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Case study: Pilot proof load test on viaduct De Beek

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Abstract

For existing bridges, proof load testing can be a suitable assessment method. This paper addresses the evaluation of a posted reinforced concrete slab bridge over the highway through proof load testing, detailing the preparation, execution and analysis of the test. As the target proof load and the required measurements for proof load testing currently are not well-defined in the existing codes, this pilot case is used to develop and evaluate proposed recommendations for proof load testing for a future guideline on proof load testing for the Netherlands. Moreover, the pilot proof load test is used to study the feasibility of proof load testing for both shear and flexure.

CE database subject headings

assessment; bridge maintenance; bridge tests; concrete slabs; field tests; flexural strength; shear strength

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Introduction

Load testing is the non-destructive field testing of bridges (Cochet et al. 2004; Frýba and Pirner 2001; NRA 2014). Two types of load testing can be distinguished. Diagnostic load testing (Ataei et al. 2016; Bentz and Hoult 2016; Farhey 2005; Fu et al. 1997; Gokce et al. 2011; Halding et al. 2017; Jauregui et al. 2010; Kim et al. 2009; Maguire et al. 2015; Matta et al. 2008; Moen et al. 2013; Murià-Vila et al. 2015; Nguyen et al. 2016; Ohanian et al. 2017; Olaszek et al. 2014; Russo et al. 2000; Sanayei et al. 2012; Sanayei et al. 2016; Stroh et al. 2010; Velázquez et al. 2000) uses lower load levels, and is used to verify assumptions made in analytical models. In practice, these models are often linear elastic, three-dimensional finite element models (Bell and Sipple 2009; Bridge Diagnostics Inc. 2012; Hernandez and Myers 2015). The structural response in the analytical model can be compared to the structural response measured in the field, and the analytical model and the resulting rating can be updated accordingly. Proof load testing uses higher load levels. In a proof load test (Aguilar et al. 2015; Anay et al. 2016; Arangjelovski et al. 2015; Cai and Shahawy 2003; Casas and Gómez 2013; Faber et al. 2000; Fu and Tang 1995; Moses et al. 1994; Olaszek et al. 2012; Olaszek et al. 2016; Saraf et al. 1996; Spaethe 1994; Zwicky and Brühwiler 2015), a load is applied that demonstrates that the bridge can carry the loads prescribed by the code satisfactorily, or that higher or lower load levels can be carried by the bridge. Whether the bridge behavior is satisfactory is typically expressed based on “acceptance criteria” or “stop criteria”. These criteria, based on, among others, deflections, crack widths and strains, identify the acceptable limits of the bridge’s structural response. If these limits are exceeded during a proof load test, and higher loads are applied, there is a risk for irreversible damage to the structure. If a stop criterion is exceeded, further loading is not permitted. The conclusion of the proof load test is then that the bridge satisfies a lower load level (i.e. the last load level that was achieved prior to exceedance of a stop criterion) than the target load level. Alternatively, when the target load level is achieved, but no stop criterion has been exceeded yet, further loading can be used to demonstrate a larger load level.

Diagnostic load testing can be used to determine the transverse flexural distribution (He et al. 2012), to determine the stiffness of a structure (Barker 2001; Zhang et al. 2011), and to verify if a design or repair intervention is functioning appropriately (Nilimaa et al. 2015; Puurula et al. 2015; Shifferaw and Fanous 2013). For structures with limited uncertainties, such as steel bridges or concrete girder bridges, diagnostic load testing is recommended. Strain gages can be placed over the girder height to determine the position of the neutral axis. The differences in structural response in the analytical model and the response measured in the field can be attributed to different contributions, such as the actual impact factor, the actual dimensions, the unaccounted stiffness of elements such as curbs and railing, the actual lateral live load distribution, the bearing restraint effect, and unintended composite action (Barker 2001). For bridges with large uncertainties, on the other hand, proof load testing is necessary. These large uncertainties can include the effect of material degradation on the structure’s response (Koekkoek et al. 2015a), the geometry and reinforcement layout for bridges without plans (Aguilar et al. 2015; Anay et al. 2016; Shenton et al. 2007), or the load path at higher load levels (Taylor et al. 2007). For bridge types such as reinforced concrete slab bridges (Saraf 1998), placing strain gages over the height is more complicated, and measurements can only be taken from the bottom of the slab, from the side faces, and, provided that it does not obstruct the loading process and that no wearing surface covering the concrete cross-section is present, from the top faces. This paper deals with a case study of proof load testing of a reinforced concrete slab bridge for both flexure and shear, and how the results of this case study can be used to develop and evaluate recommendations for proof load testing.

