

**Towards standardisation of proof load testing
pilot test on viaduct Zijlweg**

Lantsoght, Eva Olivia Leontien; Koekkoek, Rutger T.; Hordijk, Dick; de Boer, A.

DOI

[10.1080/15732479.2017.1354032](https://doi.org/10.1080/15732479.2017.1354032)

Publication date

2017

Document Version

Submitted manuscript

Published in

Structure and Infrastructure Engineering

Citation (APA)

Lantsoght, E. O. L., Koekkoek, R. T., Hordijk, D., & de Boer, A. (2017). Towards standardisation of proof load testing: pilot test on viaduct Zijlweg. *Structure and Infrastructure Engineering*, 14 (2018)(3), 365-380. <https://doi.org/10.1080/15732479.2017.1354032>

Important note

To cite this publication, please use the final published version (if applicable).
Please check the document version above.

Copyright

Other than for strictly personal use, it is not permitted to download, forward or distribute the text or part of it, without the consent of the author(s) and/or copyright holder(s), unless the work is under an open content license such as Creative Commons.

Takedown policy

Please contact us and provide details if you believe this document breaches copyrights.
We will remove access to the work immediately and investigate your claim.

1 This is an Accepted Manuscript of an article published by Taylor & Francis Group in
2 Structure and Infrastructure Engineering on 26 Jul 2017, available online:
3 <http://dx.doi.org/10.1080/15732479.2017.1354032>
4

5 **Towards standardization of proof load testing: pilot test on viaduct Zijlweg**
6
7

8 Eva Olivia Leontien Lantsoght*, Researcher, Concrete Structures, Delft University of

9 Technology, Stevinweg 1, 2628CN Delft, the Netherlands, Tel: +31152787449,

10 E.O.L.Lantsoght@tudelft.nl

11 Full Professor, Universidad San Francisco de Quito, Diego de Robles y Pampite, Sector

12 Cumbaya, EC 170157 Quito, Ecuador, Tel: +5932297-1700 ext. 1186, elantsoght@usfq.edu.ec

13 Rutger T. Koekkoek, Researcher, Concrete Structures, Delft University of Technology, 2628CN

14 Delft, The Netherlands, E-mail: R.T.Koekkoek@tudelft.nl

15 Dick Hordijk, Full Professor, Concrete Structures, Delft University of Technology, Stevinweg 1,

16 2628CN Delft, the Netherlands, Tel: +31152784434, D.A.Hordijk@tudelft.nl

17 Ane de Boer, Senior Advisor, Rijkswaterstaat, Department of Infrastructure – Section Bridges

18 and Viaducts, Ministry of Infrastructure and the Environment, H12, P.O. Box 24057, 2502MB

19 Utrecht, the Netherlands, Tel: +31887972083; ane.de.boer@rws.nl
20
21
22

23 **Acknowledgement**

24 This work was supported by the Province of Noord Brabant and the Dutch Ministry of
25 Infrastructure and the Environment (Rijkswaterstaat).

26
27 *Corresponding author

1 **Abstract**

2 Proof load tests of bridges can be very useful for structures with a lack of information, or for
3 structures of which the effect of material degradation is difficult to assess. Contrary to diagnostic
4 load testing, proof load testing is not well-defined in current standards in terms of required load
5 and analysis of measurements. The risk related to the high loads used in proof load testing
6 requires standardization for these tests. The paper highlights important considerations for proof
7 load testing, that will lead to the development of guidelines in the Netherlands, by illustrating a
8 pilot study on the viaduct Zijlweg in the Netherlands. This reinforced concrete bridge rates too
9 low in shear. Topics of interest are the required load that the bridge has to withstand to be
10 approved by the load test, and the interpretation of the measurements during the test to avoid
11 permanent damage to the structure. These measurements were compared to the stop criteria from
12 existing codes for buildings, to see if recommendations for the use with bridges can be
13 formulated. The final result of the test on this case study is that the capacity of the viaduct is
14 proven to be sufficient for shear and bending moment.

15

16 **Keywords**

17 concrete bridges; existing bridges; instrumentation; load testing; proof load testing; slabs

18

1 **1 Introduction**

2 In the Netherlands, a large number of existing bridges are found not to fulfil the code
3 requirements upon assessment. In particular, the shear capacity of a large number of reinforced
4 concrete slab bridges (Lantsoght, van der Veen, de Boer, & Walraven, 2013) is subjected to
5 discussion. The reason for this discussion is twofold: these bridges are reaching the end of their
6 originally devised service life on one hand, and on the other hand, they were designed for lower
7 demands with regard to the live loads. Moreover, the recently introduced NEN-EN 1992-1-
8 1:2005 (CEN, 2005) allow for smaller shear capacities than the previously described national
9 codes. Low ratings of slab bridges are also reported in the United States (Davids, Poulin, &
10 Goslin, 2013).

11
12 In line with the approach of Levels of Approximation as introduced in the *fib* Model Code
13 2010 (fib, 2012), for the assessment of the existing concrete bridges in the Netherlands (mostly
14 reinforced concrete solid slab bridges), Levels of Assessment (Lantsoght, De Boer, & Van der
15 Veen (2017)) have been developed:

- 16 1. Level of Assessment I: a spreadsheet-based tool, the Quick Scan (Vergoossen,
17 Naaktgeboren, 't Hart, De Boer, & Van Vugt, 2013), that quickly identifies which cross-
18 sections need further study.
- 19 2. Level of Assessment II: determination of the governing stresses with a linear finite
20 element program (Lantsoght, de Boer, Van der Veen, & Walraven, 2013).
- 21 3. Level of Assessment III: analyzing the structure with a nonlinear finite element analysis
22 (Rijkswaterstaat, 2012), or with a probabilistic analysis (Steenbergen, de Boer, & van der
23 Veen, 2011).

1 4. Level of Assessment IV: proof loading of the structure (Koekkoek, Lantsoght, Yang, de
2 Boer, & Hordijk, 2016).

3 Two types of load testing, each with different goals, can be carried out in practice. The first type
4 is diagnostic load testing, which has as its main goal to verify assumptions used in analytical
5 models, for example with regard to transverse distribution or stiffness. The measured results of
6 the diagnostic load test are then used to update the analytical model, which then, in turn, is used
7 to provide an updated rating factor. The second type is proof load testing, which is used to
8 experimentally show that a structure fulfils the requirements with regard to being able to sustain
9 the prescribed loads without signs of distress. As proof load testing involves high load levels,
10 there is a risk of damaging the structure or causing a collapse. Since shear is a brittle failure
11 mode, proof load testing for shear is not permitted by any of the current codes. Additionally, a
12 structure subjected to a proof load test needs to be closely monitored, and the structural response
13 needs to be analyzed in terms of stop criteria. Stop criteria are criteria based on the measured
14 structural response, which indicate that irreversible damage can occur if loading past the point of
15 reaching a stop criterion is carried out.

16
17 Load tests cannot answer questions with regard to the ultimate capacity of the studied structure.
18 For this purpose, collapse tests are necessary. One example of a collapse test of an existing
19 reinforced concrete slab bridge is the test on the Ruytenschildt Bridge (Lantsoght, Van der Veen,
20 De Boer & Hordijk, 2016).

21 This paper discusses a pilot proof load test, and focuses on the determination of the
22 required load to approve a bridge and on the required measurements during the test and
23 associated stop criteria. The pilot was carried out on viaduct Zijlweg, a reinforced concrete solid

1 slab bridge. The rating for shear for this bridge was too low, caused by an expected reduction in
2 the shear capacity due to alkali-silica reaction damage. The ultimate limit states of bending
3 moment and shear were studied through a proof load test. Proof load testing for shear is a new
4 area of research, since no existing code permits proof load testing for a brittle failure mode such
5 as shear.

