# Recalculation and strengthening of the double arch bridge

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## Abstract

Among the Netherlands' present infrastructure, some bridges do not conform to the prevailing Eurocode for new construction; some steel bridges are showing signs of (fatigue) damage. Over recent years, the Directorate-General for Public Works and Water Management (RWS) has been engaged in renovating the most critical bridges of the Netherlands, one bridge at a time. RWS is currently making preparations for the reinforcement of 3 steel bridges along the A27 motorway. Last year, Movares made complete recalculations for these bridges in order to determine what measures are necessary to reinforce them. This paper discusses the Merwede Bridge over the Upper Merwede river.

**Keywords:** Arch bridge; Assessment; Strengthening; Retrofitting; NEN 8700; Fatigue; Arch stability; rivets.

*Figure 1. Merwede Bridge (source: beeldbank.rws.nl)*

# 1 Introduction

The Merwede Bridge is a double-arch bridge on the busy A27 motorway, close to Gorinchem in the centre of the Netherlands. Many thousands of vehicles use the bridge daily to cross a major river, the Upper Merwede (an extension of the Rhine). The bridge is one of the few traffic connections joining the southern Netherlands to the heart of the country. In order to improve the traffic flow on the Houten to Hooipolder section of the A27, this section will have 4 lanes added in the near future, and new bridges will be built to supplement the existing bridges. The road layout on the existing bridge will be modified; see Figure 2 and Figure *3*. However, some uncertainty prevails concerning the load bearing capacity of the existing steel bridge structures that are to be retained on the A27.

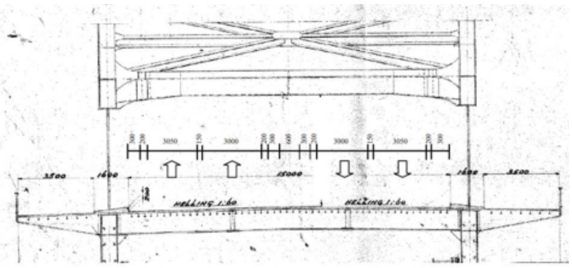

*Figure 2. Original road layout (since 1961)*

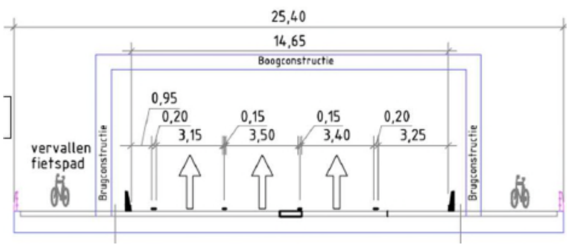


*Figure 3. Future road layout (from 2023)*

# 2 Description of the bridge

The Merwede Bridge was first commissioned in 1961. A notable feature of the bridge is that it is very narrow by today's standards. There are no hard shoulders at all, and the left (outer) lane is also narrower than normal. For that reason, overtaking is permanently prohibited for lorries. The total length of the Merwede Bridge is 780 metres. It consists of two linked arches, each 170 metres in length, 510 metres of approach spans, and a bascule bridge. This paper is about the recalculation for the two arch bridges.

## 2.1 Dimensions

The two linked arch bridges (see Figure 4) are deck-stiffened arch bridges. The arch is therefore relatively slim in comparison to the main girder. The arch and the main girder are both composed of box girders. The main girder is 2.5 metres high and 0.78 metres wide. The arch varies in height from 1 metre to 1.6 metres, and is also 0.78 metres wide. The ties connecting the main girder to the arch are locked coil cables.

The crossbeams have a relatively low construction height. They are inverted T-profiles, 875 mm high and 250 mm wide. These are welded to the deck plate. The deck plate is 10 mm thick, and is stiffened with bulb profiles.

Most of the bridge joints are welded. Only the sections for the purpose of the construction phases are riveted.

The entire main structure is made of L.Qmc 52 type steel. The yield stress of this material is 350 $N/mm^2$.

*Figure 4. Arch joint*

## 2.2 Bearing system

The bearing system of the Merwede Bridge consists of two end bearings and a central bearing for each main girder. The central bearing is the horizontal fixed point. This bearing is fully hinged.