Proof load testing

Current standards and guideline

Existing codes for load testing of bridges focus on diagnostic load testing. Examples are the French guidelines (Cochet et al. 2004), the Irish guidelines (NRA 2014) and the British guidelines (The Institution of Civil Engineers - National Steering Committee for the Load Testing of Bridges 1998). Similar procedures are followed in Italy (Veneziano et al. 1984), Switzerland (Brühwiler et al. 2012), and the Czech Republic and Slovakia (Frýba and Pirner 2001). The Manual for Bridge Rating through Load Testing (NCHRP 1998) and the Manual for Bridge Evaluation (AASHTO 2016) deal with diagnostic load testing and proof load testing. These manuals do not qualitatively describe stop criteria for proof load testing, but mention that the test should be terminated when the bridge exhibits the onset of non-linear behavior or other visible signs of distress. None of the existing codes for proof load testing allow for the testing of non-ductile failure modes, such as shear in concrete bridges.

For proof load testing of concrete structures, building codes are available. The German guidelines (Deutscher Ausschuss für Stahlbeton 2000) are originally developed for reinforced and plain concrete buildings, but are also applied to concrete bridges (Schacht et al. 2016b). For buildings, ACI 437.2M-13 (ACI Committee 437 2013), prescribing a slightly different required proof load than ACI 318-14 (ACI Committee 318 2014), is available. Since these codes are specialized for concrete structures (and buildings in particular), they contain detailed stop criteria (nomenclature used in the German guidelines) or acceptance criteria (nomenclature used in ACI 437.2M-13 (ACI Committee 437 2013)). The stop criteria are only valid for flexure-critical positions, and proof load testing for shear is not permitted. Testing for shear is a current topic of research (Schacht et al. 2016a).

Goals of proof load testing and examples

The main goal of a proof load test is to demonstrate experimentally that a bridge can withstand the factored live loads given in the code. As such, a proof load test does not give an estimate of the ultimate capacity of a bridge; only a lower bound of the capacity: the capacity is known to be larger than the load effect induced by the proof load. However, because of the high load levels involved in proof load tests, the risks for structural damage is larger. Adequate preparation to guarantee the structural safety of the bridge and the safety of the personnel is thus important (Cai and Shahawy 2003).

Some states and countries have developed special vehicles for proof loading. Examples of these vehicles include the two proof loading vehicles of Florida that can be loaded with ballast blocks (90 tons maximum each) (Shahawy 1995), and the BELFA (“Belastungsfahrzeug”, German for loading vehicle) from Germany (ifem 2013), which can apply a maximum load of 150 tons.

In the state of New Mexico, a large number of bridges without plans exist (Aguilar et al. 2015), for which a rating method based on diagnostic and proof load tests, combined with other non-destructive testing techniques has been developed. Similar testing has also been carried out in New York state (Hag-Elsafi and Kunin 2006), in Delaware (Shenton et al. 2007), and on bridges owned by the US Army (Varela-Ortiz et al. 2010), which are subjected to different live loads (military vehicles).

Another type of uncertainty that can require proof load tests, is uncertainty related to the effect of material deterioration and degradation on the structural performance of existing bridges.

An example is the proof load testing of a deteriorated bridge in Michigan (Juntunen and Isola 1995), where a proof load test with an 82-ton two-unit vehicle successfully showed that the load restriction of 45 tons did not need to be reduced because of the extensive deterioration in the bridge. A later analysis, however, showed that in the proof load test, composite action between the old beams and the newly applied overlay had occurred. This composite action is lost over time, but was still sufficient for the structure to keep the 45 ton two-unit vehicle limit.

Previous proof load tests in the Netherlands

In the Netherlands, a large number of reinforced concrete slab bridges were built in the decades following the Second World War (Lantsoght et al. 2013b). These bridges are reaching the end of their originally devised service life. To assess these structures, and to investigate their structural safety under the current live loads that are larger than those at the time of their design, an assessment is necessary. In Europe, no separate live loads models are defined for the assessment of existing bridges. Therefore, all assessment, including assessment through proof load testing, needs to be carried out based on the live load model which consists of design tandems and distributed lane loads. In North American practice, the target proof load can be calculated as a multiple (reference value = 1.4) of the truck used for assessment. In Europe, the target proof load needs to represent the full live load model.