6 Safety against failure and sufficient capacity are related to a reliability index. In the
7 Netherlands, different safety levels are used for the assessment of existing structures, as given in
8 the Guidelines for the Assessment of Reinforced Concrete Bridges of the Ministry of
9 Infrastructure and the Environment (Rijkswaterstaat, 2013) and in the Dutch code NEN
10 8700:2011 (Code Committee 351001, 2011a). An overview of these safety levels is given in
11 Table 1. The value of γ_{sw} has been modified from the value prescribed in the codes for
12 assessment. This modification results in a lower value of γ_{sw} that can be used in combination
13 with proof load testing. The reason why the value of γ_{sw} can be lowered, is that at the moment of
14 proof load testing, the self-weight of the bridge can be considered as a deterministic value. The
15 only element that remains to determine the load factor is then the model factor.

16

17 **2 Literature review**

18 ***2.1 Proof load testing for shear in the view of the current standards and guidelines***

19 In Germany, proof load testing is mostly used for buildings, and a guideline originally developed
20 for concrete buildings of which the flexural capacity has to be verified exists (Deutscher
21 Ausschuss für Stahlbeton, 2000). In Germany, research on proof load testing for shear has
22 started (Schacht, Bolle, Curbach, & Marx, 2016). In the USA, a guideline exists for diagnostic
23 and proof load testing of bridges (NCHRP, 1998), and these recommendations are also adopted

1 in the Manual for Bridge Evaluation (AASHTO, 2011). The target proof load is taken as the
2 vehicle the bridge needs to be rated for, multiplied with a certain factor (usually 1.4). This
3 method cannot be directly translated to European practice, since no Eurocodes exist yet, adopted
4 by all European countries, for the assessment of existing bridges, and that the design tandem
5 used in the design code is not a direct representation of a certain vehicle type. Most attention in
6 the NCHRP guideline is geared towards diagnostic load tests. For proof load tests, no stop
7 criteria are defined and testing for shear is not permitted. Additionally, in the USA ACI code
8 437.2M-13 for buildings (ACI Committee 437, 2013) describes proof load tests. ACI 437.2M-13
9 allows monotonic and cyclic loading protocols, and defines stop criteria (called acceptance
10 criteria). The required load is determined based on a load combination for existing structures,
11 which permits lower load factors than in the load combination for design. However, proof load
12 testing for shear is not permitted.

13 **2.2 *Effect of alkali-silica reaction***

14 *2.2.1 Introduction to alkali-silica reaction*

15 The viaduct Zijlweg has material damage caused by alkali-silica reaction (ASR), which led to
16 discussions about its structural capacity. ASR is the reaction between the reactive (glasslike)
17 silica in some aggregates and alkali from the cement paste (Neville, 2012). The result of this
18 reaction is a gel of calcium silicate hydrate. Upon contact with water, this gel will expand, and
19 this expansion causes stresses in the concrete. If these stresses exceed the tensile strength of the
20 concrete, cracking occurs, and the structural capacity is reduced. The effect of ASR on the
21 concrete strength is typically expressed as a reduction as a function of the amount of free
22 expansion of the concrete (Siemes, Han, & Visser, 2002). The reduction is larger for the tensile
23 strength than for the compressive strength, since tensile strength is governed by the weakest link.
24 In the literature, the reduction of the tensile strength due to ASR is reported to vary between 5%

1 (Ahmed, Burley, & Rigden, 1999) and 82% (Siemes et al., 2002). For reinforced concrete
2 structures, the expansion is limited in the direction of reinforcement bars, but not in the vertical
3 direction when no transverse reinforcement is available. As a result, cracking occurs parallel to
4 the reinforcement.

5 2.2.2 *Effect of ASR on structural capacity*

6 Since bending moment capacity is governed by crushing of the concrete (failure of concrete in
7 compression), ASR is expected to have a small influence on the bending moment capacity, as
8 reflected in some experiments (Giaccio, Zerbino, Ponce, & Batic, 2008; Talley, 2009). Other
9 experiments (Haddad, Shannag, & Al-Hambouth, 2008) reported a reduction of the flexural
10 capacity of beams with ASR-damage of 11% as compared to undamaged beams. However, the
11 cracking moment and stiffness of the damaged beam increased, which can be explained by the
12 fact that the restraint of the ASR-expansion can induce prestressing into a section. In Dutch
13 practice, the effect of ASR-damage on the bending moment capacity is not considered for rating
14 purposes. In terms of the shear capacity of beams with ASR damage as compared to beams
15 without damage, conflicting results can be found in the literature. In some cases (Ahmed, Burley,
16 & Rigden, 1998), the shear capacity is found to be larger in the beam with ASR damage. This
17 increase is attributed to the prestressing effect of ASR. In a literature survey (Schmidt, Hansen,
18 Barbosa, & Henriksen, 2014), it was also reported that some experiments indicate that ASR
19 damage reduces the shear capacity will others find an increase up to 78%. For beams sawn from
20 ASR-affected viaducts (den Uijl & Kaptijn, 2002), it was found that the shear capacity of the
21 beams was 77% of the capacity of undamaged beams. A 25% reduction of the capacity in shear
22 was thus recommended, and this recommendation is followed in the Netherlands for the rating of
23 structures with ASR-damage.

1 2.2.3 *Prevalence of ASR in the Dutch road network*

2 The number of structures (bridges, tunnels and locks) affected by alkali-silica reaction is
3 estimated to be between 40 and 50 in the Netherlands, including a series of bridges in and over
4 the highway A59, of which the viaduct Zijlweg is part. Deleterious ASR mainly occurs in
5 structures made with Portland cement, and generally is much less prevalent in structures with
6 blended cements with high slag contents. The aggregates that contain reactive silica leading to
7 ASR are porous chert, chalcedony, and impure sandstones that often react as porous chert
8 (Nijland & Siemes, 2002). Eighteen ASR-affected viaducts in the Netherlands are monitored
9 (Borsje, Peelen, Postema, & Bakker, 2002), including the viaduct Zijlweg.

10 2.2.4 *Testing of ASR-damaged viaducts*

11 A number of viaducts with ASR-damage have been load tested in the past by diagnostic load
12 tests and for a critical position for bending moment for all but one case. Examples of tested
13 bridges include the Hanshin expressway in Japan, viaducts in the A26 in France, a double deck
14 road structure in South Africa, and bridges in Denmark. For the French viaducts, which had been
15 load tested prior to opening, the reduction in stiffness caused by ASR was found to be no larger
16 than 10% (Talley, 2009). The Danish viaducts were studied with regard to their shear capacity,
17 and no decrease in capacity was found (Schmidt et al., 2014).

18

19 **3 Description of viaduct Zijlweg**

20 **3.1 *History of the viaduct Zijlweg***

21 In this paper, the determination of the required load and the analysis of the measurements will be
22 analyzed based on the pilot proof load test on the viaduct Zijlweg. The viaduct Zijlweg (Figure
23 1) crosses the highway A59 in the province of Brabant, the Netherlands. It is a continuous
24 reinforced concrete solid slab bridge of four spans with a 14.4° skew angle, and was built in 1965

1 for a design life of 80 years. The available information of the bridge includes the original design
2 calculations, reports about the ASR damage and monitoring, and recent inspection reports, see
3 Table 2. The ASR damage was reported in 1997 (Rijkswaterstaat, 1997), together with a very
4 small uniaxial tensile capacity and extensive cracking damage. As a result, the shear capacity
5 was questioned, and an assessment for shear gave Unity Checks (ratio of shear stress caused by
6 occurring loads to shear capacity) of up to 5.4, indicating a serious lack of shear capacity. A
7 reassessment with a finite element model and a better estimate of the material properties brought
8 the Unity Check down to maximum 1.31, indicating still insufficient shear capacity. Therefore, a
9 proof load test as a pilot for the development of standards was carried out, in order to explore the
10 feasibility of proof load testing at a shear-critical position. The viaduct was proof loaded on the
11 17th of June 2015.