The end bearings consist of two components: at about 6m from the end of the girder there is a vertical bearing that is free in the longitudinal direction of the bridge and fixed in the transverse direction of the bridge. The second component of the end bearing is located about 2 metres from the girder end. There is a tension anchorage at this location. The tension anchorage has free horizontal movement in the longitudinal direction of the bridge. The combined result of both end bearings is to create a restraint of the bridge end which can move horizontally in the longitudinal direction.

# 3 Loads on the bridge

In the Netherlands there is a supplementary code for the assessment of existing structures in case of reconstruction and disapproval: NEN 8700 (basic rules) and NEN 8701 (Actions).

These codes should be read in conjunction with the Eurocode. The NEN 8700 series contains a number of amendments to the Eurocode which can be applied. The two main amendments applied in the recalculation for the Merwede Bridge are explained in the paragraphs 3.1 and 3.2.

In addition to these codes, a supplementary guideline from RWS has been taken into account.

## 3.1 Reduction factors

The loads given in the Eurocode may be reduced due to:

- the shorter reference period than 100 years for the bridge's residual lifespan;
- a trend reduction if the period under consideration falls before the year 2060;
- the bridge's large influence lengths. This reduction factor only applies to the UDL and not the TS load.

In consultation with RWS and the Netherlands Organisation for Applied Scientific Research (TNO), the loads have been still further reduced in relation to the above factors. TNO has calculated an additional reduction factor for the static traffic load. For this purpose, TNO has analysed the measurement results for freight traffic across another, comparable bridge (the Moerdijk Bridge). Based on a probabilistic calculation, the UDL has been recalculated for an influence length of 170 metres. The additional reduction factor is ξ= 0.85, and is only applicable to the UDL and valid if the influence length is equal to 170m.

*Table 1. Factors applied to the UDL*

| component | influence length | factor |
|---|---|---|
| crossbeam | 5 m | 1.00 |
| main span | 85 m | 0.95 |
| Support and arch | 170 m | 0.68 |

## 3.2 Actual lane layout

Also, in accordance with NEN 8700, it is permitted to calculate using the actual (future) lane layout instead of the notional lane layout.

The width of the prospective loadbearing surface is 3 metres per lane. The actual (future) situation has therefore been adhered to for the location of the load. Within the lane, the positioning is adhered to in accordance with the rules of NEN-EN 1991-2.

# 4 Modelling of the bridge

In order to recalculate the bridge, we researched the most suitable method of modelling. Two types of model were made. As a basis, a global (SCIA Engineer) model was made, in which the entire bridge was modelled. For the local evaluations, a local model (ANSYS) was made.

## 4.1 SCIA Engineer

Using SCIA Engineer (FEM software), the global calculation models of the entire bridge were made from a combination of 1D bar elements and 2D plate elements.

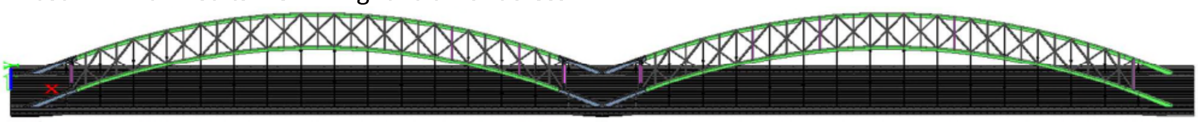

*Figure 5. SCIA Engineer model*

The following analyses were performed using this model:

- Global strength evaluation;
- Linear stability calculations;
- Determination of the stress variations caused by traffic on the main load bearing structure for the fatigue calculation.

## 4.2 ANSYS

Using ANSYS, the local calculation models were made for the deck structure. These models consist of a combination of 1D bar elements, 2D plate elements and 3D volume elements. The following analyses are performed using these models:

- Strength evaluation;
- The determination of the stress variations caused by traffic on the deck structure for the fatigue calculation.

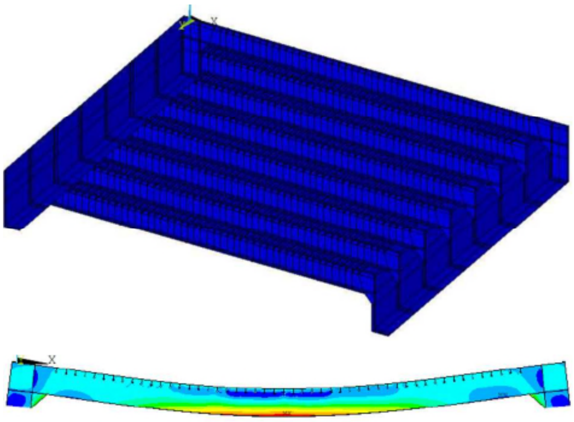

*Figure 6. ANSYS model*

In the evaluation of the components, the results of the local ANSYS model are combined with the global SCIA Engineer model.

