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Required proof load magnitude for probabilistic field assessment of viaduct De Beek

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Abstract

Proof load testing is part of the engineering practice, and can be particularly useful for the rating of existing bridges. This paper addresses how reliability-based concepts can be used in combination with proof load testing, and discusses how this approach differs from the current practice for proof load testing. Whereas the calculation methods for determining the updated reliability index after a proof load test are available in the literature, this approach is now used to determine the proof load magnitude required to demonstrate a certain reliability level in a bridge, the viaduct De Beek. To determine the required proof load magnitude, the known integrals of the limit state function are solved. The method is applied to a case of a bridge that was proof load tested in the Netherlands, viaduct De Beek. The data of this bridge are used to determine the required proof load magnitude to fulfill a given reliability index. A sensitivity study is carried out to identify the effect of the assumptions with regard to the coefficient of variation on the resistance and load effects. The result of this approach is that large loads are necessary in proof load testing if a reliability index needs to be proven in a proof load test. In the current practice of proof load testing with vehicles, it can typically only be demonstrated that a certain vehicle type can cross the bridge safely. The results in this paper provide a new insight on the required proof load magnitudes to show that the reliability index of the tested bridge is sufficient. However, consensus on the coefficients of variation that need to be used on the resistance and load effects, is still missing, which significantly affects the results for the required proof load magnitudes.

Keywords

Assessment; Existing bridges; Field testing; Load effects; Proof load testing; Reliability-based assessment

1. Introduction

Proof load testing of bridges is a practice almost as old as bridge engineering itself. In the past, prior to opening a bridge for the traveling public or during the opening ceremony of a bridge, a proof load test had to be carried out to demonstrate the proper functioning of the bridge [1]. Sometimes, indeed, the proof load test failed and the bridge collapsed. Examples of collapses include the steel-framework bridge over the Morawa near Ljubitschewo in Serbia, the road bridge near Salez in Switzerland, and the suspension bridge in Maurin, France [1].

Nowadays, the available engineering models and calculation methods are finer than in the past. Nonetheless, in countries like France [2], Italy [3], the Czech Republic, Slovakia [4] and Switzerland [5], proof load tests prior to opening are still standard practice. In other parts of the world, load tests at opening are only used for special bridges and/or when novel concepts are used, to verify the design assumptions.

For the assessment of existing bridges, proof load tests have been used since the 1890s in Switzerland together with visual inspections. In late 19th century Switzerland, a bridge was loaded with the full live load. It was then considered as sufficient if it did not collapse and fulfilled deflection and/or vibration criteria.

Nowadays, the challenge for the infrastructure networks in the industrialized world lies in maintaining, repairing and, where needed, replacing the existing infrastructure, and in particular the existing bridges. Standard ratings based on calculations can be used. All input for these calculations is not always available. For example, the effect of material degradation on the strength of cross-sections can be rather difficult to estimate [6], or structural plans can be missing [7, 8]. For those cases, load testing can be used.

Two types of load tests should be distinguished: diagnostic load tests and proof load tests. Diagnostic load tests [9-13] apply relatively low loads, and are used to compare the bridge's response to the analytically determined response. The results of the diagnostic load test can then be used to update the analytical calculation method, often in practice a finite element model. The updated model can then be used for the rating of the tested bridge. Proof load tests [7, 14-18] serve a different purpose. In a proof load test, a large load is applied, after which it can be determined that the bridge has sufficient capacity to carry the prescribed live loads. In North America, this large load is typically carried by a vehicle, loaded more heavily than the maximum load that can be expected to cross the bridge [7]. In Europe, the factored live load model is used [17], and a load is then applied that creates the same sectional shear or moment as the factored live load model. This approach requires larger loads, which are either applied by a system of hydraulic jacks and counterweights [19], or by a custom vehicle, such as the German BELFA (BELastungsFAhrzeug, loading vehicle) [20-22]. When the factored live load model is used to find the equivalent proof load, it is tacitly assumed that the resulting reliability index and probability of failure after the test (if the test is successful) of the bridge have the same value as what the load factors are calibrated for. This paper will explore this assumption, calculate the reliability index before, during, and after a proof load test, and will ultimately discuss the current barriers to a fully reliability-based proof loading strategy for existing bridges.

2. Field assessment for rating of bridges

2.1. Guidelines for proof load testing

Guidelines for proof load testing of bridges are not commonly available, except for the Manual for Bridge Evaluation (MBE) [23]. The requirements from the MBE are based on an NCHRP

report from 1998 [24], which focuses more on diagnostic load testing than on proof load testing. The MBE prescribes that the "test loads must provide for both the rating vehicles, including the dynamic load allowance, and a load factor for the required margins of safety".