12 **3.2 State of viaduct Zijlweg prior to proof load test**

13 The viaduct Zijlweg crosses the A59 highway in the North-South direction. The lengths of the
14 end spans are 10.32 m and of the mid spans 14.71 m (Figure 2a). The width of the viaduct is 6.60
15 m, with a carriageway of 4.00 m (Figure 2b) accommodating a single traffic lane. Sidewalks of
16 1.3 m are available on both edges of the width. The span subjected to proof loading is span 4,
17 between supports 4 and 5 (Figure 2a), since this span is not directly above the highway, so that
18 traffic restrictions are not necessary. The thickness of the slab varies from 550 mm at support 5
19 to 850 mm at support 4, with a radius of curvature of $r = 150$ m. The reinforcement in span 4 is
20 shown in Figure 3.

21 The concrete compressive strength is based on results of drilled cores (Witteveen+Bos,
22 2014), and is determined as $f_{c,cube,m} = 44.4$ MPa (average cube compressive strength) and $f_{ck} =$
23 24.5 MPa (characteristic concrete cylinder compressive strength). The properties of the

1 reinforcement steel are not available. The reinforcement drawings use symbols for plain bars,
2 indicating that QR22 (with a characteristic yield strength $f_{yk} = 220$ MPa) or QR24 ($f_{yk} = 240$
3 MPa) was used.

4 A visual inspection prior to the proof load test showed deterioration on the top deck, limited
5 to the edge of the sidewalk, map cracking on the bottom of the slab, and an almost fully closed
6 expansion joint as the result of the expansion by ASR. All cracks on the bottom and side faces
7 were marked before the experiment, see Figure 4.

8 **3.3 *Estimated capacity***

9 One step that is suggested for the preparation of proof load tests, is to estimate the capacity of the
10 bridge in order to evaluate the relation between the loads that will be applied and the estimated
11 average capacity. Note that for this purpose all resistance factors are taken equal to one and the
12 average expressions for the capacity are used. For bending moment, the behavior and capacity
13 are estimated by developing the moment-curvature diagrams, both for QR22 and QR24 steel. A
14 good estimate for the average maximum load on slab bridges can be estimated with the Extended
15 Strip Model (Lantsoght, van der Veen, de Boer, & Alexander, 2017). Since this calculation
16 depends on the position of the load, it needs to be developed in parallel with the calculations for
17 the development of the critical position of the load.

18

19 **4 Determination of the target proof load**

20 **4.1 *Current practice***

21 For proof load testing on bridges, in the United States the load of the vehicle for which the
22 bridge needs to be rated through the load test is multiplied with a certain factor for the
23 determination of the target proof load. The loading on the vehicle corresponds to the factored

1 load, and the magnification factor used to find the target proof load is standard 1.4 (NCHRP,
2 1998; AASHTO, 2011). In the past, the proof load was taken as twice the maximum allowable
3 load (Saraf, Nowak, & Till, 1996) to demonstrate safe performance of the bridge. If proof load
4 testing has as its goal to approve the passing of legal loads, a factor of 1.8 can be applied to the
5 maximum legal load. The large loads required for proof load testing have been achieved by using
6 military vehicles (Varela-Ortiz, Cintrón, Velázquez, & Stanton, 2010), by directly applying dead
7 weights such as steel profiles on the bridge (Olaszek, Łagoda, & Casas, 2014), or by using an
8 external structure (Schwesinger & Bolle, 2000). In Europe, no Eurocodes for the assessment of
9 existing structures are available, and no vehicles for which bridges should be rated are available.
10 Assessment of existing structures is done according to national codes. It has been suggested
11 (Casas & Gómez, 2013) to use WIM data for the development of the required load factors for
12 proof loading. These recommendations could be extended to consider all safety levels (Table 1)
13 defined in the Dutch national codes and guidelines, by repeating the calculations for all
14 reliability indices considered in the Dutch national codes and guidelines. To abide to the codes,
15 the load combinations from Table 1 thus have to be applied.

16 **4.2 Approach developed in the Netherlands**

17 Since the load combinations from Table 1 have to be used in the Netherlands, an equivalent
18 proof load representing the factored live loads should be determined in addition to the permanent
19 loads. The approach used in the Netherlands is based on the basic idea that the sectional moment
20 or shear caused by the permanent loads and the proof load should be identical to the sectional
21 moment or shear caused by the considered load combination. This analysis is based on a linear
22 finite element model. In the Netherlands, the considered live loads are as defined by Load Model
23 1 from NEN-EN 1991-2:2003 (CEN, 2003), which describes a distributed lane load and a design

1 tandem. The distributed lane load is placed over a notional lane width of 3 m, and should be
2 applied in a checkerboard pattern to find the most unfavorable situation. Outside of the lanes, on
3 the remaining parts of the carriageway, a distributed load of 2.5 kN/m^2 is applied, and on the
4 sidewalk a load of 5 kN/m^2 corresponding to pedestrian loading,

5 For the design tandem, four wheel prints with 150 kN each distributed over 4 elements
6 (distribution of wheel print of $400 \text{ mm} \times 400 \text{ mm}$ to the slab mid-depth) are used. The tandem is
7 centered in the notional lane of 3 m. When using a linear finite element model, the wheel print
8 can be distributed under 45° to the slab mid-depth. For the proof load tandem, in practice, a
9 wheel print of $230 \text{ mm} \times 300 \text{ mm}$ is used, which needs to be modeled as well at the slab mid-
10 depth.

11 To find the required magnitude and position of the proof load tandem for a proof load test
12 verifying bending moment, the following procedure is used:

- 13 1. All permanent loads and the loads from Load Model 1 are applied to the bridge, with
14 their respective load factors. The position of the design tandems is moved along the span
15 to find the position that results in the largest moment: the critical position.
- 16 2. All permanent loads (with their respective load factors) and the proof load tandem (no
17 load factor) are applied to the slab. The proof load tandem is placed at the critical
18 position found from the previous analysis. The load on the proof load tandem is
19 increased until the same value is found for the largest moment as from the previous
20 analysis. The position and load magnitude are recorded for the load test.

21 A similar recommendation is developed to find the target proof load to test for shear. Instead of
22 finding the maximum moment, the maximum shear is now sought and used as the governing
23 criterion. The peak shear is distributed over $4d$, with d the effective depth, as recommended

1 when using linear finite element models for shear assessment (Lantsoght, A. de Boer, van der
2 Veen, 2013). The critical position is at $2.5d$ from the support (Rijkswaterstaat, 2013), as
3 determined based on extensive experiments on slabs failing in shear (Lantsoght, van der Veen,
4 De Boer, & Walraven, 2014; Lantsoght, van der Veen, de Boer, & Walraven, 2015; Lantsoght,
5 van der Veen, Walraven, & de Boer, 2015; Lantsoght, van der Veen, & Walraven, 2013). The
6 highest shear concentrations occur when loading at the obtuse corner (Cope, 1985), so that for
7 skewed bridges the proof load tandem should be applied in the obtuse corner.

