Field assessment of a concrete bridge
Case study

Lantsoght, Eva

Publication date
2017

Document Version
Submitted manuscript

Published in
Eco-efficient Repair and Rehabilitation of Concrete Infrastructures

Chapter 10

Title: Field assessment of a concrete bridge: Case study

Contributor

Dr. ir. Eva O.L. Lantsoght

Concrete Structures

Delft University of Technology

Delft, the Netherlands

Politecnico

Universidad San Francisco de Quito

Quito, Ecuador

www: http://www.citg.tudelft.nl/en/about-faculty/departments/structural-engineering/sections/sbe/chairs/concrete-structures/staff/profielen/mrs-ir-eol-lantsoght/

email: e.o.l.lantsoght@tudelft.nl

Abstract

This chapter discusses a case study of the field assessment through visual inspection and load testing of a reinforced concrete bridge with cracking caused by alkali-silica reaction, the viaduct Zijlweg. The first main topic of this chapter is the preparation, execution, and post-processing of the load test, through which it could be demonstrated that the capacity of the viaduct is sufficient and structural strengthening is not required. The second topic is a discussion of the cost-savings (economic, environmental, and social) that are obtained through this procedure as compared to a replacement of the superstructure.

Key words: Concrete bridges, Field assessment, Load testing, Material degradation

10.1. Introduction

10.2. Description of the viaduct Zijlweg

10.2.1. History of viaduct

10.2.2. ASR-monitoring

10.3. Preparation of field assessment

10.3.1. Damage identification

10.3.2. Preparation of proof load test: shear and bending moment positions

10.4. Execution of field assessment

10.1. Introduction

Field assessment of concrete bridges can be carried out in different ways. One way is the traditional visual inspection. To see beyond the surface of the structure, non-destructive techniques can be used. With these techniques, regions of corrosion, delamination, cracking, and other structural faults can be identified (ASCE/SEI-AASHTO Ad-Hoc Group On Bridge Inspection Rating Rehabilitation and Replacement, 2009, Ryan et al., 2012). In some cases, samples need to be taken from the bridge to know the concrete compressive strength, the level of carbonation, the amount of chlorides, and/or the steel quality that was used. To know more about the structural behaviour of a bridge, a non-destructive load test can be carried out (Schacht et al., 2016b).

Two types of load tests can be distinguished. The first type consists of diagnostic load tests (Sanayei et al., 2016, Olaszek et al., 2014, Matta et al., 2008, Velázquez et al., 2000), which are carried out at a lower load. These types of tests can be used for comparison to an analytical model. By studying the differences between the analytical model and the field response, it can be identified which elements differ from the assumptions in the analytical model. Examples of such differences, which can occur separately or combined, depending on the bridge type, are (Barker, 2001):

- the actual impact factors as compared to the impact factor from the code,
- the actual section dimensions,
- the unaccounted stiffness of secondary elements such as curbs and railings,
- the actual transverse load distribution,
- the level of restraint at the bearings,
- the actual longitudinal load distribution,
- unintended composite action with the deck.

The second type of load testing is proof load testing (Lantsoght et al., 2016c, Liu et al., 2014, Faber et al., 2000, Lin and Nowak, 1984, Koekkoek et al., 2016). Proof load testing is chosen when there are large uncertainties that make an analytical determination of the structural response difficult. Such uncertainties include the effect of material degradation, uncertainties from a lack of information when structural plans are missing, and uncertainties with regard to the load path at higher load levels. In a proof load test, a load that corresponds to the factored live load is applied to the bridge. If the bridge can withstand the applied load without signs of distress, it is shown that the bridge can carry the prescribed loads to a satisfactory level. None of the non-destructive load tests give insight in the ultimate capacity of the tested structure. If the ultimate capacity needs to be known, and the bridge is decommissioned, a collapse test can be considered (Lantsoght et al., 2016a, Lantsoght et al., 2016b, Lantsoght et al., in press, Bagge et al., 2015, Nilimaa et al., 2015, Puurula et al., 2015, Puurula et al., 2014).

One type of material degradation that makes an analytical assessment of a bridge difficult is the effect of damage caused by alkali-silica reaction (ASR). ASR takes place when the alkali in the cement reacts with silica that is present in some aggregates. The result of this reaction is a gel. When this gel comes in contact with moisture, the gel will expand. This expansion causes internal stresses in the concrete. If these stresses exceed the tensile strength of the concrete, cracking will result. In particular, the shear capacity of elements with ASR-damage is subject to discussion, as the cracking reduces the tensile strength of the concrete (Siemes et al., 2002). Experimental research has shown that ASR-damage has a limited effect on the bending moment capacity, unless the level of expansion caused by the ASR-gel is high (Talley, 2009). For reinforced concrete members, the expansion is partially counteracted by the reinforcement, and a prestressing effect takes place. This idea is confirmed by testing reinforced concrete members with and without ASR-damage, in which it is found that the cracking moment for the specimens with ASR is higher than for the specimens without, as a result of this prestressing effect (Haddad et al., 2008). For the shear capacity, some authors report that laboratory testing leads to higher shear capacities (attributed to the beneficial prestressing effect) (Ahmed et al., 1999, Ahmed et al., 1998), whereas testing of beams (den Uijl and Kaptijn, 2004) taking from ASR-affected viaducts resulted in a reduction of the shear capacity by 25% when compared to Rafla's formula (Rafla, 1971). Additionally, a number of reported load tests on ASR-affected viaducts (Talley, 2009) in Japan, France, South Africa, and Denmark (Schmidt et al., 2014) showed that the effect of ASR on the overall structural response is limited. However, these few studies are not sufficient to declare all ASR-affected viaducts as structurally safe. Specific cases can be analysed with proof load testing. In this chapter, the proof load testing of the ASR-affected viaduct Zijlweg is discussed.

10.2. Description of the viaduct Zijlweg

10.2.1. History of viaduct

The viaduct Zijlweg is located in the road Zijlweg near Raamsdonksveer and Waspik and crosses the highway A59. The bridge was built in 1965 for the Province of Noord Brabant in the Netherlands. The originally devised service life was 80 years, and the original live loads were determined for traffic class B, which used a distributed lane load of 400 kg/m^2 and a design truck with two axles of 10 ton and one axle of 20 ton. The current bridge owner is Rijkswaterstaat, the Ministry of Infrastructure and the Environment, Direction Noord Brabant. A photograph of the viaduct is given in Figure 1.

Viaduct Zijlweg is a reinforced concrete solid slab bridge with four continuous spans under a skew angle of 14.4^{o}. The span lengths are 10.32 m for the end spans and 14.71 m for the central spans. An overview of the geometry of viaduct Zijlweg is given in Figure 2. The total width of the cross-section is 6.6 m, whereas the width of the carriageway equals 4 m. The thickness varies parabolically between 550 mm and 850 mm. The spans are supported by concrete piers at the central supports and by an abutment at the end supports, and elastomeric bearing pads are used at the supports.

In terms of documentation of the bridge, the original calculation report (Provincie Noord Brabant, 1965) is available. Repair activities were carried out in 2002, and a repair of these activities with a plan for management and maintenance is available (Rijkswaterstaat, 2002). An inspection report from 2008 (Gielen et al., 2008) is available as well, and inspections are programmed to take place every five years. During the repairs in 2002, a waterproofing layer was added on the top side of the slab, to prevent further ingress of moisture. The inspection of 2008 concluded that the viaduct is in moderate conditions. The possible insufficient capacity of the main superstructure was identified as a considerable risk. As such, it was noted that the viaduct does not fulfil all the performance requirements and has an increased risk with respect to safely fulfilling its required functions.

10.2.2. ASR-monitoring

The presence of alkali-silica reaction (ASR) was detected in 1997 in a large number of bridges over the highway A59 in the Province of Noord Brabant, including the viaduct Zijlweg (Projectteam RWS/TNO Bouw, 1997). All these bridges were built around the same time, using the same materials. For the viaduct Zijlweg, cracking had occurred as a result of the ASR-damage. When taking concrete cores from the viaduct, many of the cores would be completely intersected by cracks, and thus have a uniaxial tensile strength of 0 MPa. Other cores still had some uniaxial tensile strength, but much smaller than expected from the concrete compressive strength. At that time, the small uniaxial tensile strength of the concrete in bridges with ASR-damage caused concerns with regard to the shear capacity of these bridges. Besides the low uniaxial tensile strength, these bridges were also showing large cracking and were not provided with shear reinforcement. Moreover, the bridges were designed for lower live loads and larger shear capacities than prescribed by the currently governing codes.