# 5 Fatigue

An important part of recalculating the bridge is the evaluation of fatigue strength. This is because fatigue was not yet taken into account for traffic bridges at the time when this bridge was designed. Many old bridge structures show signs of fatigue damage over the course of time.

## 5.1 Road geometry

As described in the introduction, the road layout will change in the future (2023). Both road layouts have been used for the fatigue evaluation. The influence lines until 2023 have been made on the basis of the original road layout, and the influence lines from 2023 onward using the future road layout. An example of an influence line is given in Figure 7 for the two different road layouts.

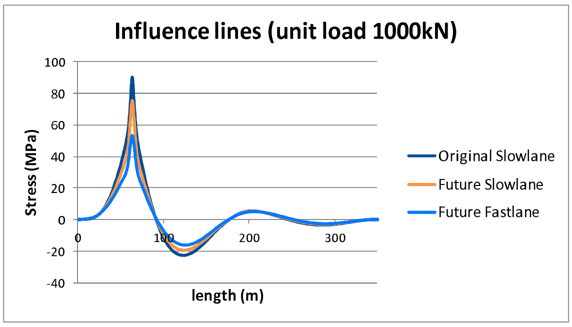


*Figure 7. Influence line example for global model*

## 5.2 Vehicles

In NEN 8701, load model 4 for fatigue (category ‘standard lorries’) has also been modified for fatigue load in relation to the Eurocode. Firstly, a distinction has been made in the lorry composition between three time periods (before 1990, 1990-2010, and 2011 to end of life). For each time period, there are small changes in, for example, the wheel type and axle load. A distinction is also made in NEN 8701 between a low, medium and high lorry load (loading of the lorry), whereby the low lorry load has a higher occurrence percentage (50%) than the high lorry load (15%). This distribution of the lorry load more closely approaches the reality than the distribution according to the Eurocode.

The performed fatigue calculations have taken account of lorry numbers, based on traffic counts. These counts have been interpolated into the past and extrapolated to the future. In the past, there was a prohibition on overtaking on the bridge due to the narrow lanes. However, this is no longer the case with the future road layout. The fatigue calculation for the future situation takes into account 10% simultaneous passage on the bridge.

It may happen that one lorry drives close behind another on the bridge (in convoy). This is an important factor for bridges with a large span. 20% of the vehicles drive in convoy; for this purpose, two lorries of the same type are assumed to drive with 50 metres between them on the same lane. It is possible for a lorry to overtake the convoy.

The guideline of RWS states that a set of lorries must be taken into account to avoid an unduly favourable result in the case of influence lines with positive and negative values that are asymmetric. The set consists of a total of 400 lorries from the tables in NEN 8701 (of all load ratios), placed in a random sequence. This takes into account the future percentages for the various lorries.

### 5.3 Asphalt

The local calculation model for fatigue takes into account the presence of asphalt. The effect of asphalt is included by applying it as a volume element, with the correct rigidity of asphalt. The asphalt is bonded to the deck plate with contact elements. The asphalt is only bonded to the deck plate vertically and not horizontally (able to slide). The asphalt's rigidity depends on the temperature. The asphalt has the following degrees of rigidity:

- at 0 degrees 15500 N/mm²
- at 15 degrees 8000 N/mm²
- at 30 degrees 500 N/mm²

The temperature distribution in these three temperature categories depends on the seasons and the time of day. The summarised distribution is given in Table 2.

*Table 2. Distribution of temperature categories*

| **Temperature** | 0° | 15° | 30° |
|---|---|---|---|
| **Percentage** | 33,33% | 47,92% | 18,75% |

The rigidity of the asphalt has a great influence on the stress variations in the deck plate and bulb profiles. In Figure 8, the influence line for the connection of a bulb profile to the deck is shown for the three temperature categories (normal stresses along the length of the bridge). With an asphalt temperature of 30°, the stress is more than twice as great as with an asphalt temperature of 15°. Inclusion of the asphalt in the model therefore has a big influence on the calculated fatigue damage.