The load factor, X_p, is the factor required to achieve a bridge rating of 1.0. After the adjusted load factor X_{pA} is obtained, with adjustments for site conditions, the factor is multiplied by the rating load plus dynamic load allowance to get the proof load magnitude required for a rating factor of 1.0. The rating factor *RF* is defined in the MBE as follows:

$$RF = \frac{C - (\gamma_{DC})(D_w) - (\gamma_{DW})(D_s) \pm (\gamma_P)(P)}{(\gamma_{LL})(L + IM)} \quad (1)$$

with, for the Strength Limit States:

$$C = \varphi_c \varphi_s \varphi R_n \text{ with } \varphi_c \varphi_s \geq 0.85 \quad (2)$$

and for the Serviceability Limit States:

$$C = f_R \quad (3)$$

The rating factor *RF* is the available capacity for live load, and is expressed as a function of the capacity *C*, the self-weight D_w, the superimposed dead load D_s, the other permanent loads *P*, the live load *L*, the dynamic load allowance *IM*, and the partial factors for self-weight γ_{DC}, for superimposed dead load γ_{DW}, for permanent loads γ_P and for live load γ_{LL}. The capacity is a function of the nominal member resistance R_n for the strength limit states, multiplied with the condition factor φ_c, the system factor φ_s and the LRFD [25] resistance factor φ. For the serviceability limit states, an allowable stress specified in the LRFD code [25], f_R, is used.

The MBE recommends a base value for X_p of 1.40, and should be replaced with the permit load factors from Table 6A.4.5.4.2a-1 from the MBE if the rating is carried out for a permit load. The value X_p = 1.4 was determined based on a first order reliability calculation,

assuming only normal distributions. The values assumed for the bias (mean to design value) and coefficient of variation are given in Table 2. The bias for the live load is based on measurements, and then extrapolated to 75 years to find the mean maximum load as 1.79 HS20 vehicles. The coefficient of variation on the live load is either 18% when both the uncertainties on the heavy truck occurrences and the uncertainties of the effect of these trucks on the members of the structure are considered, and 14% when only the uncertainties of the truck occurrences is considered. The main uncertainties after a proof load test that need to be factored in are the magnitude of future live loads and possible future deterioration. The value of X_p was derived based on an example with an 18 m span with D = L, and was verified for a shorter span and longer span with D/L = 3.0.

According to the MBE, the strength based on the test R_n is a function of X_p, the live load L, the impact factor I and the dead load D:

$$R_n = 1.40\left(L + I\right) + D \tag{4}$$

and the strength based on calculation R_n needs to include the factor for dead load γ_D and for live load γ_{LL}:

$$R_n = \gamma_{LL}\left(L + I\right) + \gamma_D D \tag{5}$$

The adjustments to X_p are as given in Table 1. These adjustments were qualitatively determined in the Manual for Bridge Rating through Load Testing [24] so that X_{pA} becomes:

$$X_{pA} = X_p\left(1 + \frac{\Sigma\%}{100}\right) \tag{6}$$

The target proof load L_T is then the live load of the rating vehicle L_R, magnified with the dynamic factor IM and X_{pA} from Eq. (6):

$$L_T = X_{pA} L_R \left(1 + IM\right) \tag{7}$$

For multiple-lane bridges, a minimum of two lanes should be loaded concurrently. The load should be applied in stages, with the first-stage loading not exceeding $0.25L_T$ and the second-stage loading not exceeding $0.5L_T$. After the proof load test, the rating factor RF_0 is determined as:

$$RF_0 = \frac{OP}{L_R \left(1 + IM\right)} \tag{8}$$

with the operating level capacity OP:

$$OP = \frac{k_0 L_p}{X_{pA}} \tag{9}$$

with L_p the maximum proof live load, and k_0 a factor which takes into account how the proof load test was terminated. The value of $k_0 = 1$ if the target load was reached and $k_0 = 0.88$ if signs of distress were observed prior to reaching the target proof load. The background to this approach is given in the annex to the Manual for Bridge Rating through Load Testing [24].