A structural assessment was carried out in 1997. In a first hand calculation, the Unity Check for shear at the end support was found to be UC = 5.4 and at the mid support as UC = 4.7 (Projectteam RWS/TNO Bouw, 1997). The Unity Check is the ratio of the shear stress caused by the considered load combination to the shear capacity. The load combination for assessment is the combination of the self-weight, superimposed dead load, and live loads, consisting of distributed lane loads and concentrated loads for the design tandem as prescribed by Load Model 1 from NEN-EN 1991-2:2003 (CEN, 2003). A Unity Check larger than 1 thus means that the structure does not fulfill the requirements. The extremely large values for the Unity Check found in the first calculations raised serious concerns with regard to the shear capacity. However, prior to deciding that the structure should be strengthened or replaced, it was decided to carry out refined calculations. In these calculations, the value of the tensile strength was determined from the combination of 51 specimens tested in uniaxial tension and 10 specimens in splitting tension. The shear stress caused by the considered load combination was determined by using a linear finite element program. From these calculations, the results were UC = 1.29 at the end supports and UC = 1.31 at the mid supports (Projectteam RWS/TNO Bouw, 1997). These results still indicate that the structural capacity of the viaduct Zijlweg is insufficient, and that further studies are required to determine if the bridge should be strengthened or replaced. It was then decided that the viaduct Zijlweg would be a good candidate for assessment through proof load testing.

In 2003, monitoring of the viaduct for ASR was applied (Koenders Instruments, 2015). The temperature, deck thickness, moisture in the concrete, and longitudinal expansion of the deck are monitored, see Figure 3. Measurements locations 3 to 7 were installed in 2003, and additional sensors on locations 8 to 10 were installed in mid-2007. The measurements are taken hourly, and were started on April 1st 2003. The inspection report of (Gielen et al., 2008) also evaluated the results of the ASR-monitoring. It was noted that the expansion in the longitudinal direction of the viaduct was reaching the maximum value. The measurements of the monitoring system can be compared to the ambient temperature from the official Dutch meteorological institute. The selected location is Gilze-Rijen, 14 km south of the viaduct Zijlweg. The full analysis of the data can be found in the experimental report (Koekkoek et al., 2015). In all plots, a clear correlation between the ambient temperature and the thickness, joint size, and moisture content can be seen. The average deck thickness follows the trend of the ambient temperature (see Figure 4), whereas the joint size is inversely correlated to the temperature. The joint size decreases when the expansion in the longitudinal direction of the deck increases as a result of increasing temperatures. As such, the observed data follow the expectations. From the data, it is also seen that the moisture increase is only observed at one measurement point, and that this increase is limited.

10.3. Preparation of field assessment

10.3.1. Damage identification

The material properties were determined in the same period as when the field test was carried out. The concrete properties were determined based on six core tests (Witteveen+Bos, 2014), resulting in a characteristic cylinder concrete compressive strength of 24.5 MPa and an average cube compressive strength of 44.4 MPa. The properties of the reinforcement steel were not determined based on sample tests. However, the symbols on the drawings indicate that plain reinforcement bars were used. These bars could be steel grade QR22 (with a characteristic yield strength of 220 MPa) or QR24 (with a characteristic yield strength of 240 MPa), and it is not specified on the drawings which grade was used. An overview of the reinforcement in viaduct Zijlweg is given in Figure 5.

Prior to the field test, a visual inspection of the bridge was carried out. In this inspection, deterioration of the top deck was observed, which was limited to the edge of the sidewalk, see Figure 6. As a result of the longitudinal expansion, the expansion joint between the deck and the abutment has become very small, see Figure 7. Given this problem with the expansion joint, it was decided to monitor the joint opening and closing during the proof load test. As a result of the ASR-damage, a typical cracking pattern (map cracking) was observed on the bottom of the slab bridge. Additionally, cracking on the side faces was detected. The drawing showing all detected cracking is given in Figure 8.

10.3.2. Preparation of proof load test: shear and bending moment positions

The northernmost span, span 4 in Figure 2, is used for a proof load test. The reason why the end span is used, is that this span is not directly above the highway, so that closing off the highway during the execution of the test is not required and no obstruction results to the traveling public. Span 4 has a varying thickness over the length. At the end support, the thickness is 550 mm. The thickness increases parabolically to 850 mm near the mid support. The parabola has a curvature radius of 150 m.

To define the positions at which the proof load should be applied, a linear finite element model is used. In this model, the sectional forces and moments are determined. The model is developed in TNO Diana (TNO DIANA, 2012). The thickness of the slab in the transverse direction was considered to be uniform; the larger depth of the sidewalk was not modeled. Instead, an additional load was applied to represent the additional self-weight from the sidewalk. Shell elements were used in the model (eight-node quadrilateral isoparametric flat shell elements). The slab is modeled as 5.7 m wide instead of the full 6.6 m (see Figure 2 for geometry). Elements with a height of 500 mm and a width of 483.15 mm were used in the model. The crossbeams at the intermediate supports have an element width of 410 mm and the crossbeams at the end supports have an element width of 250 mm. All crossbeams have elements of 500 mm height. The final mesh contains 106 elements in the longitudinal direction and 12 elements in the transverse direction. The two elastomeric bearing pads supporting the crossbeams are modelled as supports in the finite element model. Full details of the finite element model can be found in the analysis report of the proof load test (Koekkoek et al., 2015).

For an assessment of reinforced concrete slab bridges, a load combination containing the self-weight, superimposed dead load, and loads from the live load model (distributed lane loads, and concentrated live loads) is used. These loads are also applied to the finite element model. The self-weight (permanent dead weight) is automatically derived from the geometry modeled in the finite element model, using a load of 25 kN/m^3. The load at the edge, resulting from the difference between the modeled 5.7 m and the real 6.6 m width, is applied as 2.3 kN/m. Additionally, the wearing surface of 46 mm of concrete is modeled by applying a distributed load of 1.15 kN/m^2. The superimposed dead load (variable dead weight) is modeled as an asphalt layer of 110 mm with a load of 23 kN/m^3.

For the live loads, Load Model 1 from NEN-EN 1991-2:2003 (CEN, 2003) is used, consisting of a distributed lane load and concentrated wheel load pertaining to a design tandem in each lane, as shown in Figure 9. The wheel print of the design tandem is 400 mm × 400 mm, see Figure 9. An axle load of α_{Q1} × 300 kN is applied in the first lane, of α_{Q2} × 200 kN in the second lane, and of α_{Q3} × 100 kN in the third lane. Since the viaduct Zijlweg only has one lane, only α_{Q1} × 300 kN is applied. The values of α_{Qi} are nationally determined parameters, which, for the Netherlands, all take the recommended value of α_{Qi} = 1. The lane load equals α_{q1} × 9 kN/m^2 for the first lane, and α_{qi} × 2.5 kN/m^2 for all lanes with i > 1. Again, the values of α_{qi} are nationally determined parameters, which, for the Netherlands, take the value of α_{q1} = 1 when only one lane is present. The distributed lane load is applied over the width of the notional lane of 3 m, and pattern loading is used to find the most unfavorable loading arrangement. On the remaining width, $\alpha_{qr}q_{rk}$ = 2.5 kN/m^2 is applied with α_{qr} = 1 and q_{rk} the distributed load on the remaining width of the viaduct, and on the sidewalk a pedestrian load of 5 kN/m^2 is applied. Additionally, since the viaduct has less than 250000 vehicles per year, the reduction factors from NEN-EN 1991-2/NA:2011 (Code Committee 351001, 2011b) are used: 0.97 on the live loads from Load Model 1, and 0.90 on the remaining area. The reduction does not apply to the pedestrian load on the sidewalk.

The wheel print from Load Model 1, see Figure 9, is 400 mm × 400 mm. Since the finite element model uses shell elements, the loads are applied at mid-depth. It is assumed that the load is distributed under 45^o over the height, see Figure 10, so that the load is distributed over 950 mm × 950 mm in the finite element model, or 2 by 2 elements. The load per wheel of 150 kN becomes a distributed load of 0.155 N/mm^2. The tandem is centered in the notional lane of 3 m, so that distance between the edge of the lane and the face of the first wheel in the transverse direction equals 500 mm. The wheel print of the proof load tandem is 230 mm × 300 mm, which better corresponds to the actual wheel print of a vehicle. The same distribution as shown in Figure 10 leads to a contact area in the finite element model of 780 mm × 850 mm.

The viaduct Zijlweg has a skew angle of 14.4^o. Therefore, in the finite element model the position of the wheel prints is applied in two ways: parallel to the driving direction, and following the width direction. The analysis showed that for bending moment, applying the loads parallel to the driving direction is more unfavorable and that for shear, the position along the width is more critical.