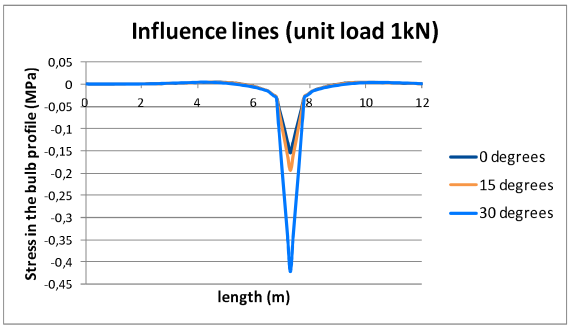


*Figure 8. Influence of asphalt temperature on the stress in a bulb profile*

### 5.4 Special fatigue details

#### 5.4.1 Rivets

The Merwede Bridge has many riveted joints which have been tested for fatigue. However, the Eurocode does not give any fatigue classifications for riveted joints. RWS has included fatigue classifications in its own guideline (based on European research projects) for the assessment of engineering structures. See Figure 9.

After evaluation of the above detail category, it has been found that not all the riveted joints in the bridge are satisfactory. Reinforcement measures are necessary to reduce the stress variations in these joints.

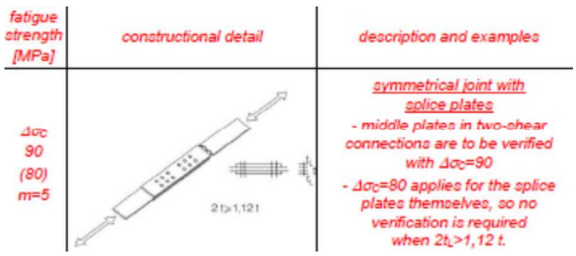


*Figure 9. Example of fatigue classification for a riveted joint*

### 5.4.2 Double fillet weld

In addition to the riveted joints, the bridge has numerous double fillet welds that must be tested for the fatigue detail as shown in Figure 10.

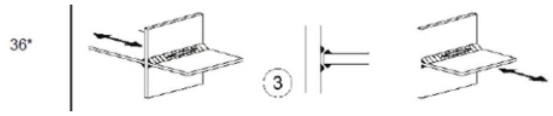


*Figure 10. Classification of a double fillet weld*

When this detail is evaluated according to the obtained normal stress from the model, the fatigue damage is very high. For this detail, the normal stress has therefore been divided into the part caused by bending stress and the part caused by normal stress in the plate. This enables more exact calculation of the stress in the weld by taking into account the relationship of the moment of resistance and surface between the double fillet weld and the plate. See Figure 11. With most details, especially with the connection of the bulb profile and crossbeam to the deck plate, this results in reductions of up to 70% on the stress in the model. This is due to the big difference in moment of resistance between the plate and the weld. See formulas (1) and (2).

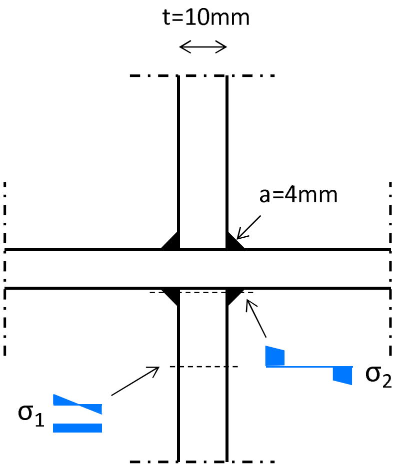


*Figure 11. Calculation of stress in a double fillet weld*

*Calculation of stress at locations 1 and 2:*

$$\sigma_1 = \frac{M}{W_1} + \frac{N}{A_1} \quad (1)$$

$$A_1 = 10mm$$

$$W_1 = \frac{\frac{1}{12}\times 1mm \times (10mm)^3}{5mm} = 17mm^3$$

$$\sigma_2 = \frac{M}{W_2} + \frac{N}{A_2} \quad (2)$$

$$A_1 = 8mm$$

$$W_2 = \frac{2\times 1mm \times (4mm)^2 + 2\times 4mm \times \left(5mm + \frac{4mm}{2}\right)^2}{5mm} = 81mm^3$$

### 5.4.3 Connection to the deck plate

In addition to the 36* detail (see Figure 10), the connection of the bulb profile and crossbeam to the deck plate has to be evaluated in terms of detail category 80 in accordance with NEN-EN 1993-1-9 (table 8.4 – detail 6). Because the deck plate tends to bend across the bulb profile/crossbeam, high bending stresses occur at the weld location. The detail referred to in the Eurocode is not suitable for this; for that reason, a detail category from a thesis by H. Kolstein [1] has been used. In this thesis, test samples were tested in accordance with the sketch given in Figure 12. The test results from this study show that the proper fatigue design category for the crossbeam to deck plate joint is category 125.