8 In the Netherlands, the research regarding proof load testing has focused so far on bridges
9 with a small width (one or two lanes). For these cases, the loading case with a single proof load
10 tandem is representative of the live loads. For wider viaducts, loading combinations should be
11 compared as well. Wider viaducts are the topic of future research.

12 ***4.3 Verification with proof load test on viaduct Zijlweg***

13 For the determination of the target proof load for the viaduct Zijlweg, a model using shell
14 elements was developed in TNO Diana (TNO DIANA, 2012). The slab was modelled as having
15 a constant thickness, and the extra weight caused by the parts with a larger depth was added as
16 an external load. For the viaduct Zijlweg, the thickness of the asphalt layer was taken as 10 cm.
17 Since the viaduct carries less than 250,000 vehicles per year (category: regional road), the Dutch
18 National Annex NEN-EN 1991-2/NA:2011 (Code Committee 351001, 2011b) prescribes a
19 reduction factor of 0.97 for the live loads from Load Model 1 and of 0.90 for the remaining area
20 of the lane. No reduction factor is used for the pedestrian traffic on the sidewalk.

21 For bending moment, the critical face-to-face distance between the first axle of the tandem and
22 the support is found as 3382 mm based on the procedure proposed in §4.2. The magnitude of the
23 load on the proof load tandem depends on the considered safety level, see Table 1. An overview

1 of the resulting maximum loads on the proof loading tandem is given in Table 3. It can be seen
2 that a proof load of 128 metric ton is required. For shear, the position at $2.5d$ for the required
3 face-to-face distance between the load and the support becomes 1208 mm, and the resulting
4 loads necessary in the load test for shear are then shown in Table 3. For shear, the target proof
5 load is 125 metric ton. The values for the target proof loads for shear and bending moment are
6 considerably larger than the target proof loads that are applied in the United States, representing
7 1.4 times the vehicle the bridge should be rated for. For these cases, target proof loads between
8 30 and 90 metric ton have been reported. However, these results are not directly comparable. In
9 fact, the difference is because the live load factor in the AASHTO MBE has been calibrated
10 taking into account the particular loading conditions existing in a proof load test, assuming that
11 the self-weight and superimposed loads are perfectly known. Moreover, proof load testing for
12 shear is a new development.

13 In the proof load test, the load is applied by placing heavy weights on a steel bridge over the
14 actual concrete bridge, and by then transferring the load gradually by using jacks, see Figure 5
15 for a step-by-step representation of how the loading system is built up. An overview of the
16 positions of the jacks in each load test is shown in Figure 6. Every load level is applied three
17 times to verify if the behavior of the structure changes at the same load level, indicating non-
18 linearity. Five load levels were defined: a low load level of 40 metric ton to verify the working of
19 all sensors, the load corresponding to the Serviceability Limit State, two load steps building up to
20 the final load, and then the final load, corresponding to the safety level of 1.05 times RBK
21 Design (Table 1). An extra 5% is added to cover uncertainties with regard to local variabilities in
22 the structure, and the conversion of the Eurocode live loads to the proof load tandem. This factor
23 can be considered as a model factor. In between the load cycles, a minimum load of 100 kN is

1 applied, to keep all measurements activated. The loading schemes, as executed, are shown in
2 Figure 7a for the bending moment position and in Figure 7b for the shear position. The
3 maximum applied load during the bending moment test was 1332.4 kN. Additionally, the weight
4 of the steel plate of 20.7 kN and jacks of 15.1 kN has to be added, resulting in a maximum
5 applied load of 1368.2 kN. The maximum load in the shear test, including the weight of the steel
6 plate and jacks was 1377.3 kN.

7

8 **5 Determination of required measurements and stop criteria**

9 **5.1 Recommendations from current guidelines**

10 The German guideline for load testing (Deutscher Ausschuss für Stahlbeton, 2000) uses a cyclic
11 loading protocol of three load levels with at least one cycle per level. The first stop criterion
12 limits the concrete strain:

$$13 \quad \varepsilon_c < \varepsilon_{c,lim} - \varepsilon_{c0} \quad (0)$$

14 The measured strain ε_c should be smaller than the limiting strain $\varepsilon_{c,lim}$ (0.8 ‰ if the compressive
15 strength is larger than 25 MPa) minus the strain ε_{c0} due to the permanent loads. The second
16 criterion is a limiting strain in the steel, which prescribes that the measured strain ε_{s2} should be
17 smaller than a fraction of the yield strain (determined based on the mean yield strength of the
18 steel f_{ym} and the Young's modulus of the steel E_s) minus the strain in the steel ε_{s02} due to the
19 permanent loads:

$$20 \quad \varepsilon_{s2} < 0.7 \frac{f_{ym}}{E_s} - \varepsilon_{s02} \quad (0)$$

21 The third criterion is based on the crack width w for new cracks and on the increase in
22 crack width, Δw , for existing cracks: new cracks can maximum reach 0.5 mm, of which 30%
23 residual crack width is allowed, and existing cracks can increase maximum to 0.3 mm, of which

1 20% residual crack width is allowed. The fourth acceptance criterion is that no non-linear
2 behavior can occur, typically evaluated on the load-deflection graph, or if more than 10%
3 permanent deformation is found after removing the load. The fifth criterion limits the strains in
4 the shear span of beams without shear reinforcement, and is thus not relevant for slabs. A test
5 also needs to be stopped when the measurements indicate critical changes in the structure, when
6 the stability of the structure is endangered, and when critical displacements occur at the supports.

7 The next set of stop criteria comes from ACI 437.2M-13 (ACI Committee 437, 2013),
8 which allows monotonic and cyclic load tests for buildings. For the cyclic loading protocol, three
9 load levels should be studied, with two cycles per load level. The first two cycles study the
10 serviceability conditions, and the last two cycles study the full test load. The first stop criterion is
11 that the structure should show no evidence of failure. The second stop criterion is the deviation
12 from linearity index, I_{DL} , with the angles α with respect to the origin of the load-displacement
13 diagram:

$$14 \quad I_{DL} = 1 - \frac{\tan(\alpha_i)}{\tan(\alpha_{ref})} \leq 0.25 \quad (1)$$

15 The angle α_i is the angle of the line through the origin of the load-displacement curve and the
16 maximum of the i -th load cycle, whereas α_{ref} is the angle of the line through the origin of the
17 load-displacement curve and the maximum of the first load cycle. The third acceptance criterion,
18 the permanency ratio I_{pr} , is defined as:

$$19 \quad I_{pr} = \frac{I_{p(i+1)}}{I_{pi}} \leq 0.5 \quad (2)$$

20 where I_{pi} and $I_{p(i+1)}$ are the permanency indexes calculated for the i -th and $(i+1)$ -th load cycles,
21 based on the residual Δ_r and maximum Δ_{max} deflections in the considered load cycles:

1
$$I_{pi} = \frac{\Delta_r^i}{\Delta_{\max}^i} \quad (3)$$

2
$$I_{p(i+1)} = \frac{\Delta_r^{(i+1)}}{\Delta_{\max}^{(i+1)}} \quad (4)$$

3 The third acceptance criterion requires that the residual deflection, Δ_r , measured at least 24 hours
4 after removal of the load, is maximum 25% of the maximum deflection or 1/180 of the span
5 length.