During the last decade, a number of proof load tests on reinforced concrete slab bridges have been carried out in the Netherlands. An overview of the program of pilot proof load tests can be found elsewhere (Lantsoght et al. in press). In this paragraph, only the main reasons for selecting the pilot cases, and main conclusions from the load tests are given. The first test was carried out on the viaduct Heidijk (Dieteren and den Uijl 2009), to see if this bridge with material degradation caused by alkali-silica reaction can carry a truck of 30 ton on a shear-critical position. The load was applied through a loading frame and hydraulic jacks with a hand pump. It was found that the 30 ton truck can be successfully carried. A second test was on the viaduct Vlijmen-Oost (Koekkoek et al. 2015b), also affected by alkali-silica reaction. The BELFA vehicle (Bretschneider et al. 2012) was used on a shear-critical position and on a critical position for bending moment. It was concluded that the bridge fulfills the current code requirements. In a next test, an existing slab bridge with insufficient flexural capacity according to the assessment calculations was tested: the Halvemaans Bridge (Fennis and Hordijk 2014). This test was the first test in which the load was applied by using a load spreader beam and hydraulic jacks. Again, the load test was used to show that the bridge fulfills the requirements. In the summer of 2014, the Ruytenschildt bridge was tested to failure (Lantsoght et al. 2016a; Lantsoght et al. 2016b; Lantsoght et al. 2016c; Lantsoght et al. available online ahead of print) in two spans. The last proof load test on a bridge with damage caused by alkali-silica reaction, the viaduct Zijlweg, studied a shear- and flexure-critical position in the first span (Koekkoek et al. 2015a; Lantsoght et al. in review). Upon assessment, it was found that the viaduct Zijlweg does not fulfill the requirements of the code for shear. Through the proof load test, it could be shown that the viaduct can carry the factored live loads of the code without signs of distress, and that it fulfills the requirements for shear and bending moment. It should be emphasized that proof load testing for shear is uncommon and typically not permitted, and that none of the existing codes or guidelines prescribes stop criteria for shear.

Description of viaduct De Beek

Restrictions on viaduct De Beek

Viaduct De Beek, a reinforced concrete slab bridge, see Fig. 1a, lies in a local road, the Beekstraat, over highway A67 close to Ommel in the province of Noord Brabant in the Netherlands. The bridge was built in 1963 and is owned and managed by the Dutch Ministry of Infrastructure and the Environment. An inspection and assessment for the current live loads in 2015 (Willems et al. 2015) led to the conclusion that the capacity of the viaduct is insufficient for two lanes of unrestricted traffic. The assessment calculations (Iv-Infra 2015) determined that the flexural capacity in the longitudinal and transverse direction is insufficient in all spans. Originally, load posting was proposed, but for practical reasons it was decided to restrict traffic to one lane by using barriers, see Fig. 1b. During the inspection of 2015, structural damage (wide cracking) was observed at the bottom of the concrete deck, compromising the durability of the structure.

Geometry of viaduct De Beek

The geometry of viaduct De Beek can be seen in Fig. 2. The viaduct has four spans, with end spans of 10.81 m and central spans of 15.40 m. The width of the viaduct is 9.94 m, with a carriageway width of 7.44 m, originally designed to carry one lane of traffic of 3.5 m wide in each direction. The viaduct has a height that varies parabolically between 470 mm and 870 mm. In the width direction, a curb with a height of 200 mm is available at the edge. The layer of asphalt is measured to be between 50 mm and 75 mm.

Material properties of viaduct De Beek

Nine cores were drilled from the slab to determine the concrete properties. The characteristic concrete compressive strength f_{ck} equals 44.5 MPa and the concrete tensile splitting strength f_{ctm} = 4.4 MPa. The design concrete compressive strength is thus f_{cd} = 30 MPa.

Three samples of the steel were taken, from which it was concluded that steel QR 24 was used. QR 22 and QR 24 are types of plain reinforcement that were used in the Netherlands during the 1950s and 1960s. The measured average yield strength f_{ym} = 291 MPa and the tensile strength f_{tm} = 420 MPa. The design yield strength can be taking as f_{yd} = 252 MPa. The reinforcement drawing is given in Fig. 3. The main flexural reinforcement in the longitudinal direction in span 1 consists of 6 layers of ϕ 25 mm with a 560 mm spacing, so that the reinforcement is A_s = 5259 mm^2/m.

Determination of target proof load

Practical application of the target proof load

As mentioned previously, the live load model that is used for assessment of existing bridges in Europe does not allow for a direct translation to a certain type of truck, unlike in North America. Whereas in North America heavy dump trucks, special vehicles, and/or military vehicles can be used for proof load tests, in Europe only the BELFA vehicle from Germany (Bretschneider et al. 2012) is available with a maximum load of 150 metric ton. Regular vehicles are not suitable. Other options for applying the target proof load in Europe include directly applying dead weights on the deck (Olaszek et al. 2014), or by using an external structure (Schwesinger and Bolle 2000).

Target proof load in North America

According to the Manual for Bridge Rating through Load Testing (NCHRP 1998) and the Manual for Bridge Evaluation (AASHTO 2016), the target proof load is based on the load L_R of the vehicle used for load rating at the legal load level, multiplied with a factor X_p and taking into account the impact allowance I. The standard value of X_p equals 1.4. This value is adjusted as follows:

- X_p needs to be increased by 15% if one lane load controls the response.
- For spans with fracture-critical details, X_p shall be increased by 10%.
- If routine inspections are performed less than every 2 years, X_p should be increased by 10%.
- If the structure is ratable, i.e. has no hidden details, X_p can be reduced by 5%.
- Additional factors including traffic intensity and bridge condition may also be incorporated in the selection of the live load factor X_p.