The different safety levels that are used in the Netherlands are described in NEN-EN 1990:2002 (CEN, 2002) for new structures, in NEN 8700:2011 (Code Committee 351001, 2011a) for existing structures, and with additional requirements for existing bridges in the Guidelines for the Assessment of Bridges ("RBK") (Rijkswaterstaat, 2013). An overview of these safety levels, their corresponding reliability index, and reference period is given in Table 1. For a bridge assessed with a proof load test, a different load combination, see Table 2 is used. For this load combination, the load factor for the self-weight becomes 1.10. For an existing structure, the self-weight can be considered a deterministic value. Only the model factor remains, which equals 1.07 in NEN-EN 1991-2+C1:2011 (CEN, 2011). This value can be rounded off to 1.10. So, for an assessment using the load combination with the Eurocode live loads, the load factors γ_{sw}, γ_{sd}, and γ_{LL} from Table 2 are used, whereas for preparations for a proof load test, the load factors γ_{sw}, γ_{sd}, and γ_{proof} from Table 2 are used.

To find the required load on the proof load tandem for the bending moment test, first the critical position is sought. For this purpose, the design tandem of the Eurocode is moved along the span (parallel to the driving direction as discussed previously) until the position is found that results in the largest sectional moment. Then, the Eurocode live loads are removed, and the proof load tandem is applied at the critical position. The load on the proof load tandem is then increased until the same sectional moment is found as with the Eurocode load combination. For bending moment, the critical position in span 4 of the viaduct Zijlweg is found at a face-to-face distance between the support and the tandem of 3382 mm (7 elements). The magnitude of the target proof load depends on the considered safety level. The resulting values for the required target proof loads are given in Table 3 as $P_{tot,bending}$.

For shear, the critical position is known to be at a face-to-face distance of $2.5d_l$ between the load and the support (Lantsoght et al., 2013b), with d_l the effective depth to the longitudinal reinforcement. An overview of the finite element model is given in Figure 12. This critical distance is derived from slab shear experiments in the laboratory (Lantsoght et al., 2015c, Lantsoght et al., 2015b, Lantsoght et al., 2013c, Lantsoght et al., 2014, Lantsoght et al., 2015a). This distance, however, has been derived for straight slabs, and the behavior of skewed reinforced concrete slabs in shear requires further research. Limited testing showed that the behavior of skewed slabs in shear is complex and that the failure mode changes as the skew angle changes (Cope et al., 1983, Cope, 1985). It is known that the obtuse corner results in the largest concentrations of shear stresses, so that the critical position is with the tandem in the obtuse corner. The peak shear stress in the linear finite element model can be distributed over $4d_l$, as was derived based on the comparison between linear finite element models and measurements of the support reaction for slabs (Lantsoght et al., 2013a). This averaged shear stress is then used for the analysis and for comparison between the shear stress caused by the load combination prescribed by the code and the load combination with the proof load tandem. Both the design tandem and the proof load tandem are placed at a face-to-face distance of $2.5d_l$ from the support. First, the sectional shear (averaged over $4d_l$) is determined caused by the load combination prescribed by the code. Then, the required proof load to get the same sectional shear (averaged over $4d_l$) is determined. The proof load tandem is placed in the obtuse corner, which is known from the literature (Cope, 1985) to lead to the largest concentrations of shear stresses. Finite element models were made to study the difference for the viaduct Zijlweg between loading at the acute and obtuse corner. It was indeed confirmed that the critical position is in the obtuse corner (Koekkoek et al., 2015). Finally, the results of the target proof loads for shear $P_{tot,shear}$ for the different safety levels are determined, as shown in Table 3. It must be mentioned that proof load testing for shear is generally not permitted by the existing codes and guidelines, and that the development of stop criteria for a proof load test for shear is still subject of research (Schacht et al., 2016a).

For practical reasons during the execution of the proof load test, the position of the load is slightly moved from the critical position determined based on the finite element models. The same centerline is kept for the test for bending moment as for the test for shear, so that the loading setup can be partially kept in place. This means that during the execution, the supports can remain in place, but that the load spreader beams, jacks, and load cells need to be moved between the two experiments.

10.4. Execution of field assessment

10.4.1. Load testing procedures

The field tests on the viaduct Zijlweg were carried out on Wednesday, June 17th 2015. On Sunday, June 14th, all measurement equipment was brought to the test site. On Monday and Tuesday morning, all sensors were applied, and on Tuesday afternoon, all sensors were tested.

For the proof load tests at the shear- and flexure-critical positions, a cyclic loading protocol was used. According to the German guideline for load testing (Deutscher Ausschuss für Stahlbeton, 2000), each load test should be carried out in at least three steps, during which the maximum load in each step is kept constant for at least two minutes. The advantage of a cyclic loading protocol (Koekkoek et al., 2016) is that reproducibility, symmetry, and linearity of the measurements can be verified. For testing of the viaduct Zijlweg, four load levels were selected:

1) A low load level of about 40 metric ton, to verify if all sensors are functioning properly. If sensor malfunctioning is detected, it is possible to make corrections prior to continuing with the proof load test. In the bending moment test, corrections were

necessary for the load cells at the first load level, and in the shear test, one LVDT had to be placed within its measurement range.

2) The Serviceability Limit State load level. For this load level, large deflections or cracks are not expected to occur, and the behavior should be linear elastic. Prior to continuing to the next load level, the measurements are interpreted to see if it is safe to load to the next load level.
3) An intermediate load level, to build up to the target proof load. The measurements are followed closely, and based on the observed structural response, it is determined if the testing can be continued.
4) The target proof load, the RBK design level from Table 3, plus 5%. This load level does not require cycles, as the stop criteria and linearity of the measurements need not be interpreted to decide if further loading is allowed. The 5% additional loading is applied to cover the local variations in the material, and to take into account the fact that only two positions are tested.

After each load cycle, the load is not returned to the level of 0 ton, but instead a lower threshold value of 10 ton is used. This minimum load level ensures that all sensors remain activated, and avoids the occurrence of noise on the acoustic emission measurements, which are sensitive to full unloading. The aim was to keep the loading speed constant during the load test. However, the speed was determined by a manual operation, so that some deviations occurred. The position of the proof load tandem in the bending moment test is shown in Figure 12, and the position in the shear test in Figure 13.

10.4.2. Sensor plan

At various locations on the bottom of the slab, side faces of the slab, and at the joint, sensors are placed to follow different responses of the bridge during the proof load test. An overview is given in Figure 14. All data are measured in real time and shown on the measurement computer in the control center. These results are used after every load cycle to determine if further loading can be permitted. The following structural responses are followed during the proof load tests:

- deflections of the slab
- deflections of the cross-beams
- crack widths
- strains on the bottom of the slab
- rotation of the end support
- acoustic emission signals

- opening of the joint
- opening of cracks on the side face

These responses are measured with linear variable differential transformers (LVDTs), and laser distance finders (lasers). The opening of cracks on the side face is monitored by applying gypsum on the side face over the crack and then checking after the test if the gypsum has cracked, which would mean that the crack was activated during the test. An overview of the applied sensors is given in Table 4. In this chapter, the results of the acoustic emission signals are not discussed, as they are a topic of further research. The analysis of the acoustic emission measurements is given elsewhere (Yang and Hordijk, 2015).

The vertical displacement of the deck is determined with measurements taken by laser distance finders and LVDTs. At the center of the notional lane, a row of four LVDTs is placed to determine the longitudinal deformation profiles. An additional LVDT is used to correct for the way the measurement frame is applied: it is attached to the slab at the mid support, but resting on the abutment at the end support. For the transverse deformation profiles, two lasers are used, at a location between the position of the wheel prints for the bending moment test and the shear test. The goal of measuring the deflections is to set up the deformation profiles in the transverse and longitudinal directions, to follow the load-displacement diagram in real time during the experiment, and to calculate residual deformations after each load step. The load-displacement diagram is used to detect non-linearity, and existing codes (Deutscher Ausschuss für Stahlbeton, 2000, ACI Committee 437, 2013) give limits to the residual deformation as a stop criterion. An overview of the applied lasers and LVDTs for measuring the displacements is given in Figure 15. The vertical displacement of the cross-beams at the end support and mid support is measured with lasers (two lasers per support). The deflection of the cross-beam results from the compression of the elastomeric bearings that are used for the supports. The position of these measurements is indicated in Figure 16.

The crack width can be measured on a crack that forms during the proof load test, or on an existing crack. The difficulty lies in estimating which crack will be activated during the test. To measure the increase in crack width, LVDTs are placed horizontally over the crack. For the proof load tests on the viaduct Zijlweg, existing cracks were monitored during the test. A longitudinal crack and a transverse crack were selected prior to the proof load test for monitoring during the experiment. The German guideline prescribes a maximum crack width during a proof load test, as well as a maximum residual crack width that needs to be verified after each load step. An overview of the positions of the LVDTs measuring crack width is given in Figure 17.

