation. The strains measured at 15 mm from the welded connection between deck plate and stiffeners web reduce about 60 % in the deck plate and 40 % in the stiffeners web. As this is one of the critical details for the fatigue life of orthotropic bridges it can be concluded that this renovation technique seems to be a promising solution to extend the fatigue life of orthotropic decks of movable bridges.

1 INTRODUCTION

Orthotropic steel decks are used in most of the world's major long span bridges where low deadweight is an important factor. For the same reason, they are also largely used in movable bridges.

Recently, orthotropic steel bridges experienced some early fatigue failures of several welded connections in the steel deck plate. The most threatening cracks initiate at the welded connection between the stiffener web and the deck plate. Some of these fatigue cracks are located at the crossbeam and grow through the thickness of the steel deck plate. They are caused by the cyclic loading of the axles of heavy vehicles in the heavy vehicle lane (Jong, 2006; Kolstein, 2007). It became clear that new renovation techniques to stiffen the deck structure were needed to reduce the stress cycles.

Previous researches were focused on the development of a stiffer surfacing layer for renovation of orthotropic decks on fixed bridges. This renovation system consists of replacing the usual mastic asphalt wearing course by a layer of 50 mm reinforced high performance concrete (Jong, 2006). This renovation system has been applied in several fixed bridges in the Netherlands. On one of them, Moerdijk Bridge, strain measurements were performed. It was concluded that the strains in the deck plate reduce at least 80 % and the strains in the stiffener web 60 % (Kolstein & Sliedrecht, 2008).

Movable bridges demand more effective solutions due to restricted weight and height limits. One of the proposed solutions is also based on a concrete renovation layer where the usual 7 mm epoxy surfacing on movable bridges is replaced by a thin layer of reinforced ultra high performance con-

crete (UHPC) of 20 mm to 30 mm (Boeters, 2007; Schrieks, 2006). Further study is needed to improve the efficiency of this solution.
The other proposed solution is to add a second steel plate to the existing bridge deck (Labordus, 2006). The original wearing course is removed and a new steel plate is bonded to the existing deck by means of an interface layer. Afterwards, a new 7 mm wearing course is applied.
This rehabilitation technique was for the first time applied on the orthotropic deck of the movable Bridge Scharsterrijn in March 2009. The rehabilitation consisted of bonding a steel plate of 6 mm thickness to the existing deck plate in the heavy traffic lane.
The study presented in this paper reports the result of static strain measurements carried out using a calibrated truck before and after the Scharsterrijn Bridge rehabilitation. The obtained strain reduction factor determined from the measurements is compared with analytical studies previously carried out (Freitas, Kolstein & Bijlaard, 2008).

2 SCHARSTERRIJN BRIDGE RENOVATION

Scharsterrijn Bridge is located in Friesland, a northern province of the Netherlands, in the highway A6. The bridge is composed of two independent bridges, one for each traffic direction, both with a fixed and a movable part. The main girders of the movable bridge have a total span of 8300 mm. The bridge deck consists of an orthotropic steel plate of 12 mm thickness, stiffened by “U-shape” longitudinal stiffeners and divided by 4 crossbeams of 2530 mm span. The nominal thickness of the epoxy surfacing system is 7 mm (Figure 1).

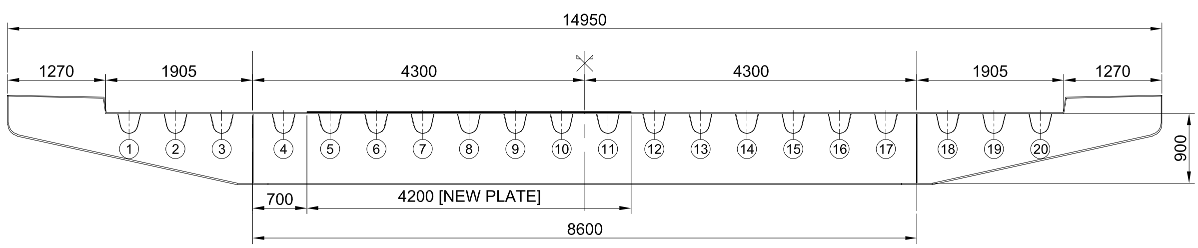

Figure 1. Cross section of Scharsterrijn movable bridge and new plate position (dimensions in mm).