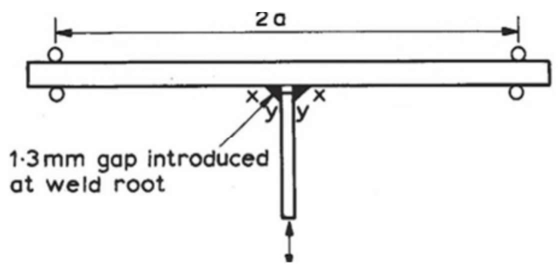


*Figure 12. Specimen crossbeam to deck joint [1]*

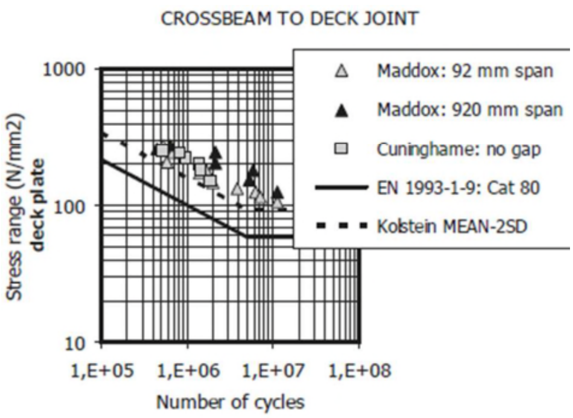


*Figure 13. Specimen crossbeam to deck joint [1]*

## 6 Arc stability

In order to check whether the arch is stable under all conditions, the stability was tested. Because the check for buckling length via the standard calculation rules is fairly conservative, the finite element model was used to calculate the eigenvalue.

In order to calculate the eigenvalue, it is important to properly model the bridge's rigidity. In this case, the use of a bar model for the arch is not adequate. On the first analysis, it was found that the arch portal, in particular, did not have adequate stability. However, the portal has much more rigidity than can be applied in a bar model due to (among other things) the radii of curvature and the spatial connection of the arch to the bridge (the impost). The two arches' connection to the intermediate support point also provides more rigidity, which cannot be modelled in a bar model. For this reason, it was decided to model the arch impost and portal in plates.

The disadvantage of modelling in plates is that, in a stability check, many eigenvalues for plate buckling first become visible in the model because these have lower eigenvalues than the arch's buckling form.

The discovered eigenvalue is $\alpha_{cr} = 9.24$ for the buckling form of the arch portal. And $\alpha_{cr} = 6.54$ for the buckling of the centre of the arch. See Figure 14 and Figure 15.

The eigenvalues were then used to determine the critical buckling force, after which a complete stability evaluation was performed in accordance with the Eurocode. The result of this evaluation is that the stability is not quite adequate. A minor reinforcement, whereby the arch's plates are restored from class 4 to class 3 (a gain of over 20%) is already sufficient to give adequate stability.