The strain on the bottom of the slab can be measured by applying an LVDT horizontally over 1 m. Three LVDTs are used for measuring strains, and a reference LVDT is placed on a part of the bridge that is not loaded, to measure the influence of temperature and humidity. Especially since the material of the LVDT support construction is made out of aluminum, which has a large coefficient of thermal expansion, it is important to correct for the effect of temperature (and humidity). The German guideline prescribes a limiting concrete strain as a stop criterion. Therefore, the strain on the bottom of the cross-section needs to be followed during the experiment. If a new crack develops within the 1 m over which the strain LVDT measures, this event will also be measured during the proof load test. An overview of the tree strain LVDTs is given in Figure 18.

The expansion caused by alkali-silica reaction resulted in a clear expansion of the viaduct in the longitudinal direction, leaving less space for the joint. To measure the movement in the joint and the rotation of the end support, and to check if at some point insufficient space is left in the expansion joint, and/or if the required rotation of the bridge deck during the proof load tests becomes restrained, two LVDTs are placed on both sides of the viaduct. The LVDTs measure the joint horizontally. One end of the LVDT is connected to the abutment and the other end to the slab. The layout of the LVDTs on the joint at the west side of support 5 is shown in Figure 19. For the east side, LVDTs are applied at the same positions as for the west side; the only difference then is the numbering of the LVDTs, which are LVDT 11 and 12 for the east side.

A last measurement is the measurement of the applied load during the proof load test at the four different wheel prints by four separate load cells. The load cells have a capacity of 1000 kN and an accuracy of 1% (10 kN). The measurements of the load are important during the proof load test, as they are used to follow the load-displacement diagram in real time. When the load-displacement diagram ceases to be linear, one of the stop criteria is exceeded.

10.5. Post-processing of field assessment

10.5.1. Test results

First, the results of the proof load test at the flexure-critical position are studied. The maximum measured load in the proof load tandem was 1332 kN. Adding the weight of the steel plate and jacks results in a total load of 1368 kN. Non-linearity is studied based on the envelope of the load-displacement diagram, in which the measured load on the four jacks is used. The envelope of the load-displacement diagram is given in Figure 20. The black lines indicate the tangent to the load-displacement diagram, or the stiffness. If the angle of the black lines changes significantly, non-linear behavior is observed. In the third black line, the stiffness has decreased mildly, whereas for the last loading and unloading step, stiffening occurred in the unloading branch. This stiffening could be caused by redistribution of stresses at the higher loads, interaction between the applied loading frame and the structural behavior of the bridge, or the lower loading speed in the last loading step.

The LVDTs in the longitudinal direction are used to make the plots of the deflection in the span direction at the different load steps. The results show behavior as expected, see Figure 21, in which the axles of the proof load tandem are indicated as well. As expected, the LVDT under the proof load tandem measures the largest deflections. A similar plot, see Figure 22, can be made in the transverse direction. In the transverse direction, the measurement point closer to the sidewalk is deflecting more at lower load levels, and less at higher load levels. A possible explanation for this observation is that at low load levels, the behavior is less stiff because of the observed cracking on the side of the slab with the sidewalk. The larger stiffness at higher loads can be explained by the presence of stirrups in the sidewalk. The measured strains are fully linear.

Three cracks were followed during the proof load test. LVDT 14 measures a possibly critical shear crack, LVDT 15 measures a longitudinal crack on the bottom of the slab, and LVDT 16 a transverse crack. The results of the crack width for different load levels during the test is plotted, see Figure 23. From this plot, it can be seen that the possible shear crack was not activated by the test. The longitudinal crack was activated and the behavior was mostly linear. The transverse crack was more activated than the longitudinal crack. Note that the maximum increase in crack width for all measured cracks was very small, and that no signs of non-linearity are observed. Finally, the results of the measurements of the reference LVDT are shown, see Figure 24. These strains are compared to the ambient temperature, taken from the published measurements of the Dutch Royal Meteorological Institute (KNMI, 2015) for a location 14 km south of the load testing location, as well as to the average temperature of the bridge deck, which is measured as a part of the ASR monitoring system. It can be seen that the behavior of the temperature of the bridge deck and the ambient temperature is similar. The behavior of the strain is inversely proportional to the temperature. A negative strain represents compression of the LVDT, or elongation of the aluminum strip, which is caused by the increasing temperature. The measured strains thus are as expected, and they are used to correct the strains measured on the bottom of the slab.

Now, the results of the proof load test at a shear-critical position can be revised. The load-displacement diagram is shown in Figure 25. The maximum applied load during the proof load test for shear was 1342 kN, which leads to a maximum load of 1377 kN, taking into account the weight of the jacks and the steel plate. From the load-displacement diagram (Figure 25), it can be seen that the behavior is fully linear and that no signs of non-linearity are observed. Next, the deflection plots in the longitudinal direction are shown in Figure 26. All results are as expected, except for the last measurement point. A more detailed study of the output of this LVDT showed that the results were suddenly shifted to larger strains between 600 kN and 1100 kN, and then shifted back to smaller strains afterwards. This observation is explained by the fact that the LVDT was possibly out of its measurement range, resulting in it being fully compressed, and only being able to move slightly as result of changes in the aluminum measurement frame due to temperature. The deflection plot in the transverse direction is as expected, see Figure 27. The deflection under the sidewalk is slightly less than the deflection in the span, which is expected since the sidewalk is provided with shear reinforcement and thus has slightly stiffer behavior. The measured strains are fully linear. Again, the opening of three cracks was followed with LVDT 14 over a possible shear crack at the same position as the bending moment test, LVDT 15 over a longitudinal crack on the bottom face and LVDT 16 over a transverse crack on the bottom face. The increase in crack width with the applied load is shown in Figure 28. It can be seen that all monitored cracks are slightly activated (note that the maximum crack width is less than 0.025 mm). The increase in crack width in the possible shear crack was small and linear, and did not cause concerns about the structural safety of the tested bridge. The joint did not fully close or cause rotation during the experiment, and the effect of temperature was as discussed for the bending moment test.

10.5.2. Discussion and comparison to international guidelines

In the German guideline (Deutscher Ausschuss für Stahlbeton, 2000), stop criteria are defined. These criteria are derived from the output of the sensors, and indicate when further loading is not permitted. The stop criteria from the German guideline can be used for bending moment. For shear, currently no stop criteria are defined, but research on this topic is carried out (Schacht et al., 2016a). Since shear is a brittle failure mechanism, this failure needs to be avoided, and adequate stop criteria need to be defined. For this purpose, the acoustic emission measurements are used as well, to gain more insight in the internal cracking occurring in the slab during the load test. As no stop criteria for shear are defined, the acoustic emission signals were followed closely to capture signs of increased cracking activity and instable crack development, and the output of all sensors was closely followed to capture signs of changes to the structure and nonlinearity. Moreover, the effect of ASR-damage on the shear capacity is unknown. On one hand, the cracking caused by ASR reduces the uniaxial tensile strength, which is expected to reduce the shear capacity. On the other hand, the restraint of expansion in the direction of the reinforcement creates a prestressing effect on the cross-section, which increases the shear capacity. As such, one has to be extra careful when proof loading a shear-critical viaduct with ASR-damage.

In this section, the stop criteria from the German guideline are analyzed (Deutscher Ausschuss für Stahlbeton, 2000). These stop criteria are derived for buildings, and only consider flexure. Further research on stop criteria is necessary for the application to proof load testing of existing concrete bridges that are shear-critical. The first stop criterion from the German guideline says that the ratio of the residual to maximum deformation is limited to 10%. For the bending moment test, this ratio was 9.7% and for the shear test 9.7%. For both proof load tests, the first stop criterion is thus fulfilled. It must be remarked here that the residual deformation during the proof load test is measured at the moment when the base load level is still acting on the bridge. Unloading to 0 kN is not used during the bridge to keep all sensors activated. Therefore, a recommendation for the development of stop criteria that can be used during a proof load test would be to express the stop criterion for the residual deformation differently. The new expression then would take into account the fact that no full unloading occurs after each load cycle. Moreover, it should be defined if the residual deformation after the *i*-th load cycle is based on the difference between the start of the test and load cycle *i*, or based on the difference between load cycles *i*-1 and *i*.