The second steel plate of 6 mm thickness, 4200 mm width and 8200 mm length was bonded to the existing deck plate in the right lane of the movable bridge (Figure 1). The interface layer between the existing and new plate consists of a low-viscosity epoxy resin with 2 mm nominal thickness. The adhesive material is isotropic and its Young modulus is 4910 MPa. The epoxy is vacuum infused between the two steel plates aiming a void-free bond line.
The renovation procedure consists of the following chronologic steps: (1) remove the existing wearing course; (2) examine the bridge deck for existing cracks; (3) repair cracks if needed; (4) grit blast the steel top surface of the existing deck plate (Sa 2½) followed by primer application; (5) glue steel spacers of 2 mm thickness to the deck plate; (6) place the new steel plate in the correct position above the steel spacers; (7) prepare the cavity between steel plates to create vacuum; (8) vacuum inject the adhesive in the cavity; (9) cure during 16 hours between 40°C to 50°C temperature; (10) place the new wearing surface on the new steel plate.
The rehabilitation process was carried out by Takke LSBV Brugdekken VOF. (www.bridge-rehabilitation.com) by order of the Ministry of Transport and Public Works in the Netherlands.

3 EXPERIMENTAL PROCEDURE

In order to obtain information about the efficiency of this renovation technique, a monitoring plan was carried out. This monitoring includes static strain measurements performed before and after the

renovation. For both situations, the same procedure was carried out and it took place when no traffic was running on the bridge.
The strains on the deck plate were recorded using a calibrated truck, which was placed at 15 transverse positions on the deck. These positions where located in the right lane where the rehabilitation took place (see Figure 2). The transverse position of the calibrated truck was fixed by visual observation and using laser equipment. From the measurements, influence lines of the strains along the deck plate can be drawn due to the axle load. The reference point for the wheel position is the middle of the tyre for the front wheel and the middle of the outside tyre for the rear wheel.
A total of 16 strain gauges were applied in the middle span cross section of the movable bridge. For a proper understanding of the strain gauge measurements, it should be noted that at this location, no transverse web is present underneath the bridge deck. The locations of the 16 strain gauges are shown in Figure 2. The strain gauges next to the welds (2, 3, 5, 6, 8, 9, 11 and 12) are positioned 15 mm from the weld toe and measure transverse strains (Figure 3). The strain gauges 1, 7, 13, 15 and 16 are positioned at middle span between stiffener's webs, 1, 7, 13 measuring transverse strains and 15 and 16 longitudinal strains (Figure 4). Strain gauges 4, 10 and 14 measure the longitudinal strains at the bottom of the stiffeners.

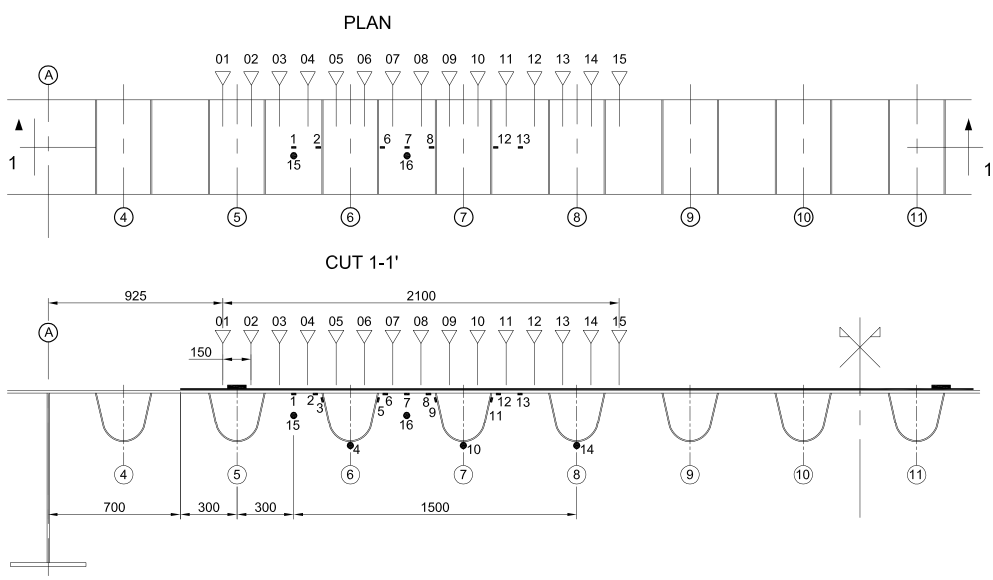

Figure 2. Strain gauges location and the 15 wheel load positions.