Taking into account the effect of these adjustments, the target live load factor X_{pA} is found as follows:

$$X_{pA} = X_p\left(1+\frac{\Sigma\%}{100}\right) \tag{1}$$

The value of the target proof load is then determined as:

$$L_T = X_{pA} L_R\left(1+I\right) \tag{2}$$

with $1.3 \leq X_{pA} \leq 2.2$.

Application to Eurocode live loads and Dutch safety levels

It has been suggested for Europe to use WIM data to determine the target proof load (Casas and Gómez 2013), but these data are not available for most bridges. In the Netherlands, different safety levels, associated with different reliability indices are defined for existing structures in the national code NEN 8700:2011 (Code Committee 351001 2011) and the Guidelines Assessment Bridges (Richtlijn Beoordeling Kunstwerken = “RBK”) (Rijkswaterstaat 2013). An overview of these different levels is given in Table 1, together with the ultimate limit state and the serviceability limit state from the Eurocode for design of new structures (CEN 2002). These different safety levels correspond to different load factors. The load factors that are used to determine the proof load are given in Table 2. Note that here the load factor of the self-weight, γ_{sw} = 1.10 for all safety levels (except the serviceability limit state). The reason why a lower load factor for the self-weight is used is that, because the calculations involve an existing structure, the dimensions of the structure are not a random variable anymore, but can be considered deterministic (i.e., the actual dimensions of the structure). Only the model factor remains, which equals 1.07 in NEN-EN 1992-2+C1:2011 (CEN 2011). This value is rounded off to 1.10. The target proof load to approve the structure is calculated for each safety level. According to the RBK (Rijkswaterstaat 2013), the recommended safety level for the assessment of existing bridges is the RBK Usage level. For the pilot proof load test, higher loads have been applied to study the behavior of the bridge under all safety levels.

The proof load needs to be equivalent to the loads from Load Model 1 of NEN-EN 1991-2:2003 (CEN 2003), which consists of a design tandem in each lane and a distributed lane load. The position of the proof load is determined as the most critical position for bending moment and the most critical position for shear. The proof load is applied as a single proof load tandem, of which the load magnitude needs to represent the design tandem in both lanes, and the distributed lane loads.

Case study: use of recommended target proof load in proof load test viaduct de Beek

On viaduct De Beek, the proof load test was carried out in span 1. The critical span for the assessment, and the span with the largest cracking damage, is span 2. However, span 2 is over the highway. Testing span 2 would require the closing of the highway for safety reasons, which is practically impossible. Therefore, span 1 is tested, and the results are then interpreted in the light of the assessment of span 2. As currently no methods are available to extrapolate results from a load test on one span to another span, an assessment of span 2 based on plastic redistribution will be presented later in this paper. Both a flexure- and shear-critical position are tested.

The following procedure is used to determine the required magnitude and position of the proof load for bending moment:

1. A linear finite element model of the bridge is developed. The loads that need to be considered are the self-weight of the concrete, the weight of the asphalt layer, and the live loads from Load Model 1 from NEN-EN 1991-2:2003 (CEN 2003).
2. The design tandems from Load Model 1 are moved in their respective lanes until the position of the tandems that causes the largest bending moment, distributed over 3 m in the transverse direction, is found. The corresponding position of the design tandem in the first lane is the critical position of the proof load tandem.
3. The live loads from Load Model 1 are removed and replaced by the proof load tandem at the critical position. The load on the proof load tandem is now increased until the same bending moment (distributed over 3 m transversely) is found as for the bridge subjected to the live loads from Load Model 1 at the critical position.

For viaduct De Beek the critical position is found at 3.55 m from the end support. This position (shown as position "A") is sketched in Fig. 4. The required values of the proof load at the different safety levels are then given in Table 3.

A similar procedure is used for the shear-critical position. The main difference is that the critical position is predetermined as $2.5d_l$ for the face-to-face distance between the load and the support (Lantsoght et al. 2013b). The distribution width in the transverse direction for the peak shear stress is taken as $4d_l$ per wheel load (Lantsoght et al. 2013a). For viaduct De Beek, the critical position for shear is at 1.1 m from the end support. The position of the proof load tandem for the shear test is shown as position "B" in Fig. 4. An overview of the required values of the proof load at the different safety levels is given in Table 3.

Resulting loading protocol

The load is applied with four hydraulic jacks and a load spreader beam, see Fig. 5, so that if a large deflection occurs, the load is removed from the bridge. The simulated tire contact area (steel loading plate) is 230 mm × 300 mm. The loading speed was determined as 5.4 kN/s in the bending moment test, and as 7.3 kN/s in the shear test. A cyclic loading protocol was chosen, as it allows for checking the stop criteria after each cycle, and linearity. In the bending moment test, the following loading steps, referring to the load levels from Table 1 and Table 2, see Fig. 6a, were used:

1. A low load level of 550 kN to check the functioning of all sensors.
2. A load level of 950 kN, which is slightly lower than the serviceability limit state.
3. A load level of 1350 kN, which corresponds with the RBK Usage level (Rijkswaterstaat 2013).