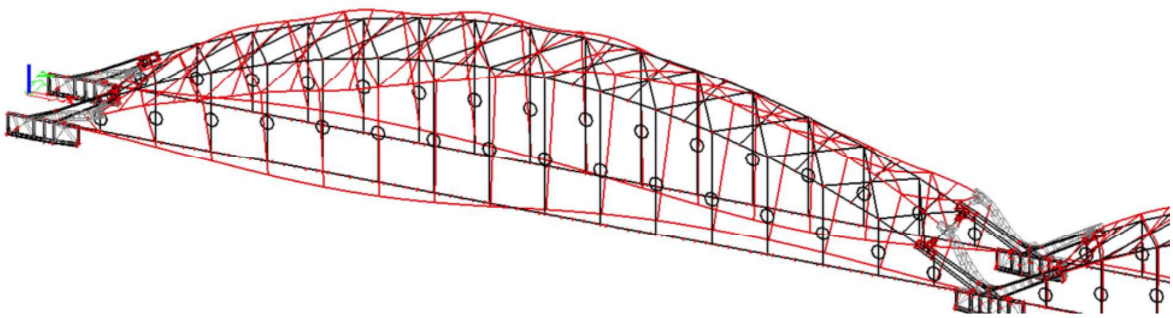

*Figure 14. Stability of the arch portal with an $\alpha_{cr}$ of 9.24*

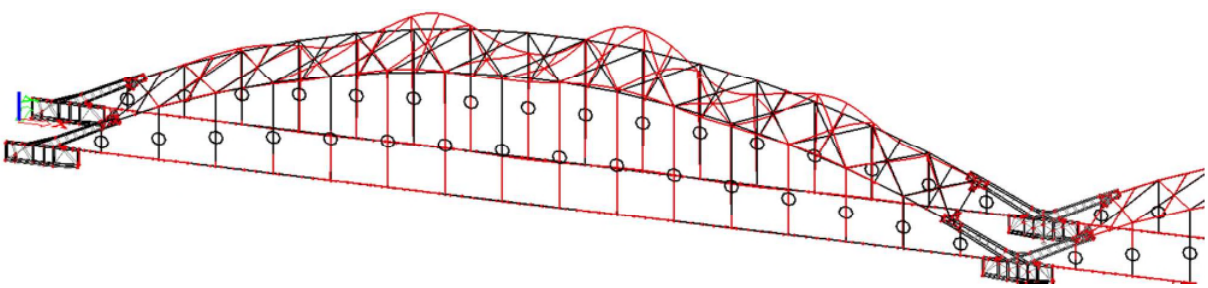

*Figure 15. Stability of the centre of the arch with an $\alpha_{cr}$ of 6.54*

## 7 Strengthening and retrofitting

The recalculation of the Merwede Bridge showed that the main girder, in particular, is not adequate in terms of strength and fatigue. To ensure that the bridge still has sufficient service life, reinforcement measures have been proposed. In the first instance, it was examined whether the proposed reinforcement measures are a workable solution to prevent fatigue problems in the future. An estimate was made of the dimensions of the reinforcements. The cross-sections of the crossbeam and main girder have been changed, as shown in Figure 16

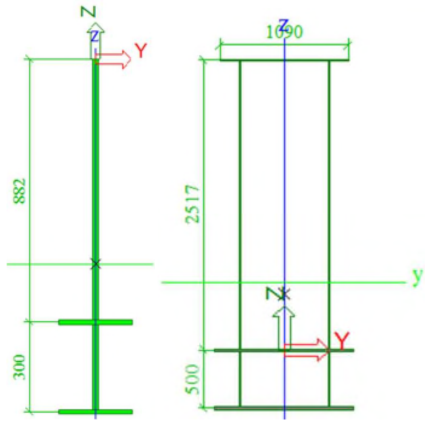


*Figure 16. Reinforced crossbeam and main girder*

Using the revised cross-section, the possibility was examined of making the structure adequately proof against fatigue. This calculation took into account any damage already existing. It concerns damage at the level of save life high consequence.

- If the damage in the current situation is less than 1, the remaining 'damage' can be used in the future situation;
- If the damage in the current situation is greater than 1 and the detail can be inspected, the future situation must be assigned a stress that is under the cut-off limit to ensure that no further damage ensues;
- If the damage in the current situation is greater than 1 and the detail cannot be inspected, it must expected that the original structure no longer makes any contribution in the future situation. The reinforcement must then transfer the full force.

## 8 Conclusions

In the Netherlands there is a specific code available for the recalculation of (steel) bridges: the NEN 8700 series. This makes it possible to calculate using less conservative loads and a modified safety level. In the case of bridges that do not conform to the Eurocode for new construction, the reinforcement measures can be minimised or avoided through these modified loads.

In the case of the Merwede Bridge, a number of components still do not conform when the NEN 8700 series is applied. For example, there are some local details (deck plate and bulb profiles) that do not conform in terms of fatigue. The damage levels are relatively minor, which means problems can be checked for with an inspection routine until the bridge is reinforced and given major maintenance.

## 9 References

[1] Kolstein M.H. Fatigue Classification of Welded Joints in Orthotropic Steel Bridge Decks. Delft; 2007.

[2] NEN 8700: 2011, Assessment of existing structures in case of reconstruction and disapproval - Basic Rules

[3] NEN 8701: Assessment of existing structures in case of reconstruction and disapproval - Actions