The second stop criterion from the German guideline considers the strains in the steel. Typically, a bridge owner might not allow the removal of the concrete cover to measure the steel strains. Therefore, this stop criterion is considered as not practical. The third stop criterion considers the strains measured in the concrete. This stop criterion is formulated as follows:

$$\varepsilon_c < \varepsilon_{c,\lim} - \varepsilon_{c0} \tag{1}$$

with ε_c the measured strain in the concrete, $\varepsilon_{c,lim}$ the limiting strain of 800 με, and ε_{c0} the strain caused by the permanent loads. The maximum strain observed in the bending moment experiment is 240 με at LVDT 2. The value of ε_{c0} is determined from the linear finite element program. As a result, the maximum measured strain has to be smaller than 800 με – 38 με = 762 με, and this requirement is fulfilled. The maximum strain in the shear experiment was 224 με at LVDT 2, and the strain caused by the permanent loads is taken from the finite element model as 45 με. The requirement now becomes that 224 με has to be smaller than 800 με – 45 με = 755 με, and this requirement is fulfilled.

The last stop criterion from the German guideline is related to crack width. The guideline has different requirements for existing cracks and newly developed cracks. Since in the two proof load test only existing cracks were monitored, only the stop criteria for existing cracks need to be verified. The first requirement is that the maximum crack width increase during the proof load test, $\Delta w \leq 0.3$ mm. The second requirement considers the residual crack width, after a load cycle, for which holds that the value of the residual crack width $\leq 0.2 \times \Delta w$. An overview of these results is given in Table 5 for the bending moment test and in Table 6 for the shear test. For both cases, it can be seen that the studied crack widths are extremely small. In general, crack widths smaller than 0.05 mm can be neglected (Koekkoek et al., 2016). When neglecting the small crack widths, the conclusion is that none of the cracks was activated nor needs to be considered. When following the German guideline for crack widths regardless of how small they are, it is found that all requirements are fulfilled in the bending moment test. For the shear test, the requirement for the residual crack width is not fulfilled. However, no signs of distress were observed during the proof load test, and it can be recommended to add the requirement that crack widths smaller than 0.05 mm be neglected to the prescribed requirements for maximum and residual crack width. The physical significance of cracks smaller than 0.05 mm is almost non-existent.

10.5.3. Repair recommendations

Both proof load tests on the viaduct Zijlweg were carried out successfully. It was shown experimentally that the viaduct can carry the loads prescribed by the code, using the load factors of the RBK Design load level with an additional 5%. This means that the structure fulfills the same requirements as a designed and newly built structure. The viaduct was monitored closely with a large number of sensors during the experiments, and no signs of distress were found. The final conclusion is that it is safe to keep the viaduct open to all traffic, and that load restrictions and posting are not necessary. This conclusion is important, because the very low uniaxial strength of the concrete with ASR-damage led to discussion about the shear capacity of the viaduct. To prevent durability problems resulting from further cracking caused by ASR, a waterproofing layer on top of the slab was added in 2002. Since moisture is necessary for the ASR gel to expand and cause cracking, providing waterproofing and preventing the ingress of moisture are a good solution to prevent further cracking and future durability problems. Moreover, regular inspections and continued monitoring of the effects of ASR are necessary. Special attention should be paid to the space in the expansion joints, that has become small as a result of the longitudinal expansion of the slab caused by ASR.

10.6. Cost of decision-making based on field assessment

10.6.1. Replacement cost: economic, environmental, and social

In the field of bridge engineering, the current trend is to consider the life cycle cost of the structure (Frangopol et al., 2016). From the past concept of only considering the cost of design and construction, bridge engineering is moving towards a concept of determining the cost of design, construction, inspection, maintenance, strengthening, demolition, and the salvage value (Kim and Frangopol, 2011). These costs are the so-called economic costs. For a full sustainability analysis of a bridge, the environmental and social cost need to be determined as well (Gervasio et al., 2012, Beck et al., 2012). For buildings, rating for sustainability is a common practice. For bridges on the other hand, sustainability analyses are still the topic of research. In the majority if the cases, when tendering a bridge project, it is the offer with the lowest initial cost that is most successful (Beck et al., 2012). In some countries, such as the United Kingdom where the Sustainability Index for Bridges is developed (Hendy and Petty, 2012), bridge authorities are trying to turn the tide. In general, five stages are considered during the life of a structure (Beck et al., 2012):

1. the product stage,
2. the construction process,
3. the usage stage,
4. the end-of-life stage, and

5. the stage identified as “supplementary information beyond the building life cycle”, which contains benefit and loads beyond the system boundary.

In bridges, the operation phase plays a much less important role than in buildings. As a result, the relative importance of the construction stage and end-of-life stage increases. During the end-of-life stage, the total sustainability impact is determined by the demolition processes, transportation of materials, and finally waste processing for reuse, recovery, and recycling.

To assess the sustainability impact of field testing, the economic, environmental, and social impact should be addressed. The Sustainability Index for Bridges from the United Kingdom (Hendy and Petty, 2012) pays attention to the influence on climate change and the use of resources as well. The analysis will be carried out assuming that the superstructure of the viaduct Zijlweg should be replaced, and will then be compared to the result of the field test, which shows that the viaduct still fulfills all requirements. The difficulty in determining the impact on sustainability, is that a number of parameters must be combined. Not all of these parameters can be determined in quantitative terms. Moreover, combining elements from the fields that study the effect on the environment and on society require a basic insight in concepts that are generally not covered in engineering education. An additional challenge is to determine how to weigh the different elements that form part of the full assessment. These different problems should be thought through and analyzed before a certain repair or replacement scheme is selected.

10.6.2. Cost savings through field assessment

To determine the cost savings obtained from the field testing of the bridge, the sustainability cost of replacing the ASR-affected superstructure of the viaduct Zijlweg is studied and compared to the cost of field testing. It must be noted here that the a structure that is found to be sufficient in a field test can still require replacement later during its service life. Moreover, since the topic of the application of proof load testing to shear-critical structures is still under development, it is expected that the cost of proof load testing will decrease significantly once standardization of the procedures is obtained.

First, the economic cost is determined. This cost can be calculated based on the assumption of a cost of 800 – 1000 €/m^2, which is the construction cost for slab bridges in the Netherlands. The viaduct Zijlweg has four spans: two end spans of 10.32 m and two mid spans of 14.71 m. The width is 6.6 m, so that the total area equals 330.4 m^2. The economic cost of replacing the superstructure would thus lie in between 264300 € and 330400 €, assuming that the geometry is not altered. If a bridge that facilitates more than one lane of traffic should be built, then the cost increases, and an additional complication becomes that the existing substructure might not be sufficient to carry the additional loading.

To assess the environmental impact, the Carbon Calculator for Construction Activities is used (Environment Agency, 2016). First, the total volume of concrete needs to be determined. Since the thickness of the slab varies between 550 mm and 850 mm, the average value of 700 mm is used for these exploratory calculations. Multiplying this value with the area of 330.4 m^2 gives a volume of 232 m^3 of concrete, assuming that the design of the replacement bridge will be very similar to the existing bridge. Then, the amount of reinforcement steel needs to be determined. Using the assumption of 120 kg of steel per 1 m^3 of concrete leads to an approximate value of 28 ton. If the distance between reinforcement steel producer and construction site is 75 km, the footprint of the reinforcement steel becomes 41 ton CO_2. Then, for 232 m^3 of concrete with an exposure class of XC4, and assuming a distance of 20 km between the plant and the construction site, the estimated carbon footprint becomes 67 ton CO_2. The effect of transportation of fewer than 8 people on site for 48 weeks gives a footprint of 15 ton CO_2 for the transportation of personnel. The total footprint then becomes 109 ton CO_2. A breakdown of the contributions is shown in Figure 29. In this calculation, a few elements are not considered. The life cycle conversion factors for waste disposal are not considered. The emissions from plant and equipment, which is the fuel consumption on site and the distance over which this fuel is transported, is not considered. The choice of fuel depends on the contractor, and is difficult to estimate a priori. This fuel consumption needs to be split between the plant and equipment used on site, and the site accommodations. For example, a diesel generator can be used to power a mobile plant as well as site offices. For the considered small project, only site offices would be powered. The choice of fuel, and the amount of water used on site, can then be used to calculated this additional contribution to the carbon footprint.