6

7 **5.2 Instrumentation of viaduct Zijlweg**

8 To verify the existing stop criteria, and to monitor the bridge behavior carefully during proof
9 load testing, a number of sensors were applied to the viaduct Zijlweg prior to the proof load test.
10 The load effects that were measured during the test are: deformations of the deck, deformations
11 of the cross-beams, crack width growths, strains, movements in the joint and rotations of the end
12 support, and acoustic emission signals (Yang & Hordijk, 2015). The interpretation of acoustic
13 emission signals on existing bridges during proof load testing is a topic of ongoing research. For
14 these measurements, 16 LVDTs, 6 laser distance finders and 15 acoustic emission sensors were
15 used. The LVDTs and lasers used to measure the vertical deflections of the slab are shown in
16 Figure 8a. The LVDTs measuring strain over 1 m are shown in Figure 8b. The lasers used for
17 measuring the support deformations are shown in Figure 8c. The movements in the joint and
18 rotations of the end support are measured by two LVDTs on each side (east side and west side),
19 see Figure 9. An overview of the grid of acoustic emission sensors is given in Figure 10. The
20 positions of the cracks that were monitored during the test are indicated in Figure 4. The
21 longitudinal and transverse crack close to the support were monitored during the shear tests,
22 whereas the longitudinal and transverse crack further in the span were monitored during the

1 bending test. As such, for each test a longitudinal and transverse crack close to the position of the
2 load are monitored. One of the LVDTs is used as a reference to correct the measurements for
3 effect of temperature and humidity changes. This LVDT was applied on the abutment, and thus
4 not affected by the structure's response to load testing.

5 **5.3 *Measurements from viaduct Zijlweg***

6 First, the post-processing of the measurements will be discussed, and in a next section the
7 comparison to the stop criteria. The envelope of the load-displacement diagram from the proof
8 load test to demonstrate sufficient bending moment capacity is shown in Figure 11a. The load-
9 displacement diagram is complemented with tangent lines, indicating slightly less stiff behavior
10 during the third and fourth increase of the load. This observation could be the result of
11 redistribution of stresses, interaction between the loading frame and the bridge, possible friction
12 in the joint, or the fact that the loading speed was lower in these steps. During unloading after the
13 last loading step, the displacement was smaller than during the loading step, possibly caused by
14 transverse stress redistribution or friction in the joint. These observations led to the
15 recommendation to have a fixed loading speed for all load cycles. The profile of the
16 longitudinal deflection for different loads during the experiment for bending moment is shown in
17 Figure 11b. The position of the two axles is indicated by arrows. These results are as expected,
18 indicating increasing deflections for increasing loads. Finally, in Figure 11c, the increase in crack
19 width as a function of increased load is given. LVDT14 measured a crack on the side face that
20 could develop into a shear crack, which was not activated during the test. LVDT15 and LVDT16
21 measured a longitudinal and transverse crack, respectively, on the bottom of the slab. The
22 maximum increase in crack width was very small.

1 For the shear position, the envelope of the load-displacement diagram is shown in Figure
2 12a. Until the maximum load, the measured behavior was fully linear. The profiles of the
3 deflection in the longitudinal direction are shown in Figure 12b, in which LVDT8 had shifted out
4 of its measurement range between loads of 600 kN and 1100 kN. Finally, the cracks width
5 measurements show a slight activation of the crack that could develop into a shear crack, and in
6 general, very small increases in crack width, see Figure 12c.

7 **5.4 Analysis of stop criteria**

8 The maximum deformation in the bending moment test δ_{max} was 3.42 mm and the residual
9 deformation was $\delta_{residual} = 0.33$ mm, so that the ratio $\delta_{residual}/\delta_{max} = 9.7\%$, which is below the
10 maximum 10%. The maximum deformation in the shear test δ_{max} was 2.94 mm and the residual
11 deformation was $\delta_{residual} = 0.28$ mm immediately after unloading and $\delta_{residual} = 0.18$ mm when all
12 equipment was removed. The ratios are thus 9.7% and 6.1%, respectively, which both mean that
13 the maximum value of 10% was not exceeded. The 25% residual deflection form ACI 437.2M-
14 13 is obviously also achieved.

15 For the concrete strain, Eq. (0) is used. The strain caused by the permanent loads, ε_{c0} , is
16 taken from the finite element program. The results are given in Table 4. The difference in order
17 of magnitude of the strains between LVDT1 and LVDTs 2 and 3 is explained by the fact that
18 LVDT1 measures the strain in the transverse direction and LVDTs 2 and 3 in the longitudinal
19 direction. The results of the maximum measured strains are given in Table 4 as $\varepsilon_{c,max,meas}$, and the
20 values corrected with the reference LVDT for temperature as $\varepsilon_{c,max}$. The value of $\varepsilon_{c,lim}$ can be
21 calculated with the recommended expression (Walraven, 2012) :

$$22 \quad \varepsilon_{c,lim} = 0.9 \frac{f_{ck}}{E_c} = 0.9 \frac{24.5\text{MPa}}{30000\text{MPa}} = 735\mu\varepsilon \quad (4)$$

1 The value of $\varepsilon_{c,max}$ has to be smaller than $\varepsilon_{c,lim} - \varepsilon_{c0}$, a value which is given in the last column of
2 Table 4. Therefore, the stop criterion for concrete strain is never exceeded during the test, both
3 for the bending moment and shear tests.

4 For existing cracks, the German guideline prescribes a maximum increase in width Δw of
5 0.3 mm and a maximum residual crack width of $0.2 \times \Delta w$. The results are given in Table 5. It can
6 be seen that the increase in crack width is small, and that the stop criterion for crack width is
7 fulfilled for both the shear and bending moment positions. However, further research on control
8 specimens in the laboratory is required to define the maximum allowable limits for crack width
9 and increase in crack width, and to investigate if small crack widths can be neglected.

10 The remaining stop criteria from ACI 437.2M-13 are not analysed here, because these stop
11 criteria require that the loading scheme is exactly the same as the cyclic loading protocol
12 described in the code. For testing the viaduct Zijlweg, more load steps were used.

13

14 **6 Recommendations**

15 *6.1 Assessment of viaduct Zijlweg and suitability of proof load tests for ASR-affected* 16 *viaducts*

17 The proof load tests on the viaduct Zijlweg were successful to experimentally show that the
18 bridge can carry its prescribed factored live loads. The moment and shear load tests
19 corresponded to the safety level of RBK Design + 5% extra. The final conclusion for the viaduct
20 Zijlweg itself is the viaduct does not require posting. During the proof load experiment, there
21 was no sign of distress in the measurements. This information is important, because the uniaxial
22 tensile strength of the concrete in the viaduct is very low as a result of ASR-damage, so that the
23 shear capacity of this viaduct was subject to discussion. The proof load test on the viaduct in the

1 Zijlweg showed the important conclusion that the capacity of a viaduct affected by ASR is still
2 sufficient for the current traffic as prescribed by the live load model from NEN-EN 1991-2:2003
3 (CEN, 2003). The problem of the small strength of the concrete in uniaxial tension is thus of a
4 lower magnitude than feared, and similar tests can be recommended to experimentally approve
5 other ASR-affected viaducts, or viaducts where the uncertainties related to material degradation
6 are large.

7 **6.2 *Lessons learned for the development of guidelines for proof load testing***

8 For the determination of the target proof load, the presented method which is based on equivalent
9 sectional shears or moments caused by the proof load tandem and the considered live loads can
10 be recommended. The use of a single proof load tandem can be recommended for viaducts with a
11 small width. For wider viaducts, a combination of proof load tandems or loading vehicles needs
12 to be explored.