Internationally, current guidelines for the load testing of bridges exist in Ireland [26], in Great Britain [27], and in France [2]. These guidelines only permit the use of diagnostic load tests. The German guideline, which deals with proof load tests, [28] was originally developed for plain and reinforced concrete buildings [1], but is now also applied to concrete bridges [29]. The ACI code 318-14 [30] briefly touches upon proof load testing of new concrete buildings. More detailed provisions for the determination of the proof load and the loading sequence for existing concrete buildings are given in ACI 437.2M-13 [31]. For concrete buildings, the target proof load, when only part of the portions of a structure are suspected containing deficiencies or that have been repaired, and when the members are statically indeterminate, is determined as:

$$TLM = 1.3\left(D_W + D_S\right) \tag{10}$$

$$TLM = 1.0D_W + 1.1D_S + 1.6L + 0.5\left(L_r \text{ or } SL \text{ or } RL\right) \tag{11}$$

$$TLM = 1.0D_W + 1.1D_S + 1.6\left(L_r \text{ or } SL \text{ or } RL\right) + 1.0L \tag{12}$$

In Equations (10), (11), and (12), *TLM* is the test load magnitude, D_w is the load caused by the self-weight of the concrete, D_s is the superimposed dead load, L is the live load, L_r is the live load on the roof, *SL* is the snow load and *RL* is the rain load. This load level was originally determined as 85% of the ULS load combination from the ACI 318 code [30]. The target load can be reduced when all suspect portions are to be load tested, when flaws are controlled by flexural tension, or when the structure is statically determinate.

2.2. Experience in the Netherlands

In the Netherlands, the option of using proof load testing to demonstrate sufficient capacity in the existing bridges is under investigation. For this purpose, a number of pilot proof load tests [32] have been carried out, see Table 3. Throughout the pilot program, the following approach was developed for concrete bridges to determine the target proof load to verify bending moment or shear:

1. A linear finite element model of the bridge is constructed, and the applied loads (dead load, superimposed load, and distributed and concentrated live loads) are factored loads.
2. The concentrated live loads are moved along the span length to find the critical position, which causes the largest sectional moment. For shear, the critical position is taken at $2.5d_l$ from the support.
3. The factored live loads are then removed, and replaced with two axles of the proof load tandem at the critical position. The load on the proof load tandem is increased until the

same maximum sectional moment or shear as for the factored load combination is found. This load determines the target proof load.

The considered linear finite element model only serves the purpose to find the position and magnitude of the proof load. Therefore, it is sufficient to use shell elements, use idealized support conditions, and to replace barriers, sidewalks, and other elements with a load representing the self-weight of these elements.

The considered load factors depend on the safety level. For new structures, the safety levels from the Eurocode NEN-EN 1990:2002 [33] should be followed. For the assessment of existing structures, the Eurocodes are still under development. Therefore, in the Netherlands, a series of codes, the NEN 8700-series, is developed following the safety philosophy and code numbering of the Eurocodes: the basis for assessment in NEN 8700:2011 [34], the load factors in NEN 8701:2011 [35], concrete structures in NEN 8702 (under development), etc. For existing bridges, these concepts are gathered in the Guidelines Assessment Bridges "RBK" [36] for the Netherlands. Different safety levels with different reference periods were derived [37], resulting in different sets of load factors as shown in Table 4. When the load factors are applied for proof loading, γ_{DC} can be replaced by 1.1. The partial factor combines the material, geometry, and model uncertainties. For an existing bridge, only the model uncertainty of the self-weight remains, defined as 1.07 in NEN-EN 1990:2002 [33], because the structure is built and the materials and geometry have become deterministic. The value of 1.07 was rounded off to 1.10 for practical reasons.

The considered load combination for assessment is self-weight, superimposed dead load, and live loads as prescribed by NEN-EN 1991-2:2003/NA:2011 [38]. The live loads consist of a

distributed lane load and a design tandem in each lane. Therefore, to find the largest possible sectional moment or shear, the design tandems need to be moved in their respective lanes.

Recommendations for the loading protocol and for the stop criteria were defined as well [39-41]. During the proof load tests, the stop criteria from the German guidelines [28] were used. These stop criteria have then be evaluated after the pilot proof load tests, and additional experiments in the laboratory on beams were carried out to propose different stop criteria for bending moment and shear [40]. The stop criteria are based on the measurements of the sensors, and if one of these criteria is exceeded, there is an indication that further loading can cause irreversible damage to the structure. The applied sensors typically are LVDTs (linear variable differential transformers), laser distance finders, and acoustic emission sensors, to measure deflections, crack widths, strains, and acoustic emissions. The results are recommendations for proof load testing of reinforced concrete slab bridges [42], a common bridge type in the Netherlands. Many of the existing slab bridges rate low, so that the research effort was mostly geared towards the implementation for this bridge type.