The breakdown shown in Figure 29 is based on Portland cement. Since the largest contribution to the carbon footprint of the construction of the superstructure comes from the concrete, it is interesting to explore the effect of using different types of cement. The types of cement that are analyzed here are: Portland cement (with 6% limestone), Portland fly ash cement (with 28% of fly ash), Portland slag cement (with 35% of ggbs), blastfurnace slag (with 80% of ggbs), and Pozzolanic cement (with fly ash and 45.5% of ggbs). The results of this analysis, and the resulting reduction on the total CO_2 footprint are shown in Table 7. These calculations are based on the assumption that 14% of the weight of the total concrete is the weight of cement. For 232 m^3 of concrete, assuming 2.4 ton/m^3, a weight of 556.8 ton of concrete is found, which equals 78 ton of cement. The effect of this amount of cement is studied in Table 7. The effect of improved concrete mixtures with reduced amounts of cement is not considered here. In Table 7, it becomes clear that significant reductions of the carbon footprint can be achieved when blastfurnace slag (with 80% ggbs) is used: 46% of the carbon emissions can then be saved as compared to the case with Portland cement (with 6% limestone). In the Netherlands, blastfurnace slag is the most common choice for cementitious material, for reasons of availability. The environmental impact of this standard choice is large, and positive.

Finally, the social dimension depends on a large number of aspects, such as visual impact, time delays, job opportunities, and more (Zinke et al., 2012). Currently, the social impact is most often calculated based on the driver delay costs. These driver delay costs are caused by delays resulting from the obstruction of traffic at the construction site, and the need for drivers to use another route, where more congestion is created. These costs depend by and large on the location of the structure. It is important to study these costs, as it has been shown that for bridges in densely-populated areas, the delay costs can be about 9 times higher than the direct economic costs (Zinke et al., 2012). The viaduct Zijlweg serves less than 250000 vehicles per year. If a driver delay cost of 10 €/vehicle/hour is assumed, and a reroute to the next bridge creates a 20-minute delay, the total driver delay cost for a year of demolition and replacement can be estimated at 833000 €, which is 2.8 times larger than the direct economic cost. This first estimate already shows that even for a bridge that is subjected to only local traffic, the social impact caused by driver delays is significant.

For the proof load testing, similar calculations can be carried out. Again, it must be stressed that further standardization and implementation of proof load testing will reduce the costs. Moreover, the practice of proof load testing of shear-critical structures is still in the stage of research, which makes the cost of the minimum required instrumentation and other minimal requirements difficult to estimate.

The economic cost includes the cost of the load application, the cost of material research, the cost of scaffolding and site provisions, and a budget for research on the topic of proof load testing related to this pilot. As such, the economic cost of about 80000 € is a cost that can be reduced once standardization of the procedures is in place. It must be noted as well that this cost includes a budget for research to achieve standardization and optimization, whereas no research budget is required for the option of replacement of the superstructure. The environmental impact is now only caused by the transportation of people to the construction site, and is about 0.3 tCO_2. For the social cost, the driver delays are calculated assuming a price of 10€/vehicle/hour and a detour of 20 minutes. The bridge was closed for five days, so 3425 vehicles were affected, resulting in 11415 € in driver delays. This value is most likely even lower, as the testing was carried out during the summer holidays.

The presented calculation of the sustainability cost shows the important economic, environmental, and social savings that result from an improved assessment of reinforced concrete slab bridges through the use of proof load testing. A replacement of the ASR-affected superstructure would cost on average 297000 €, create carbon emissions of at least 60 ton CO_2 if blasfurnace slag with 80% ggbs is used, and would create significant additional costs caused by driver delays. The successful proof load tests showed that the ASR-affected superstructure still fulfills the requirements with regard to strength, and can carry the prescribed loads without signs of distress. The replacement costs can thus be avoided. The remaining costs of the service life of the existing structure are as determined in the maintenance plan, and these have been budgeted for. A field tests (including a research budget) currently costs about 27% of a replacement, results in negligible CO_2 emissions, and leads to only 1% of the driver delay costs of a replacement scheme. Finalization of the research will lead to standardization and optimization, so that a cost of 5 – 10 % of the replacement cost (for the economic cost) is hoped to be achieved in the future. It can thus be seen that the economic savings of field testing are considerable, but that the positive impact on the environment and the social costs is much larger than on the economic cost. When taking into account a full sustainability analysis, the option of field testing thus becomes even more attractive than when only economic costs are considered.

One final remark with regard to the previous calculations is that the major savings obtained by proof load testing are only valid when the proof load test is successful, i.e. when it can be proven through a proof load test that the tested structure can carry the prescribed live loads, and that replacement is not necessary. For this reason, it is of the utmost importance to identify which bridge structures are good candidates for proof load testing. Especially structures with large uncertainties are interesting in this regard: bridges without plans, bridges that are expected to have large redistribution capacity beyond the codified calculation methods, and bridges where the effect of material degradation on the structural capacity is unknown. Further research to develop clear guidelines on how to select good candidate structures for proof load testing is recommended.

10.7. Future trends

Future research on the topic of field assessment through load testing will focus on the reliability aspects of the determined target proof load, and on determining stop criteria for shear. Currently, several countries (Germany, USA, the Netherlands) are working towards the development or updating of the guidelines for field testing of bridges. Load testing will become more and more important for existing structures, as it is an excellent method to assess the structure in its real conditions. As more of the existing bridges are ageing and the traffic loads and volumes are increasing, more accurate methods for the assessment of existing bridges are necessary.

In the Netherlands, the goal is to develop guidelines for proof load testing of existing bridges that can be followed by the industry. To achieve this goal, more research on the stop criteria for shear is necessary, so that a safe execution of a proof load test when using the guideline is guaranteed. Moreover, faster methods to determine the target proof load and the minimum required number of sensors need to be developed. These actions are necessary to develop a cheap and fast method for proof load testing, through which an existing bridge can quickly be assessed by the industry.

10.8. Summary and conclusions (approx. 200 words)

As a result of the aging bridge stock, assessment of existing bridges becomes increasingly important. Research is carried out to determine if proof load testing can be a cost-effective method for a direct field assessment of an existing bridge. The viaduct Zijlweg was selected as a pilot project, as large cracking caused by ASR-damage raised concerns with regard to the shear capacity of the viaduct. This viaduct is also monitored to study the effect of ASR, and to find out when the elongation has become so large that the functioning of the expansion joint is endangered.

The first goal for proof load testing of the viaduct Zijlweg was to gain more experience in the technique of proof load testing. An extensive sensor plan was developed to monitor the structural response during the proof load tests, and to verify the usefulness of the existing stop criteria from the German guideline. These stop criteria are only valid for flexure, and new stop criteria for shear need to be developed. Moreover, the stop criteria from the German guideline do not take into account the effect of existing cracking on the structure. A second goal of proof load testing of the viaduct Zijlweg was to experimentally show that the viaduct can carry the prescribed loads without signs of distress. This goal was successful, and the cost savings from a sustainability perspective can be calculated.

10.9. Acknowledgements

I'd like to express my gratitude and sincere appreciation to the Dutch Ministry of Infrastructure and the Environment (Rijkswaterstaat) and the Province of Noord Brabant for financing this research work. I'd also like to thank Rutger Koekkoek from Delft University of Technology for all his contributions to the preparation of the test, the finite element modelling, the planning, the on-site work, and the evaluation of the experiment. Thanks to Prof. Dick Hordijk from Delft University of Technology for the coordination of the project and the photographs from the visual inspection he did. I'd also like to acknowledge Dr. Ane de Boer from Rijkswaterstaat for all his input and the fruitful discussions on this topic. Credit also goes to former M.Sc. student Werner Vos of Delft University of Technology, who analysed the data of the ASR-monitoring and patiently drew the map of cracks. The contributions and help of our colleagues Albert Bosman, Dr. Cor van der Veen, and Dr. Yuguang Yang, and of student Arthur Ennouri from Delft University of Technology during planning, execution, and analysis of this proof load test have been of great value and help, and are gratefully acknowledged. I'm also thankful for the fruitful discussions with Frank Linthorst and Danny den Boef of Witteveen+Bos, responsible for the logistics and safety, and with Otto Illing and the late Chris Huissen of Mammoet, responsible for applying the load.

10.10. References

ACI COMMITTEE 437 2013. Code Requirements for Load Testing of Existing Concrete Structures (ACI 437.2M-13) and Commentary Farmington Hills, MA.

AHMED, T., BURLEY, E. & RIGDEN, S. 1998. The static and fatigue strength of reinforced concrete beams affected by alkali-silica reaction. *ACI Materials Journal,* 95**,** 376-388.

AHMED, T., BURLEY, E. & RIGDEN, S. 1999. Effect of alkali-silica reaction on tensile bond strength of reinforcement in concrete tested under static and fatigue loading. *ACI Materials Journal,* 96**,** 419-428.

ASCE/SEI-AASHTO AD-HOC GROUP ON BRIDGE INSPECTION RATING REHABILITATION AND REPLACEMENT 2009. White Paper on Bridge Inspection and Rating. *Journal of Bridge Engineering,* 14**,** 1-5.

BAGGE, N., SAS, G., NILIMAA, J., BLANKSVARD, T., ELFGREN, L., TU, Y. & CAROLIN, A. 2015. Loading to failure of a 55 year old prestressed concrete bridge. *IABSE Workshop.* Helsinki, Finland.