13 When assessing the stop criteria, the physical significance should never be forgotten. As can
14 be seen from Table 5, very small values for the maximum crack widths are found. A threshold
15 value below which the crack width can be considered as zero should be defined. A proposed
16 value is $w_{min} = 0.05$ mm. The problem with using a value such as a “residual deflection” and
17 “residual crack width” after each loading step is that the load has to be constant. This load could
18 be 0, but for practical purposes always a low load level has to be maintained so that all
19 measurements stay activated, for example 100 kN. Returning to this load level of exactly 100 kN
20 when using a system based on jacks is not easy in practice. Therefore, stop criteria that combine
21 both the effect of the applied load and the structural response are to be preferred. The stop
22 criterion related to the steel strains, Eq. (0), cannot be recommended for practical purposes, as it
23 requires removal of the concrete cover, which is something not all bridge owners would allow

1 for a non-destructive test such as a proof load test. The analysis of the measurements shows that
2 following the load-displacement diagram and the plots of the deformations in the longitudinal
3 and transverse directions is important to identify possible non-linear behaviour. Defining clear
4 stop criteria is a topic of current research.

5 Finally, the method present in this article can be applied to reinforced concrete slab
6 bridges. Slab bridges typically show signs of distress, even before a brittle shear failure. For
7 more brittle structural types, such as prestressed girder bridges, a further experimental validation
8 of stop criteria will be necessary.

9 **7 Summary and Conclusions**

10 Load testing is part of the standard engineering practice. However, one form of load testing,
11 proof load testing, is less common. This type of testing allows direct experimental approval of a
12 bridge if it can carry its factored live loads without significant signs of distress. If signs of
13 distress occur prior to reaching the target proof load required to show adequate performance of
14 the bridge, the bridge can be approved for the last load level that could be sustained without
15 significant signs of distress. If this last load level is lower than the minimum safety requirements,
16 posting or strengthening of the bridge may be necessary. Note that these minimum safety
17 requirements should be calculated with a live load factor calibrated with the same target value of
18 the reliability index as appears in Table 1, but considering the nominal values of self-weight, as
19 this value has now become a deterministic value. This decision however needs to be taken by the
20 bridge owner. Proof load tests are typically not allowed for shear-critical situations. Because a
21 large number of the existing reinforced concrete slab bridges in the Netherlands rate
22 insufficiently for shear, proof load testing for shear needs to be developed. A pilot proof load test
23 was carried out to evaluate the applicability of existing procedures, and to highlight the areas that

1 need further research. In particular, guidelines for reinforced concrete slab bridges need to be
2 developed.

3 One important area is to determine the required target proof load. Whereas in practice in
4 the USA this load is (typically) 1.4 times the load of the design truck the bridge needs to be rated
5 for, in Europe, rating and assessment is not carried out based on a set of standard vehicles, but by
6 using the regular live load models, or by following national recommendations. These models
7 consist of combined distributed lane loads and concentrated live loads. Therefore, it is proposed
8 here to find the target proof load as the load that causes the same sectional moment or shear as
9 the factored live loads from the code. For bending moment, the critical position is found by
10 moving the design tandems in their respective lane, and finding the largest bending moment. The
11 critical position for shear is taken at $2.5d$, as was determined from tests on half-scale slab bridges
12 in the laboratory, and at the obtuse corner. The viaduct Zijlweg was successfully tested at these
13 two positions, showing that proof load testing for reinforced concrete slab bridges is feasible and
14 can be used to demonstrate that the considered bridge can carry the prescribed live loads without
15 significant distress. It is important to point out that this ASR-affected viaduct was difficult to
16 assess analytically, as the effect of ASR on the shear capacity is currently not well-understood.

17 A second important topic is the measurements and stop criteria. Current codes and
18 guidelines only prescribe stop criteria for bending moment. From the analysis of this pilot test, it
19 was found that the deviation from linearity index and the permanency ratio are strongly
20 dependent on the prescribed loading protocol, and cannot be applied directly to the loading
21 protocol followed in this pilot test. The analysis also showed that measuring steel strains is not
22 practical, as it requires removal of the concrete cover, which many bridge owners do not allow.
23 The stop criterion for the concrete strain was found to be interesting, as well as the criterion for

1 the crack width and increase in crack width, provided that a lower bound of crack width is added
2 to these considerations. Additionally, when loads are applied through hydraulic jacks and cannot
3 be controlled very accurately, it can be recommended to develop stop criteria that contain both
4 the load and the structural response.

5 This pilot proof load test on a reinforced concrete slab bridge with material damage
6 shows that proof load testing can be used for the assessment of such bridges. It is shown that
7 proof load testing for shear is possible. For practical application, the stop criteria need to be
8 defined for both shear and flexure. Some useful stop criteria have been identified based on this
9 pilot test, and research on beams cast in the laboratory will further define and identify suitable
10 stop criteria.

11

12 **Notation List**

13 The following symbols are used in this paper:

14	d	the effective depth
15	$f_{c,cube,m}$	the average cube compressive strength
16	f_{ck}	the characteristic concrete cylinder compressive strength
17	f_{yk}	the characteristic yield strength
18	f_{ym}	the average yield strength
19	r	the radius of curvature
20	w	the crack width
21	w_{min}	the crack width below which the crack width can be neglected
22	E_s	the Young's modulus of the steel
23	F	the load

1	F_{tot}	the maximum required load on the proof load tandem to approve a bridge for a certain safety level
2		
3	I_{DL}	the deviation from linearity index
4	I_{Pi}	the permanency index of the i -th load step
5	I_{PR}	the permanency ratio
6	α_i	the angle formed by the line between the origin and the loading point
7	α_{ref}	the angle of the load-displacement diagram of the first cycle
8	β	the reliability index
9	δ	the measured deformation
10	δ_{max}	the maximum deformation
11	$\delta_{residual}$	the residual deformation
12	ε_c	the measured strain in the concrete
13	$\varepsilon_{c,lim}$	the limiting concrete strain
14	$\varepsilon_{c,max,meas}$	the maximum measured concrete strain
15	$\varepsilon_{c,max}$	the governing maximum concrete strain
16	ε_{c0}	the strain caused by the permanent loads
17	ε_{s2}	the strain in the steel
18	ε_{s02}	the strain in the steel caused by the permanent loads
19	ρ_c	the density of concrete
20	Δw	the increase in crack width of an existing crack
21	Δ_r^i	the residual deflection (non-cumulative) after the i -th load cycle
22	Δ_{max}^i	the maximum deflection after the i -th load cycle
23		

1 **Acknowledgement**

2 The authors wish to express their gratitude and sincere appreciation to the Dutch Ministry of
3 Infrastructure and the Environment (Rijkswaterstaat) and the Province of Noord Brabant for
4 financing this research work. The contributions and help of our colleagues Albert Bosman, Cor
5 van der Veen, and Yuguang Yang, and students Arthur Ennouri and Werner Vos of Delft
6 University of Technology, of Witteveen+Bos, responsible for the logistics and safety, and of
7 Mammoet, responsible for applying the load, are gratefully acknowledged. The fruitful
8 discussions with Frank Linthorst and Danny den Boef of Witteveen+Bos and with Otto Illing and
9 the late Chris Huissen of Mammoet are acknowledged.