2.3. Determining the reliability index after load testing

Methods for determining the reliability index before, during, and after proof load testing are available in the literature [43-46]. Using the distribution functions of the load S and resistance R, the limit state function can be determined as $g = R - S$, and failure occurs when $g < 0$. The probability that $g < 0$ is also expressed as the probability of failure P_f, from which the reliability index β is found as:

$$\beta = \Phi^{-1}\left(1 - P_f\right) \tag{13}$$

The probability prior to the proof load test can be determined as:

$$P_{fb} = \int_{-\infty}^{+\infty} \left(1 - F_s\left(r\right)\right) f_R\left(r\right) dr \tag{14}$$

During the proof load test, the load is a deterministic value s_p:

$$P_{fd} = F_R\left(s_p\right) \tag{15}$$

After the proof load test, it is known that the capacity is equal to or larger than s_p, so that:

$$P_{fa} = \frac{1}{1 - F_R\left(s_p\right)} \int_{s_p}^{+\infty} \left(1 - F_s\left(r\right)\right) f_R\left(r\right) dr \tag{16}$$

These probability density functions are sketched in Fig. 1. The procedure is illustrated in Fig. 2.

In order to use the convolution integral from Eq. (16) to determine the required target proof load and to demonstrate a certain reliability index, the distributions of the load and resistance have to be described. A starting point for the distributions can be taken from the recommendations of the Probabilistic Model Code of the Joint Committee on Structural Safety (JCSS) [47, 48] for the model uncertainties. An overview of the relevant distributions of the load and resistance is given in Table 5.

For the application to a proof load test [15, 46, 49-53] that aims at demonstrating sufficient bending moment capacity in a reinforced concrete slab bridge, for example, the load effect would be “moments in plates” and the resistance model “bending moment capacity”. In this study, the convolution integrals were programmed into a MathCad spreadsheet, and the value of the target proof load s_p was changed until the required reliability index after proof loading according to the safety formats from Table 4 was found. The results of this approach are illustrated with the example of viaduct De Beek.

According to NEN-EN 1990:2002 [33] Annex C the influence factor α for stochastic loads and resistance is taken as α_S = -0.7 and α_R = 0.8. In a proof load test, the value of α_R = 0.8

is used for multiplication with the reliability index β. As such, the information that is obtained in the proof load test, i.e. that the capacity is larger than or equal to the capacity achieved in the proof load test, is taken into account and a lower reliability index can be used. An overview of the target reliability indices for Consequences Class 3 after proof load testing is added to Table 4.

For the development of the target load factor from the MBE [54], the target values of the reliability index β were defined as 3.5 for the inventory design levels and 2.3 for the operating rating levels. These values were said to reflect "past rating practices at the operating levels" [24].

3. Viaduct De Beek

3.1. Description of bridge

Viaduct De Beek [55, 56] is a reinforced concrete slab bridge with four spans (end spans of 10.81 m and central spans of 15.40 m) over highway A67 in the province of North Brabant in the Netherlands. It has been in service since 1963. In 2015, a visual inspection and an assessment for the current live loads [57] indicated that a load restriction is necessary. Prior to proof load testing of the viaduct, an inspection of the viaduct was carried out. Significant cracking (crack widths between 0.3 mm and 0.6 mm and spaced 150 to 200 mm center-to-center) in the transverse direction was observed in spans 2 and 3.

3.2. Geometry of viaduct De Beek

The bridge is 9.94 m wide, with a carriageway of 7.44 m. The original lane layout was two lanes (one lane of 3.5 m wide each way), which has been replaced by one single lane since 2015. The thickness varies parabolically from 470 mm to 870 mm. An overview of the geometry is given in Fig. 3. Additionally, an asphalt layer with a thickness between 50 mm and 75 mm is

present. Cross-sectional checks in 2015 by an engineering firm led to the conclusion that the flexural capacity of the slab is insufficient [58].

3.3. Material properties

Prior to the load test, the characteristic concrete compressive strength was determined based on drilled cores as f_{ck} = 44.5 MPa, leading to a design compressive strength of f_{cd} = 30 MPa. Sampling of the steel led to the conclusion that steel QR 24 was used. This reinforcement type consists of plain bars with a design yield strength of 240 MPa. The measurements showed an average yield strength of f_{ym} = 291 MPa, tensile strength of f_{tm} = 420 MPa and the design yield strength was derived as f_{yd} = 252 MPa.