BARKER, M. G. 2001. Quantifying Field-Test Behavior for Rating Steel Girder Bridges. *Journal of Bridge Engineering,* 6**,** 254-261.

BECK, T., FISCHER, M. & PFAFFINGER, M. 2012. Life Cycle Assessment for representative steel and composite bridges. *In:* BIONDINI, F. & FRANGOPOL, D. M. (eds.) *Proceedings of the Sixth*

International Conference on Bridge Maintenance, Safety and Management. Stresa, Lake Maggiore, Italy: Taylor & Francis Group.
CEN 2002. Eurocode – Basis of structural design, NEN-EN 1990:2002 Brussels, Belgium: Comité Européen de Normalisation.
CEN 2003. Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges, NEN-EN 1991-2:2003. Brussels, Belgium: Comité Européen de Normalisation.
CEN 2011. Eurocode 2: Design of concrete structures - Concrete bridges - Design and detailing rules. NEN-EN 1992-2+C1:2011. Brussels, Belgium: Comité Européen de Normalisation.
CODE COMMITTEE 351001 2011a. *Assessement of structural safety of an existing structure at repair or unfit for use - Basic Requirements, NEN 8700:2011 (in Dutch),* Delft, The Netherlands, Civil center for the execution of research and standard, Dutch Normalisation Institute.
CODE COMMITTEE 351001 2011b. Eurocode 1 - Actions on structures - Part 2: Traffic loads on bridges, EN 1991-2/NA:2011. Delft, The Netherlands: Civil engineering center for research and regulation, Dutch Normalization Institute.
COPE, R. J. 1985. Flexural Shear Failure of Reinforced-Concrete Slab Bridges. *Proceedings of the Institution of Civil Engineers Part 2-Research and Theory,* 79**,** 559-583.
COPE, R. J., RAO, P. V. & EDWARDS, K. R. 1983. Shear in skew reinforced concrete slab bridges – analytical and experimental studies – A report to the Department of Transport. Liverpool, UK: University of Liverpool.
DEN UIJL, J. A. & KAPTIJN, N. 2004. Structural consequences of ASR: an example on shear capacity. *Heron,* 47**,** 1-13.
DEUTSCHER AUSSCHUSS FÜR STAHLBETON 2000. DAfStb-Guideline: Load tests on concrete structures. Deutscher Ausschuss fur Stahlbeton,.
ENVIRONMENT AGENCY. 2016. *Carbon Calculator for Construction Activities* [Online]. Available: https://www.ice.org.uk/disciplines-and-resources/best-practice/environment-agency-carbon-calculator-tool.
FABER, M. H., VAL, D. V. & STEWART, M. G. 2000. Proof load testing for bridge assessment and upgrading. *Engineering Structures,* 22**,** 1677-1689.
FRANGOPOL, D. M., SABATINO, S. & DONG, Y. 2016. Bridge Life-Cycle Performance and Cost: Analysis, Prediction, Optimization and Decision Making. *In:* BITTENCOURT, FRANGOPOL & BECK (eds.) *Maintenance, Monitoring, Safety, Risk and Resilience of Bridges and Bridge Networks.* Foz do Iguacu, Brazil.
GERVASIO, H., SILVA, L. S. D., PERDIGAO, V., BARROS, P., ORCESI, A. & NIELSEN, K. 2012. Life Cycle analysis of highway composite bridges. *In:* BIONDINI, F. & FRANGOPOL, D. M. (eds.) *Proceedings of the Sixth International Conference on Bridge Maintenance, Safety and Management.* Stresa, Lake Maggiore, Italy: Taylor & Francis Group.
GIELEN, C., KOLKMAN, M. & VELDHUIZEN, A. C. 2008. Inspection report object 44G-113-01: bridge over highway - Zijlweg (in Dutch).
HADDAD, R. H., SHANNAG, M. J. & AL-HAMBOUTH, M. T. 2008. Repair of reinforced concrete beams damaged by alkali-silica reaction. *Aci Structural Journal,* 105**,** 145-153.
HENDY, C. & PETTY, R. 2012. Quantification of sustainability principles in bridge projects. *In:* BIONDINI, F. & FRANGOPOL, D. M. (eds.) *Proceedings of the Sixth International Conference on Bridge Maintenance, Safety and Management.* Stresa, Lake Maggiore, Italy: Taylor & Francis Group.
KIM, S. & FRANGOPOL, D. M. 2011. Cost-Effective Lifetime Structural Health Monitoring Based on Availability. *Journal of Structural Engineering-Asce,* 137**,** 22-33.
KNMI. 2015. *Uurgegevens van het weer in Nederland* [Online]. Available: http://www.knmi.nl/klimatologie/uurgegevens/datafiles/350/uurgeg_350_2011-2020.zip. [Accessed 26/6/2015.
KOEKKOEK, R., LANTSOGHT, E., YANG, Y., BOER, A. D. & HORDIJK, D. 2016. Defining loading criteria for proof loading of existing reinforced concrete bridges. *fib symposium 2016: Performance-based approaches for concrete structures.* Cape Town, South Africa.
KOEKKOEK, R. T., LANTSOGHT, E. O. L. & HORDIJK, D. A. 2015. Proof loading of the ASR-affected viaduct Zijlweg over highway A59. Delft, The Netherlands: Delft University of Technology.
KOENDERS INSTRUMENTS 2015. Zijlweg monitoring system (in Dutch).
LANTSOGHT, E., YANG, Y., VAN DER VEEN, C., DE BOER, A. & HORDIJK, D. 2016a. Ruytenschildt Bridge: field and laboratory testing. *Engineering Structures,* 128**,** 111-123.
LANTSOGHT, E. O. L., DE BOER, A., VAN DER VEEN, C. & WALRAVEN, J. C. 2013a. Peak shear stress distribution in finite element models of concrete slabs. *In:* ZINGONI, A. (ed.) *Research and Applications in Structural Engineering, Mechanics and Computation.* Cape Town, South Africa: Taylor and Francis.
LANTSOGHT, E. O. L., VAN DER VEEN, C., DE BOER, A. & HORDIJK, D. A. 2016b. Probabilistic prediction of the failure mode of the Ruytenschildt Bridge. *Engineering Structures,* 127**,** 549-558.