4. A maximum load of 1699 kN, which corresponds with the Eurocode Ultimate Limit State level.

The applied maximum load at the jacks was 1699 kN. Adding the weight of the setup, results in the maximum total applied load of 1751 kN, which is 6% above the calculated Eurocode Ultimate Limit State level. The additional percentage takes into account local material variability, and can be considered as a model factor for a proof load test.

In the shear test, the following load levels (Fig. 6b) were applied:

1. A low load level of 250 kN to check the functioning of all sensors.
2. A load level of 750 kN, which is slightly lower than the serviceability limit state.
3. A load level of 1250 kN, which corresponds with the RBK Usage level (Rijkswaterstaat 2013).
4. A maximum load of 1508 kN, which corresponds with the Eurocode Ultimate Limit State level.

The maximum applied load, including the weight of the setup, was then 1560 kN, or the calculated Eurocode ultimate limit state + 2%.

Determination of required measurements and stop criteria

Current practice

As mentioned earlier, the only codes and guidelines that contain stop criteria for concrete structures (originally developed for concrete buildings) are ACI 437.2M-13 (ACI Committee 437 2013) and the German guideline (Deutscher Ausschuss für Stahlbeton 2000), and these stop criteria are only valid for flexure. In ACI 437.2M-13 (ACI Committee 437 2013), the stop criteria depend on the loading protocol, which can be monotonic or cyclic. As the loading protocol for viaduct De Beek is cyclic, the focus here will be on the cyclic loading protocol. The cyclic loading protocol of ACI 437.2M-13 consists of three load levels with two cycles per load level. The first load level is the serviceability load level, and the final load level corresponds to the target proof load. In ACI 437.2M-13, the stop criteria are defined as acceptance criteria – criteria that need to be fulfilled for the acceptance of the structure after the proof load test. The first acceptance criterion is that the structure should show no evidence of failure. The second acceptance criterion is called the deviation from linearity index, I_{DL}, derived from the load-displacement diagram. The angles α are determined based on the origin of the load-displacement diagram and the maximum point in a load cycle. The acceptance criterion for the deviation from linearity index is determined as:

$$I_{DL} = 1 - \frac{\tan(\alpha_i)}{\tan(\alpha_{ref})} \leq 0.25 \tag{3}$$

The third acceptance criterion is the permanency ratio I_{pr}, expressed as:

$$I_{pr} = \frac{I_{p(i+1)}}{I_{pi}} \leq 0.5 \tag{4}$$

$I_{p(i+1)}$ and I_{pi} are the permanency indices for the (i+1)th and ith load cycles:

$$I_{pi} = \frac{\Delta_r^i}{\Delta_{\max}^i} \tag{5}$$

$$I_{p(i+1)} = \frac{\Delta_r^{(i+1)}}{\Delta_{\max}^{(i+1)}} \tag{6}$$

The final acceptance criterion is related to the residual deflection Δ_r, measured at least 24 hours after removal of the load. This value has to be smaller than or equal to 25% of the maximum deflection or 1/180 of the span length.

The second set of stop criteria comes from the German guideline for load testing (Deutscher Ausschuss für Stahlbeton 2000). This guideline uses a cyclic loading protocol of three load levels with at least one cycle per level. The first stop criterion is based on the measured strains in the concrete, ε_c:

$$\varepsilon_c < \varepsilon_{c,\lim} - \varepsilon_{c0} \quad (7)$$

The limiting strain $\varepsilon_{c,lim}$ is 0.8 ‰ if the concrete compressive strength is larger than 25 MPa, minus the strain ε_{c0} caused by the permanent loads. The second stop criterion is based on the measured strains in the steel reinforcement, ε_{s2}, which requires removal of the concrete cover:

$$\varepsilon_{s2} < 0.7\frac{f_{ym}}{E_s} - \varepsilon_{s02} \quad (8)$$

The third stop criterion evaluates the crack width w for new cracks and the increases in crack width Δw for existing cracks. New cracks can be maximum 0.5 mm, of which 30% is permitted as residual crack width, and existing cracks can increase with maximum 0.3 mm, of which 20% is permitted as residual crack width. The fourth stop criterion says that nonlinear behaviour should not take place, and that the residual deformation is limited to 10% of the maximum deformation.

Sensor plan for viaduct De Beek

Since the proof load test on viaduct De Beek was a pilot test and part of a program of proof load tests, the viaduct was heavily instrumented, so that the behavior of the viaduct could be closely monitored during the experiment. Another goal was to analyze the measurements after the test in order to come up with recommendations for proof loading of reinforced concrete slab bridges and to evaluate the existing stop criteria for flexure. The following responses of the bridge were measured:

1. The vertical deflections of the deck at different positions in the longitudinal and transverse direction are measured with linear variable differential transformers (LVDTs) and laser triangulation sensors.