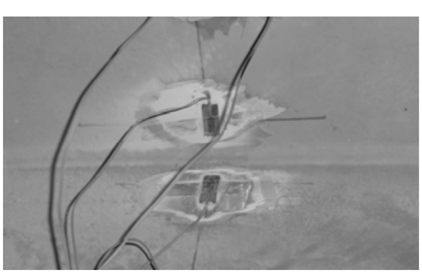
Figure 3. Strain gauges 2 and 3.

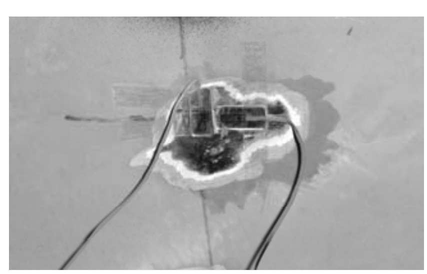
Figure 4. Strain gauges 1 and 15.

The calibrated truck used was a four axle lorry, with one axle at the front side with single tyres and three axles at the back side with double tyres. The measurements were taken using the front axle and the heaviest rear axles (middle rear axle).
In each test, before and after renovation, a total of four series of static measurements were carried out: two with the front wheel and two with the rear wheels. The data from all the measurements performed was recorded in eight series, four series in the situation before the renovation and another four series after the renovation.

For each load case the measurements were carried out twice. The measurements using the front wheel carried out before the renovation were recorded in Series 1 and Series 2. For the same situation but using the rear wheels the measurements were recorded in Series 3 and Series 4. After the renovation, the set of measurements for the front wheels was recorded in Series 5 and Series 6 and for the rear wheels Series 7 and Series 8.
The wheel loads and wheel prints used in each series are presented in Table 1.

Table 1. Calibrated truck: Wheel loads and prints, axle and wheel distance.

Wheel	Wheel Loads		Wheel Prints		Distance between	
	Before	After	Width	Length	Wheels	Axles
Front Wheel (single tyre)	37 kN (Series 1&2)	38.5 kN (Series 5&6)	270 mm	320 mm	2155 mm	5320 mm
Rear Wheel (double tyre)	38.5 kN (Series 3&4)	36.5 kN (Series 7&8)	620 mm	180 mm	1830 mm	

The front axle and rear axle have approximately the same load of 75 kN. The area of the wheel prints is 864 cm^2 for the front wheels and 972 cm^2 for the rear wheels.
These values can be compared with the set of standard lorries defined in the fatigue load model 4 of Eurocode 1 (EN1991-2, 2003) used for the fatigue calculations of bridges.
The front axle load of the calibrated truck is slightly higher than the 70 kN standard load for the front axles. The rear axle load used is lower then the minimum standard loads according to the same load model, 75 kN and 90 kN respectively. The wheel prints are the load area of the wheel, which is the contact area of the wheel with the deck plate. The wheel prints measured for the calibrated truck are in accordance with the classification given for the same load model. The front wheel is comparable to the wheel print type C (270 mm width and 320 mm length) and the rear wheels comparable to the wheel print type B (640 mm width and 320 mm length). The axle spacing and the transverse distance between wheels are in accordance to the values of the same load model.

4 RESULTS

The test results were recorded for all the sixteen strain gauges applied to the deck plate. The strain gauges were divided in five groups, each one for a typical location of the deck where measurements were recorded. Only the results from five strain gauges are described in more detail in this paper. Each of the five strain gauges represents a group. The selected strain gauges are believed to be representative of each comparable location.
For the transverse strains at the deck plate between stiffeners webs (Group I), only the results from strain gauge number 7 are presented. Strain gauge 1 and 13 are also part of this group. The transverse strains at the deck plate next to the weld were recorded in strain gauges 2, 6, 8 and 12 (Group II). In this paper the results from strain gauge number 2 are shown. The transverse strain at the stiffeners web next to the weld define the third group which includes strain gauges number 3, 5, 9 and 11 (Group III). The results presented in this paper are from strain gauge 3. Group IV and V include the longitudinal strains measured. For the values at the bottom of the stiffener (Group IV), from the strain gauge 4, 10 and 14, results are shown only for strain gauge number 10. For the values measured between the stiffeners web at the bottom side of the deck plate in strain gauge 15 and 16, the last is presented (Group V). The strain influence lines of the selected strain gauges are presented in Figure 5.
For strain gauge number 7, the measurements recorded before the renovation, in Series 1 and 2 for front wheel and Series 3 and 4 for the rear wheels, are presented in Figure 5 (a) and (b), respectively. The rear wheels exact position is much harder to achieve than the front wheel. This result in a bigger difference between series for the rear wheels measurements (see Figure 5 (a) and (b)). It can also be observed that strain values are higher for the front wheel when compared to the rear wheels (Figure 5 (c)). Having approximately the same load, the front wheel has a smaller wheel print than the rear wheels and therefore induces higher load stress on the deck plate.