LANTSOGHT, E. O. L., VAN DER VEEN , C., DE BOER, A. & HORDIJK, D. A. in press. Collapse test and moment capacity of the Ruytenschildt Reinforced Concrete Slab Bridge *Structure and Infrastructure Engineering*.
LANTSOGHT, E. O. L., VAN DER VEEN, C., DE BOER, A. & WALRAVEN, J. 2014. Influence of Width on Shear Capacity of Reinforced Concrete Members. *ACI Structural Journal,* 111**,** 1441-1450.
LANTSOGHT, E. O. L., VAN DER VEEN, C., DE BOER, A. & WALRAVEN, J. 2015a. One-way slabs subjected to combination of loads failing in shear. *ACI Structural Journal,* 112**,** 417-426.
LANTSOGHT, E. O. L., VAN DER VEEN, C., DE BOER, A. & WALRAVEN, J. C. 2013b. Recommendations for the Shear Assessment of Reinforced Concrete Slab Bridges from Experiments *Structural Engineering International,* 23**,** 418-426.
LANTSOGHT, E. O. L., VAN DER VEEN, C., DE BOER, A. & WALRAVEN, J. C. 2015b. Proposal for the extension of the Eurocode shear formula for one-way slabs under concentrated loads. *Engineering Structures,* 95**,** 16-24.
LANTSOGHT, E. O. L., VAN DER VEEN, C., WALRAVEN, J. & DE BOER, A. 2015c. Experimental investigation on shear capacity of reinforced concrete slabs with plain bars and slabs on elastomeric bearings. *Engineering Structures,* 103**,** 1-14.
LANTSOGHT, E. O. L., VAN DER VEEN, C. & WALRAVEN, J. C. 2013c. Shear in One-way Slabs under a Concentrated Load close to the support. *ACI Structural Journal,* 110**,** 275-284.
LANTSOGHT, E. O. L., YANG, Y., TERSTEEG, R. H. D., VAN DER VEEN, C. & DE BOER, A. 2016c. Development of Stop Criteria for Proof Loading. *IALCCE 2016.* Delft, The Netherlands.
LIN, T. S. & NOWAK, A. S. 1984. Proof Loading and Structural Reliability. *Reliability Engineering,* 8**,** 85-100.
LIU, Y., LU, D. & FAN, X. 2014. Reliability updating and prediction of bridge structures based on proof loads and monitored data. *Construction and Building Materials,* 66**,** 795-804.
MATTA, F., BASTIANINI, F., GALATI, N., CASADEI, P. & NANNI, A. 2008. Distributed Strain Measurement in Steel Bridge with Fiber Optic Sensors: Validation through Diagnostic Load Test. *Journal of Performance of Constructed Facilities,* 22**,** 264-273.
NILIMAA, J., BLANKSVÄRD, T. & TALJSTEN, B. 2015. Assessment of concrete double-trough bridges. *Journal of Civil Structural Health Monitoring,* 2015**,** 29-36.
OLASZEK, P., LAGODA, M. & RAMON CASAS, J. 2014. Diagnostic load testing and assessment of existing bridges: examples of application. *Structure and Infrastructure Engineering,* 10**,** 834-842.
PROJECTTEAM RWS/TNO BOUW 1997. Safety evaluation existing structures: Reinforced concrete bridges (in Dutch).
PROVINCIE NOORD BRABANT 1965. Viadukt in de Zijlweg. 1965.
PUURULA, A. M., ENOCHSSON, O., SAS, G., BLANKSVÄRD, T., OHLSSON, U., BERNSPÅNG, L., TÄLJSTEN, B., CAROLIN, A., PAULSSON, B. & ELFGREN, L. 2015. Assessment of the Strengthening of an RC Railway Bridge with CFRP Utilizing a Full-Scale Failure Test and Finite-Element Analysis. *Journal of Structural Engineering,* 141**,** D4014008:1-11.
PUURULA, A. M., ENOCHSSON, O., SAS, G., BLANKSVÄRD, T., OHLSSON, U., BERNSPÅNG, L., TÄLJSTEN, B. & ELFGREN, L. 2014. Loading to failure and 3D nonlinear FE modelling of a strengthened RC bridge. *Structure and Infrastructure Engineering,* 10**,** 1606-1619.
RAFLA, K. 1971. Empirical formula for calculation of shear capacity of reinforced concrete beams (in German). *Strasse Brucke Tunnel,* 23**,** 311-320.
RIJKSWATERSTAAT 2002. Management and maintenance plan viaduct Zijlweg over highway A59 (in Dutch).
RIJKSWATERSTAAT 2013. Guidelines Assessment Bridges - assessment of structural safety of an existing bridge at reconstruction, usage and disapproval (in Dutch).
RYAN, T. W., MANN, E., E., Z., CHILL, M. & OTT, B. T. 2012. Bridge Inspector's Reference Manual. *In:* US DEPARTMENT OF TRANSPORTATION, F. H. A. (ed.).
SANAYEI, M., REIFF, A. J., BRENNER, B. R. & IMBARO, G. R. 2016. Load Rating of a Fully Instrumented Bridge: Comparison of LRFR Approaches. *Journal of Performance of Constructed Facilities,* 2016**,** 2.
SCHACHT, G., BOLLE, G., CURBACH, M. & MARX, S. 2016a. Experimental Evaluation of the shear bearing safety (in German). *Beton- und Stahlbetonbau,* 111**,** 343-354.
SCHACHT, G., BOLLE, G. & MARX, S. 2016b. Load testing - international state of the art (in German). *Bautechnik,* 93**,** 85-97.
SCHMIDT, J. W., HANSEN, S. G., BARBOSA, R. A. & HENRIKSEN, A. 2014. Novel shear capacity testing of ASR damaged full scale concrete bridge. *Engineering Structures,* 79**,** 365-374.
SIEMES, T., HAN, N. & VISSER, J. 2002. Unexpectedly low tensile strength in concrete structures. *Heron,* 47**,** 111-124.
TALLEY, K. G. 2009. *Assessment and Strengthening of ASR and DEF Affected Concrete Bridge Columns.* Ph.D., The University of Texas at Austin.
TNO DIANA 2012. Users Manual of DIANA, Release 9.4.4. Delft, The Netherlands.
VELÁZQUEZ, B. M., YURA, J. A., FRANK, K. H., KREGER, M. E. & WOOD, S. L. 2000. Diagnostic load tests of a reinforced concrete pan-girder bridge. Austin, TX, USA: The University of Texas at Austin.

WITTEVEEN+BOS 2014. Material research bridges, case 31084913: 44G-113-01: viaduct Zijlweg (in Dutch). Deventer.

YANG, Y. & HORDIJK, D. A. 2015. Acoustic Emission Measurement and Analysis on Zijlwegbrug. Delft, The Netherlands: Delft University of Technology.

ZINKE, T., UMMENHOFER, T., PFAFFINGER, M. & MENSINGER, M. 2012. The social dimension of bridge sustainability assessment - Impacts on users and the public. *In:* BIONDINI, F. & FRANGOPOL, D. M. (eds.) *Proceedings of the Sixth International Conference on Bridge Maintenance, Safety and Management.* Stresa, Lake Maggiore, Italy: Taylor & Francis Group.

List of figures

List of tables

Table 1.

Safety level	β	Reference period	γ_{sw}	γ_{sd}	γ_{LL}
ULS Eurocode	4.3	100 years	1.35	1.35	1.50
RBK Design	4.3	100 years	1.25	1.25	1.50
RBK Reconstruction	3.6	30 years	1.15	1.15	1.30
RBK Usage	3.3	30 years	1.15	1.15	1.25
RBK Disapproval	3.1	15 years	1.10	1.10	1.25
SLS Eurocode	1.5	50 years	1.00	1.00	1.00

with

β the associated reliability index

γ_{sw} the load factor on the self-weight

γ_{sd} the load factor on the superimposed load

γ_{LL} the load factor on the live load

Table 2.

Safety level	γ_{sw}	γ_{sd}	γ_{LL}	γ_{proof}
ULS Eurocode	1.10	1.35	1.50	1.00
RBK Design	1.10	1.25	1.50	1.00
RBK Reconstruction	1.10	1.15	1.30	1.00
RBK Usage	1.10	1.15	1.25	1.00
RBK Disapproval	1.10	1.10	1.25	1.00
SLS Eurocode	1.00	1.00	1.00	1.00

with

γ_{sw} the load factor on the self-weight

γ_{sd} the load factor on the superimposed load

γ_{LL} the load factor on the live load

γ_{proof} the load factor for the proof load tandem

Table 3.

Safety level	$P_{tot,bending}$ (kN)	$P_{tot,shear}$ (kN)
ULS Eurocode	1259	1228
RBK Design	1257	1228
RBK Reconstruction	1091	1066
RBK Usage	1050	1027
RBK Disapproval	1049	1025
SLS Eurocode	815	791

Table 4.

Name	Range (mm)	Application
LVDT1	10	Strain over 1 m
LVDT2	10	Strain over 1 m
LVDT3	10	Strain over 1 m
LVDT4	10	Reference for change in temperature
LVDT5	20	Deflection of the slab (on a longitudinal line)
LVDT6	20	Deflection of the slab (on a longitudinal line)
LVDT7	20	Deflection of the slab (on a longitudinal line)
LVDT8	20	Deflection of the slab (on a longitudinal line)
LVDT9	10	Displacement of the joint
LVDT10	10	Displacement of the joint
LVDT11	10	Displacement of the joint
LVDT12	10	Displacement of the joint
LVDT13	10	Deflection of the slab (on a longitudinal line)
LVDT14	10	Crack width
LVDT15	10	Crack width
LVDT16	10	Crack width
Laser1	100	Deflection of the slab (on a transverse line)
Laser2	20	Deflection of the slab (on a transverse line)
Laser3	20	Deformation of support (N)
Laser4	20	Deformation of support (N)
Laser5	100	Deformation of support (S)
Laser6	100	Deformation of support (S)

Table 5.

	Measured Δw (mm)		$0,2 \times \Delta w$ (mm)
	during proof loading	after proof loading	
LVDT14	0,0163	0,0147	0,00326
LVDT15	0,0248	0,0117	0,00496
LVDT16	0,0183	0,0061	0,00366

Table 6.

	Measured Δw (mm)		$0.2 \times \Delta w$ (mm)
	during proof loading	after proof loading	
LVDT14	0.0163	0.0147	0.00326
LVDT15	0.0248	0.0117	0.00496
LVDT16	0.0183	0.0061	0.00366

Table 7.

Type of cement	**CO_2 emission (tCO_2e/ton)**	**Total CO_2 cement (ton)**	**Total CO_2 superstructure (ton)**	**Saving (%)**
Portland cement	0.88	68.60	110.2	-
Portland fly ash cement	0.67	52.23	93.83	14.9
Portland slag cement	0.62	48.33	89.93	18.4
Blastfurnace slag	0.23	17.93	59.53	46.0
Pozzolanic cement	0.51	39.76	81.36	26.2