2. The vertical deflections of the support beam are measured with LVDTs.
3. The strain in the reinforcement steel is measured at a few locations where the concrete cover is removed, and strain gages are applied to the steel.
4. The strain in the concrete is measured at the bottom surface by applying LVDTs over 1 m.
5. The opening of existing cracks is followed by applying an LVDT over the crack.
6. The applied load is measured with load cells at the four wheel print positions of the proof load tandem.

The position of the sensors is given in Fig. 7.

Measurements of viaduct de Beek

Some interesting measurements and post-processing results of the bending moment test are shown in Fig. 8. The first result that is studied is the load-deflection diagram, of which the envelope is given in Fig. 8a. The maximum deflection during the proof load test was 11 mm. From the results of the load-deflection diagram, the reduction of the slope over the applied load cycles can be studied, see Fig. 8c. A 25% reduction of the slope is indicated in Fig. 8c with a red line. It can be seen that during none of the load cycles this limit, which was proposed as a possible stop criterion based on beam tests in the laboratory (Lantsoght et al. (in press)), is exceeded.

Another element of post-processing is the determination of the deflection profiles in the longitudinal and transverse directions. The longitudinal deflection profile is given in Fig. 8d, from which it can be observed that the increases in deflection increase linearly with the load. The supporting calculations can be found in the background report (Koekkoek et al. 2016).

The measurements of the deflections and strains can be compared to the results of the linear finite element program. This comparison indicated that the stiffness of uncracked concrete, 32.9 GPa can be used for the finite element model. However, it must be noted that in the simplified finite element model possible additional sources of stiffness (Barker 2001), such as the effect of curbs and railings and the bearing restraint stiffness of aged bearings, were not taken into account. The strain measurements showed good correspondence between the steel and concrete strains. The calculated strains also corresponded reasonably well with the measured strains, see Fig. 8b.

For the shear position test, the most important measurements and post-processing results are shown in Fig. 9. The first result that is studied is the load-deflection diagram, of which the envelope is given in Fig. 9a. The maximum deflection during the proof load test was 7 mm. The reduction of the slope over the applied load cycles is shown in Fig. 9c. During none of the load cycles the limit of maximum 25% reduction of the slope is exceeded. The longitudinal deflection profile is given in Fig. 9d, from which it can be seen that under the applied loads the behavior was linear.

The measurements of the deflections and strains can be compared to the results of the linear finite element program. From the deflection results, it was concluded again that a stiffness of uncracked concrete, 32.9 GPa can be used, see Fig. 9b.

Evaluation of stop criteria

In this section, the existing stop criteria that are developed for buildings for flexure are evaluated. The residual deformation after the test was determined. In the bending moment test the ratio of the residual to maximum deflection was 15%, which does not fulfil the stop criterion of the German guideline but fulfills the acceptance criterion of ACI 437.2M-13. In the shear test the ratio of the residual to maximum deflection was 8%, which is below the limit of the German guideline and ACI 437.M-13.

The stop criteria for the strains from Eq. (7) and (8) must be verified. The strain caused by the self-weight of the concrete and the layer of asphalt is ε_{c0} = 163 με. The limiting strain $\varepsilon_{c,lim}$ = 800 με, so that the measured strain should be smaller than 637 με. This maximum is exceeded in the experiment, in the loading step leading up to the target load level, as can be seen in Fig. 8b. The stop criterion was exceeded at 97% of the target load. Loading to a higher load level than the target load level could have resulted in permanent damage to the structure. The limiting steel strain leads to a maximum strain of 857 με, which is not exceeded during the experiment. The stop criteria with regard to concrete and steel strains are not exceeded during the shear experiment. This observation is not surprising, since the shear position activates less flexural response.

The maximum measured opening of an existing crack during the bending test was 0.12 mm, after which the residual crack width was 0.03 mm. It is assumed that crack widths smaller than 0.05 mm can be neglected. The conclusion is then that the studied crack fully closed after the maximum load, and that no permanent damage was inflicted on the structure by the proof load test. The maximum measured opening of an existing crack was 0.11 mm during the shear test, after which the residual crack width was 0.01 mm. The studied crack fully closed after the maximum load.

Assessment of viaduct De Beek

Assessment of the tested span

All assessments for viaduct De Beek are carried out based on the original two lanes of traffic, to see if the current traffic restrictions (Fig. 1b) can be removed. All acting bending moments m_{Ed} are determined based on a transverse distribution of 3 m. With the reinforcement from Fig. 3, the moment capacity in span 1 is determined as m_{Rd} = 579 kNm/m. The factored acting moment in the cross-section with the load factors of the RBK Usage level, which is used for the assessment of existing highway bridges (Rijkswaterstaat 2013) is m_{Ed} = 463 kNm/m. As a result, the Unity Check for bending moment equals UC = 0.8. The Unity Check is determined as the ratio of the load effect over the capacity. This result does not correspond with the 2015 assessment of the bridge (Iv-Infra 2015), which resulted in the lane restrictions applied to the bridge. The 2015 assessment combined a calculation of the UCs based on a linear finite element model with a visual inspection. A comparison showed that the 2015 assessment did not consider all reinforcement as shown in Fig. 3. Moreover, the proof load test showed that the viaduct can carry the factored live loads of the Eurocode Ultimate Limit State.