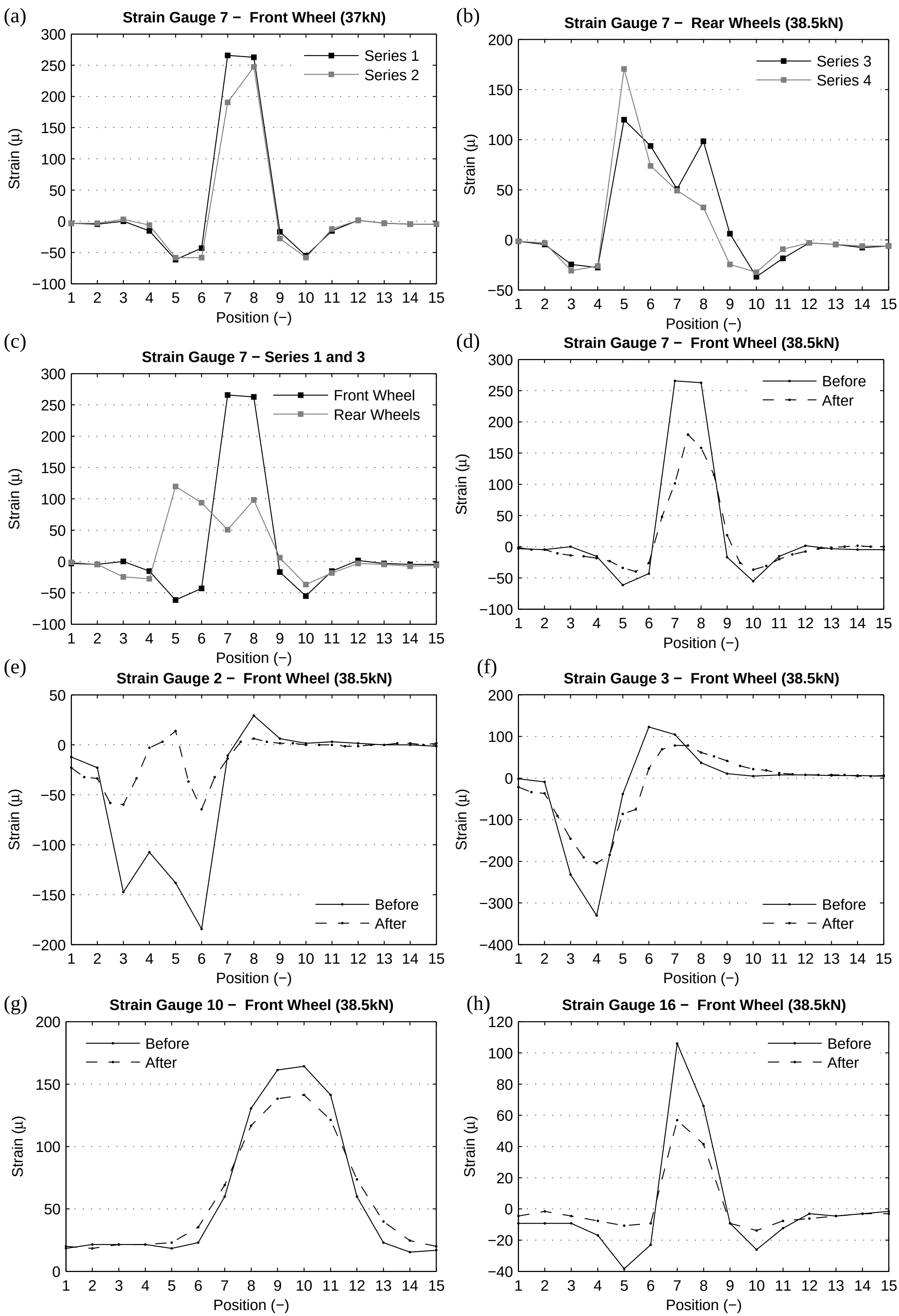

Figure 5. Measurements recorded for strain gauges 7 (a, b, c and d), 2 (e), 3 (f), 10 (g) and 16 (h).