10

11 **References**

- 12 AASHTO. (2011). *The manual for bridge evaluation* (2nd ed.). Washington, D.C.: American
13 Association of State Highway and Transportation Officials.
- 14 ACI Committee 437. (2013). Code Requirements for Load Testing of Existing Concrete
15 Structures (ACI 437.2M-13) and Commentary (pp. 24). Farmington Hills, MA.
- 16 Ahmed, T., Burley, E., & Rigden, S. (1998). The static and fatigue strength of reinforced
17 concrete beams affected by alkali-silica reaction. *ACI Materials Journal*, 95(4), 376-388.
- 18 Ahmed, T., Burley, E., & Rigden, S. (1999). Effect of alkali-silica reaction on tensile bond
19 strength of reinforcement in concrete tested under static and fatigue loading. *ACI*
20 *Materials Journal*, 96(4), 419-428.
- 21 Borsje, H., Peelen, W. H. A., Postema, F. J., & Bakker, J. D. (2002). Monitoring Alkali-Silica
22 Reaction in Structures. *Heron*, 47(2), 96-109.
- 23 Casas, J. R., & Gómez, J. D. (2013). Load Rating of Highway Bridges by Proof-loading. *KSCE*
24 *Journal of Civil Engineering*, 17(3), 556-567.
- 25 CEN. (2003). Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges, NEN-EN
26 1991-2:2003 (pp. 168). Brussels, Belgium: Comité Européen de Normalisation.
- 27 CEN. (2005). Eurocode 2: Design of Concrete Structures - Part 1-1 General Rules and Rules for
28 Buildings. NEN-EN 1992-1-1:2005 (pp. 229). Brussels, Belgium: Comité Européen de
29 Normalisation.
- 30 Code Committee 351001. (2011a). *Assesment of structural safety of an existing structure at*
31 *repair or unfit for use - Basic Requirements, NEN 8700:2011 (in Dutch)*. Delft, The
32 Netherlands: Civil center for the execution of research and standard, Dutch Normalisation
33 Institute.

- 1 Code Committee 351001. (2011b). Eurocode 1 - Actions on structures - Part 2: Traffic loads on
2 bridges, EN 1991-2/NA:2011 (pp. 38). Delft, The Netherlands: Civil engineering center
3 for research and regulation, Dutch Normalization Institute.
- 4 Cope, R. J. (1985). Flexural Shear Failure of Reinforced-Concrete Slab Bridges. *Proceedings of*
5 *the Institution of Civil Engineers Part 2-Research and Theory*, 79(SEP), 559-583.
- 6 Davids, W. G., Poulin, T. J., & Goslin, K. (2013). Finite-Element Analysis and Load Rating of
7 Flat Slab Concrete Bridges. *Journal of Bridge Engineering*, 18(10), 946-956.
- 8 den Uijl, J. A., & Kaptijn, N. (2002). Structural consequences of ASR: an example on shear
9 capacity. *Heron*, 47(2), 1-13.
- 10 Deutscher Ausschuss für Stahlbeton. (2000). DAfStb-Guideline: Load tests on concrete
11 structures (pp. 7 (in German)): Deutscher Ausschuss für Stahlbeton,.
- 12 fib. (2012). *Model code 2010: final draft*. Lausanne: International Federation for Structural
13 Concrete.
- 14 Giaccio, G., Zerbino, R., Ponce, J. M., & Batic, O. R. (2008). Mechanical behavior of concretes
15 damaged by alkali-silica reaction. *Cement and Concrete Research*, 38(7), 993-1004. doi:
16 10.1016/j.cemconres.2008.02.009
- 17 Haddad, R. H., Shannag, M. J., & Al-Hambouth, M. T. (2008). Repair of reinforced concrete
18 beams damaged by alkali-silica reaction. *Aci Structural Journal*, 105(2), 145-153.
- 19 Koekkoek, R., Lantsoght, E., Yang, Y., Boer, A. d., & Hordijk, D. (2016). *Defining loading*
20 *criteria for proof loading of existing reinforced concrete bridges*. Paper presented at the
21 fib symposium 2016: Performance-based approaches for concrete structures, Cape Town,
22 South Africa.
- 23 Koenders Instruments. (2015). Zijlweg Monitoring systeem (pp. 4).
- 24 Lantsoght, E. O. L., De Boer, A., & Van der Veen, C. (2017) Levels of Approximation for the
25 shear assessment of reinforced concrete slab bridges. *Structural Concrete* 18:143-152.
- 26 Lantsoght, E. O. L., Van der Veen, C., De Boer, A., & Hordijk, D. A. (2016). Collapse test and
27 moment capacity of the Ruytenschildt Reinforced Concrete Slab Bridge *Structure and*
28 *Infrastructure Engineering*. DOI: 10.1080/15732479.2016.1244212
- 29 Lantsoght, E. O. L., de Boer, A., Van der Veen, C., & Walraven, J. C. (2013). *Peak shear stress*
30 *distribution in finite element models of concrete slabs*. Paper presented at the Research
31 and Applications in Structural Engineering, Mechanics and Computation, Cape Town,
32 South Africa.
- 33 Lantsoght, E. O. L., van der Veen, C., de Boer, A., & Alexander, S. D. B. (2017). Extended Strip
34 Model for Slabs under Concentrated Loads. *Aci Structural Journal*. 114(2), 565-574.
- 35 Lantsoght, E. O. L., van der Veen, C., De Boer, A., & Walraven, J. (2014). Influence of Width
36 on Shear Capacity of Reinforced Concrete Members. *Aci Structural Journal*, 111(6),
37 1441-1450.
- 38 Lantsoght, E. O. L., van der Veen, C., de Boer, A., & Walraven, J. (2015). One-way slabs
39 subjected to combination of loads failing in shear. *Aci Structural Journal*, 112(4), 417-
40 426.
- 41 Lantsoght, E. O. L., van der Veen, C., de Boer, A., & Walraven, J. C. (2013). Recommendations
42 for the Shear Assessment of Reinforced Concrete Slab Bridges from Experiments
43 *Structural Engineering International*, 23(4), 418-426.
- 44 Lantsoght, E. O. L., van der Veen, C., Walraven, J., & de Boer, A. (2015). Experimental
45 investigation on shear capacity of reinforced concrete slabs with plain bars and slabs on
46 elastomeric bearings. *Engineering Structures*, 103, 1-14.