Using the rating factor from the Manual for Bridge Evaluation (AASHTO 2016) resulted in RF = 1.32 > 1, so that the first span fulfills the requirements.

The shear capacity according to the RBK (Rijkswaterstaat 2013) was $v_{Rd,c}$ = 1.002 MPa. For the RBK Usage level, the acting shear stress is v_{Ed} = 0.482 MPa when using averaging over a distance of $4d_l$ (Lantsoght et al. 2013a), so that UC = 0.48. The first span thus fulfills the requirements for shear, prior to taking into account the information from the proof load test.

Assessment of span 2

According to the reinforcement drawings, Fig. 3, less reinforcement is present in span 2 as compared to span 1 (4 layers of ϕ 25 mm bars with a spacing of 560 mm as compared to 6 layers of ϕ 25 mm bars with a spacing of 560 mm), while span 2 has a larger span length. The moment capacity now is m_{Rd} = 335 kNm/m for the cross-section at the midspan. The bending moment caused by the factored loads acting on this cross-section is m_{Ed} = 422 kNm/m, so that

UC = 1.26, which means that the cross-section does not fulfill the requirements for bending moments under the RBK Usage loads (Rijkswaterstaat 2013). A further analysis of the cross-section is thus necessary.

In a next step, the analysis is carried out with plastic redistribution. In this case, the Unity Check for the hogging moment over support 2 is considered. The ultimate moment capacity at support 2 equals m_{Rd} = 1022 kNm/m. Using plastic redistribution means that a plastic hinge will form in the midspan cross-section when a moment of 335 kNm/m is achieved in this cross-section. If higher loads are applied, redistribution of the moment diagram will occur, and higher sectional moments will occur over the support. The moment m_{Ed} = 335 kNm/m is reached in the midspan cross-section at 78% of the full factored RBK Usage loads. The moment at support 2 is then m_{Ed} = 900 kNm/m. The midspan of the slab is now modeled as a plastic hinge over the full width of the slab. With this model, the acting bending moments under the factored RBK Usage live loads (Rijkswaterstaat 2013) are m_{Ed} = 960 kNm/m at support 2 and m_{Ed} = 335 kNm/m at midspan. The amount of plastic redistribution that has taken place is 6.7%. With plastic redistribution, UC = 0.94 over support 2 and UC = 1 at midspan. These results indicate that a direct assessment of span 2 based on the test results does not lead to a recommendation for the removal of the traffic restrictions. Only when plastic redistribution is allowed to take place, and cracking and the reduction of the durability of the structure are acceptable by the owner, the traffic restrictions can be removed.

The assessment for shear (Iv-Infra 2015) gave UC = 0.51 for the cross-section close to the intermediate support in span 2. The second span thus fulfills the requirements for shear.

Recommendations

Viaduct de Beek

Based on the presented analyses, it was recommended to check the reinforcement in span 2 with a scanner or by removing the concrete cover locally to verify the spacing between bars. The reinforcement layout presented in the plans is unexpected, since the longer middle spans are provided with less reinforcement. The acting bending moment for the RBK Usage level in span 1 is 463 kNm/m and in span 2 422 kNm/m. The reduction of the span moment due to the support moment is thus rather limited in the second span. It is also recommended to carry out an additional inspection of the cracks in span 2, and to carefully check for signs of corrosion, which would further reduce the flexural capacity. If the condition of span 2 is considered satisfactory in terms of present corrosion, the current traffic restriction can be removed, provided that plastic redistribution is allowed.

Lessons learned for proof load testing

The pilot proof load test shows that proof load testing can be carried out at flexure- and shear-critical positions. The determination of the target proof load is currently carried out based on equivalent sectional moments and shears. The presented method which uses a single proof load tandem is valid for bridges of small width.

The analysis of the stop criteria shows that the concrete strain criterion of the German guideline is suitable for the combination with proof load tests for flexure and shear. The criterion for the steel strains cannot always be used, as not all bridge owners allow for the removal of the concrete cover. The crack width criterion is useful, provided that cracks of less than 0.05 mm are neglected. The residual deflection of 10% is rather conservative; the value of 25% from ACI 437.2M-13 could be more suitable. The other stop criteria from ACI 437.2M-13 could not be evaluated, as these are directly associated with the loading protocol of ACI 437.2M-13, which was not the same as the loading protocol used for viaduct De Beek. Stop criteria to evaluate possible shear failure still need to be developed.

Summary and Conclusions

The viaduct De Beek is a reinforced concrete slab bridge with a traffic restriction that reduces the use of the viaduct from one lane in each direction to a single lane, as the bending moment capacity was found to be insufficient for the prescribed loads. The bridge was evaluated in a pilot proof load test, which also served to study if proof load testing for shear is possible, and if the existing stop criteria derived for buildings can be used in proof load tests for bridges. As the stop criteria are a topic of research, a large number of sensors were applied on the viaduct to closely monitor the structural response during the test.