The strain values before and after renovation for strain gauges 7, 2, 3, 10 and 16 are shown in Figure 5 (d), (e), (f), (g) and (h) respectively. The values presented refer to the front wheel load at the most critical series (higher strain values). The strain values measured before the renovation have been corrected to a wheel load of 38.5 kN.
The transverse strain of Group I (strain gauge 7 in Figure 5 (d)) have their maximum value in positive strains when the front wheel is positioned aligned with the strain gauge. Instead, the strains at the deck plate next to the weld, Group II (strain gauge 2 in Figure 5 (e)), have their maximum value in negative strains when the wheel is just before the next weld. This stress distribution confirms the mechanical model assumed for the stress calculation in (Jong, 2006) where the deck plate between the two stiffeners webs is modeled as a clamped beam.
Group III (strain gauge 3 in Figure 5 (f)) presents one peak value at negative strains and another with lower absolute values at positive strains. The longitudinal strains at the bottom of the stiffener (Group IV, strain gauge 10 in Figure 5 (g)) have a maximum positive strain when the wheel is aligned with the strain gauge. Finally, Group V has a similar behaviour as Group I but in a lower range of values (strain gauge 16 in Figure 5 (h)).
Looking to the difference before and after the renovation in each strain influence line, it can be observed that the strains after the renovation are considerably lower than before the renovation. For comparison of the results, a strain reduction factor was calculated for each strain gauge by expression (1).

$$SR_i = \left(1 - \frac{\varepsilon_i^{After}}{\varepsilon_i^{Before}}\right) \times 100 \qquad (1)$$

Considering the peak values, Table 2 presents the strain reduction factor of the sixteen strain gauges belonging to the five main group locations.

Table 2. Strain reduction factor.

Group	Strain Gauge	SR_i (%)	Location
I	1 7 13	55 38 45	Transverse strain measure at the deck plate at middle span between stiffeners webs
II	2 6 8 12	63 53 59 57	Transverse strain measure at the deck plate 15mm from the welded connection between deck plate and stiffeners web.
III	3 5 9 11	37 37 35 39	Transverse strain measure at the stiffeners web 15mm from the welded connection between deck plate and stiffeners web.
IV	4 10 14	18 14 7	Longitudinal strain measure at the bottom of the stiffeners.
V	15 16	57 47	Longitudinal strain measure at the deck plate at middle span between stiffeners webs

The strain reduction factor determined reveals consistent results between strain gauges of the same group. This gives an extra confidence of the results shown.
The transverse strain reduction in the deck plate between the stiffeners web amounts about 45 % (Group I). The measured strain reduction is lower than the one determined by analytical solutions (Freitas et al., 2008). The analytical values vary between 55 % and 65 % while the real values vary between 38 % and 55 %. The analytical calculations are based on very simple models that don’t include the complex geometry of an orthotropic deck. This can be the cause for such a difference. It is expected that the difference between values will decrease when compared to Finite Element Models results where the complete deck geometry is modeled.

The transverse strains next to the weld reduce around 60 % in the deck plate and 40 % in the stiffeners web.
The longitudinal strains in the bottom of the stiffener have the lowest strain reduction factor. The values are between 18 % and 7 % and decrease in the direction of middle span between main girders. The reason for a lower reduction might be related with a dominant global behaviour on the strains at this location. Due to that fact the local reduction of the new steel plate has less effect.
Finally, the longitudinal strains in the deck plate between the stiffeners webs reduce about 50 % after the renovation.

5 CONCLUSIONS

The orthotropic deck of the movable bridge Scharsterrijn was renovated by bonding a second steel plate of 6 mm thickness to the exiting 12 mm deck. The results presented in this paper are part of the monitoring plan carried out on this bridge rehabilitation.
The measurements reveal that the strains next to the weld between deck plate and stiffeners web reduce about 60 % in the deck plate and 40 % in the stiffeners web. As this is one of the critical details for the fatigue life of orthotropic bridges it can be concluded that the use of this technique can be an effective solution to prolong the life span of movable orthotropic bridges. Bonding a second steel plate to the existing deck in the heavy traffic lane looks like a promising solution for rehabilitation of orthotropic decks of movable bridges.
Further results include strain spectra measurements under real traffic conditions and dynamic test using a calibrated truck. These results will give more information about the strain reduction effect of this rehabilitation technique.

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