- 1 Lantsoght, E. O. L., van der Veen, C., & Walraven, J. C. (2013). Shear in One-way Slabs under a
2 Concentrated Load close to the support. *Aci Structural Journal*, *110*(2), 275-284.
- 3 NCHRP. (1998). Manual for Bridge Rating through Load Testing (Vol. NCHRP Project 12-
4 28(13)A, pp. 152). Washington, DC.
- 5 Neville, A. (2012). *Properties of Concrete, 5th edition*: Pearson Education
- 6 Nijland, T. G., & Siemes, A. J. M. (2002). Alkali-Silica Reaction in The Netherlands:
7 Experiences and current research. *Heron*, *47*(2), 81-84.
- 8 Olaszek, P., Łagoda, M., & Casas, J. R. (2014). Diagnostic load testing and assessment of
9 existing bridges: examples of application. *Structure and Infrastructure Engineering*,
10 *10*(6), 834-842.
- 11 Provincie Noord Brabant. (1965). Viaduct Zijlweg (pp. 12). 1965. (in Dutch)
- 12 Rijkswaterstaat. (1997). Safety evaluation of existing structures - Reinforced concrete bridges (in
13 Dutch) (pp. 102). Utrecht: Rijkswaterstaat.
- 14 Rijkswaterstaat. (2002). Management- and Maintenance plan Viaduct Zijlweg of Highway A59
15 (pp. 36). (in Dutch)
- 16 Rijkswaterstaat. (2008). Inspection report of object 44G-113-01 (in Dutch) (pp. 26).
- 17 Rijkswaterstaat. (2012). Guidelines for Nonlinear Finite Element Analysis of Concrete Structures
18 (pp. 65).
- 19 Rijkswaterstaat. (2013). Guidelines Assessment Bridges - assessment of structural safety of an
20 existing bridge at reconstruction, usage and disapproval (in Dutch) (pp. 117).
- 21 Saraf, V. K., Nowak, A. S., & Till, R. (1996). *Proof load testing of bridges*. Paper presented at
22 the Probabilistic Mechanics & Structural Reliability: Proceedings of the Seventh
23 Specialty Conference. <Go to ISI>://WOS:A1996BG42M00127
- 24 Schacht, G., Bolle, G., Curbach, M., & Marx, S. (2016). Experimental Evaluation of the shear
25 bearing safety. *Beton- und Stahlbetonbau*, *111*(6), 343-354. (in German)
- 26 Schmidt, J. W., Hansen, S. G., Barbosa, R. A., & Henriksen, A. (2014). Novel shear capacity
27 testing of ASR damaged full scale concrete bridge. *Engineering Structures*, *79*, 365-374.
28 doi: 10.1016/j.engstruct.2014.08.027
- 29 Schwesinger, P., & Bolle, G. (2000). *EXTRA - a new experiment supported condition assessment*
30 *method for concrete bridges*. Paper presented at the Proc. SPIE 3995, Nondestructive
31 Evaluation of Highways, Utilities, and Pipelines IV.
- 32 Siemes, T., Han, N., & Visser, J. (2002). Unexpectedly low tensile strength in concrete
33 structures. *Heron*, *47*(2), 111-124.
- 34 Steenbergen, R. D. J. M., de Boer, A., & van der Veen, C. (2011). *Safety assessment of existing*
35 *concrete slab bridges for shear capacity*. Paper presented at the Applications of Statistics
36 and Probability in Civil Engineering.
- 37 Talley, K. G. (2009). *Assessment and Strengthening of ASR and DEF Affected Concrete Bridge*
38 *Columns*. (Ph.D.), UT Austin, Austin.
- 39 TNO DIANA. (2012). Users Manual of DIANA, Release 9.4.4. Delft, The Netherlands.
- 40 Varela-Ortiz, W., Cintrón, C. Y. L., Velázquez, G. I., & Stanton, T. R. (2010). Load testing and
41 GPR assessment for concrete bridges on military installations. *Construction and Building*
42 *Materials*, *38*, 1255-1269.
- 43 Vergoossen, R., Naaktgeboren, M., 't Hart, M., De Boer, A., & Van Vugt, E. (2013). *Quick Scan*
44 *on Shear in Existing Slab Type Viaducts*. Paper presented at the International IABSE
45 Conference, Assessment, Upgrading and Refurbishment of Infrastructures, Rotterdam,
46 The Netherlands.

- 1 Walraven, J. C. (2012). Proof loading of concrete bridges (in Dutch) (Stevin Report 25-5-2012).
2 Delft: Delft University of Technology.
- 3 Witteveen+Bos. (2014). Material research bridges case 31084913: 44G-113-01 - viaduct Zijlweg
4 (pp. 23). Deventer.
- 5 Yang, Y., & Hordijk, D. A. (2015). Acoustic Emission Measurement and Analysis on
6 Zijlwegbrug (Stevin Report 25.5-15-01, pp. 27). Delft, The Netherlands: Delft University
7 of Technology.

1 **List of Figures**

2 **Figure 1.** Photograph of viaduct Zijlweg.

3 **Figure 2.** Structural system of viaduct Zijlweg: (a) longitudinal section; (b) cross-section. Units:
4 cm.

5 **Figure 3.** Reinforcement of viaduct Zijlweg: (a) top view; (b) side view; (c) transverse beam
6 above abutments. Bar diameters in mm, all other dimensions in cm.

7 **Figure 4.** Map of existing cracks in span 4 of viaduct Zijlweg. The cracks that were monitored
8 during a test are indicated in bold.

9 **Figure 5.** Setting up the loads for the proof load test: (a)
10 application of jacks; (b) application of steel girders; (c) positioning of counterweights.

11 **Figure 6.** Positions of jacks during proof load tests: (a) bending moment position; (b) shear
12 position.

13 **Figure 7.** Loading schemes shown as the time history of the applied load: (a) bending moment
14 position, (b) shear position.

15 **Figure 8.** Instrumentation on bottom of slab of viaduct Zijlweg: (a) LVDTs and lasers measuring
16 vertical deflections; (b) LVDTs measuring strains; (c) lasers measuring support deflections, also
17 showing the position of the jacks during the proof load tests.

18 **Figure 9.** Measurement of joint closing and rotation, showing LVDTs on west side of slab.
19 LVDTs 11 and 12 are applied to the east side in a similar way as on the west side.

20 **Figure 10.** Position of acoustic emission sensors.

21 **Figure 11.** Observations during the bending moment proof load test: (a) load-displacement
22 diagram; (b) deflection in the longitudinal direction; (c) increase in crack width of existing
cracks versus force.

1 **Figure 12.** Observations during the shear proof load test: (a) load-displacement diagram; (b)
2 deflection in the longitudinal direction; (c) increase in crack width of existing cracks versus
3 force.
4

1 **List of Tables**

2 **Table 1.** Different safety levels for assessment of existing structures and associated load factors,
3 when used in combination with proof load testing.

Reliability level	β	Reference period	γ_{sw}	$\gamma_{superimposed}$	γ_{LL}
ULS Eurocode	4.3	100 years	1.10	1.35	1.50
RBK Design	4.3	100 years	1.10	1.25	1.50
RBK Reconstruction	3.6	30 years	1.10	1.15	1.30
RBK Usage	3.3	30 years	1.10	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

4

5

1 **Table 2.** History of documentation of the viaduct Zijlweg.

Description	Year	Ref
Original calculations	1965	(Provincie Noord Brabant, 1965)
Detection of ASR	1997	(Rijkswaterstaat, 1997)
Overview of repair activities and plan for management and maintenance	2002	(Rijkswaterstaat, 2002)
Description of monitoring system	2003 2007	(Koenders Instruments, 2015)
Inspection	2008	(Rijkswaterstaat, 2008)
Material properties	2014	(Witteveen+Bos, 2014)

2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25

1 **Table 3.** Required maximum loads on the proof loading tandem at different safety levels.

Safety level	F_{tot} (kN)	F_{tot} (metric ton)	F_{tot} (kN)	F_{tot} (metric ton)
	Bending moment test		Shear test	
ULS Eurocode	1259	128	1228	125
RBK Design	1257	128	1228	125
RBK Reconstruction	1091	111	1066	109
RBK Usage	1050	107	1027	105
RBK Disapproval	1049	107	1025	104
SLS Eurocode	815	83	791	81

2

3

4

5

6

7

8

9

10

11

12

13

14

15

1 **Table 4.** Results of stop criterion of concrete strain for bending moment position

Measurement	ε_{c0} ($\mu\varepsilon$)	$\varepsilon_{c,max,meas}$ ($\mu\varepsilon$)	$\varepsilon_{c,max}$ ($\mu\varepsilon$)	$\varepsilon_{c,lim} - \varepsilon_{c0}$ ($\mu\varepsilon$)
Bending moment proof load test				
LVDT1	2.8	27.5	41.3	732.2
LVDT2	38.1	216.0	240.0	696.9
LVDT3	33.0	144.9	168.0	702.0
Shear proof load test				
LVDT1	2.8	86.7	38.0	732.2
LVDT2	45.1	274.2	224.0	690.0
LVDT3	38.3	201.6	153.0	696.7

2

3

4

5

6

7

8

9

10

11

12

13

14

1 **Table 5.** Results of stop criterion of increase in crack width for bending moment position

	Measured Δw (mm)		$0.2 \times \Delta w$ (mm)
	during proof loading	after proof loading	
Bending moment proof load test			
LVDT14	0.00	0.00	(no action)
LVDT15	0.01	0.00	0.00
LVDT16	0.04	0.00	0.01
Shear proof load test			
LVDT14	0.02	0.01	0.00
LVDT15	0.02	0.01	0.00
LVDT16	0.02	0.01	0.00

2