A proof load test was carried out at a flexure- and shear critical position in the first span of the viaduct. For both tests, the target proof load was achieved. The analysis of the measurements showed that the structural response remained sufficiently close to the linear behavior. However, some stop criteria from the German guideline were exceeded, which indicates that further loading of the structure could have resulted in permanent damage to the structure. Further research should focus on the development of stop criteria for shear.

The assessment with the Unity Checks showed that the capacity of span 1 is sufficient, and was proven to be sufficient in the proof load tests, but the capacity of span 2 cannot directly be proven to be sufficient. In an additional analysis, plastic redistribution was allowed. It was found that if 6.7% of plastic redistribution is allowed to take place, the Unity Checks at the support and in the midspan cross-section of span 2 can fulfill the requirements, provided that a reduction of the durability is accepted.

Notation List

The following symbols are used in this paper:

d_l	effective depth to the longitudinal reinforcement
f_{cd}	design concrete compressive strength
f_{ck}	characteristic concrete compressive strength
f_{ctm}	characteristic tensile splitting strength of the concrete
f_{tm}	average tensile strength of the steel
f_{yd}	design yield strength of the steel
f_{ym}	average yield strength of the steel
m_{Ed}	design action moment on cross-section
m_{Rd}	design resistance moment of cross-section
$v_{Rd,c}$	design shear resistance
A_s	longitudinal reinforcement
E_s	modulus of elasticity of reinforcement steel
I	the AASHTO specifications impact allowance
I_{DL}	deviation from linearity index
I_{pi}	permanency index for the *i*-th load cycle
I_{pr}	permanency ratio
K_a	updating factor based on test results
K_b	updating factor based on situation of considered structural member
L_R	the comparable live load due to the rating vehicles for the lanes loaded
L_T	target proof load
$P_{load,bending}$	required proof load for bending moment

$P_{load,shear}$	required proof load for shear
RF	rating factor
RF_T	updated rating factor based on proof load test results
UC	unity check
X_p	factor to determine target proof load, without adjustments
X_{pA}	target live load factor
α	angle of line between origin of load-displacement diagram and maximum value of considered load cycle
α_i	angle of line between origin of load-displacement diagram and maximum value of load cycle *i*
α_{ref}	angle of line between origin of load-displacement diagram and maximum value of first load cycle
β	reliability index
γ_{as}	load factor for the superimposed dead load
γ_{ll}	load factor for the live load
γ_{sw}	load factor for the self-weight
ε_c	the theoretically determined strain in the finite element model under the maximum proof load
$\varepsilon_{c,meas}$	strain measured during proof loading
$\varepsilon_{c,lim}$	limit value of the concrete strain : 0.6 ‰, and for $f_{cd} \geq 25$ MPa this can be increased up to maximum 0.8 ‰.
ε_{c0}	analytically determined short-term strain in the concrete caused by the permanent loads that are acting on the structure before the application of the proof load

ε_{s02} analytically determined strain (assuming cracked conditions) in the reinforcement steel caused by the permanent loads that are acting on the structure before the application of the proof load.

ε_{s2} steel strain during experiment: directly measured or derived from other measurements

ε_T the measured strain during the proof load test under the maximum proof load

Δ^i_{max} the maximum deflection after the *i*-the load cycle

Δ^i_r the residual deflection (non-cumulative) after the *i*-th load cycle

Δ_r residual deflection, measured at least 24 hours after removal of the load

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Table 1. Overview of different safety levels used in the Netherlands for the assessment of existing highway bridges (Data from CEN 2002; Rijkswaterstaat 2013).

Reliability level	***β***	**Reference period**
Eurocode Ultimate Limit State	4.3	100 years
RBK Design	4.3	100 years
RBK Reconstruction	3.6	30 years
RBK Usage	3.3	30 years
RBK Disapproval	3.1	15 years
Eurocode Serviceability Limit State	1.5	50 years

Table 2. Overview of load factors associated with the different reliability levels as used for proof load testing.

Reliability level	γ_{sw}	γ_{as}	γ_{ll}
Eurocode Ultimate Limit State	1.10	1.35	1.50
RBK Design	1.10	1.25	1.50
RBK Reconstruction	1.10	1.15	1.30
RBK Usage	1.10	1.15	1.25
RBK Disapproval	1.10	1.10	1.25
Eurocode Serviceability Limit State	1.00	1.00	1.00

Table 3. Determined required proof load for bending moment and shear

Reliability level	$P_{load,bending}$ **[kN]**	$P_{load,shear}$ **[kN]**
Eurocode Ultimate Limit State	1656	1525
RBK Design	1649	1516
RBK Reconstruction	1427	1311
RBK Usage	1373	1262
RBK Disapproval	1369	1257
Eurocode Serviceability Limit State	1070	976