3.4. Determination of critical cross-section for bending moment and shear

A limitation for the execution of the proof load test on viaduct De Beek was that only the first span could be tested. Testing the more critical second or third spans was not allowed, as these spans are above the highway. To safely test these spans, the highway would have to be closed, which was not permitted by the road authority.

First, the assessment calculations for the first span are discussed for bending moment and shear. To determine the bending moment capacity, it must be taken into account that the thickness of the cross-section changes along the span, and that the reinforcement layout changes. An overview of the reinforcement in span 1 is shown in Fig. 4. To determine the bending moment caused by the combination of the self-weight, superimposed dead load, and live loads according to NEN-EN 1991-2:2003 [59], a linear finite element model is used as explained in §2.2. It is found that the largest sectional bending moment caused by the Eurocode loads is found when the first design tandem is placed at 3.55 m from the end support. This critical position is used for the assessment. The load levels RBK Usage and Eurocode ULS from Table 4 are used

for the assessment. The resulting comparison between the load and resistance at the RBK Usage and Eurocode ULS load levels are then shown in Fig. 5. At the RBK Usage level, the Unity Check (ratio of acting moment to moment capacity) is maximum UC = 1.02 and at the Eurocode ULS level, the maximum value is UC = 1.10, which indicates that the section does not fulfill the requirement for the Eurocode ULS level according to the assessment.

For shear, it is known that for reinforced concrete slab bridges the critical cross-section is close to the support [60]. The shear capacity is calculated according to the RBK [36] recommendations for existing reinforced concrete slab bridges:

$$v_{Rd,c} = 0.12 k_{cap} k \left(100 \rho_l f_{ck}\right)^{1/3} \geq v_{\min} = 0.83 \times k_{cap}^{3/2} \times k^{3/2} \sqrt{\frac{f_{ck}}{f_{yk}}} \quad (17)$$

For viaduct De Beek, k_{cap} = 1.2, ρ_l = 1.437% in the considered cross-section, f_{ck} = 44.5 MPa, f_{yk} = 240 MPa and

$$k = 1 + \sqrt{\frac{200mm}{d_l}} = 1 + \sqrt{\frac{200mm}{366mm}} = 1.74 \quad (18)$$

As a result, $v_{Rd,c}$ = 1.002 MPa ≥ v_{min} = 0.680 MPa. The acting shear stress is found by placing the first design tandem of the Eurocode live load model at a distance 2.5d_l from the support, which equals a distance 1050 mm. The resulting value for the acting shear stress at the RBK Usage level is v_{Ed} = 0.482 MPa, so that for shear, the value of UC = 0.48. It can thus be concluded that the cross-section fulfills the requirements for shear.

After the strength calculations and assessment, the required loads for proof load testing were determined. Both the positions that are critical for bending moment and shear in span 1 are proof load tested. Even though the structure is not shear-critical, it was decided to test the shear-critical position for research purposes. As described in §2.2., the magnitude of the required load

on the proof load tandem (one single tandem as used in Eurocode load model 1 [59]) at the critical position is determined by seeking the load that creates the same sectional shear or sectional moment as the load combination that includes the Eurocode live loads, replacing γ_{DC} by 1.1. An overview of these values is given in Table 6. Note that to fulfil the requirements of the RBK [36], only the RBK usage level needs to be demonstrated for existing bridges.

3.5. Execution and result of the proof load test

The values for the target proof load from Table 6 were used to develop the loading protocol for the load test. As large loads were required, the loads were applied by using a steel spreader beam (with supports coinciding with the supports of the bridge superstructure), counterweights, and a system of four jacks to transfer the load gradually and in a controlled manner to the bridge. An overview of the loading protocol as applied during the test is shown in Figure 6.

The structural safety of the bridge was verified during the proof load test by following the responses of all sensors in real-time, and verifying the stop criteria from the German guideline [28], ACI 437.2M-13 [31], and stop criteria that were developed as part of this research [40]. No signs of distress were observed during the proof load tests. The maximum load that was applied during the bending moment test was 1751 kN (including the weight of the equipment), which corresponds to the Eurocode ULS safety level, plus 6% extra. The additional loading beyond the Eurocode ULS safety level was done to gain more insight in the behavior of the bridge at high load levels for research purposes, and to have an extra safety factor on the method of proof load testing. Further research should determine if such an additional safety factor is required at all. The proof load test thus showed that the viaduct De Beek has sufficient capacity in bending

moment in span 1. For the shear test, the maximum applied load (including the weight of the equipment) was 1560 kN, which corresponds to the Eurocode ULS safety level, plus 2% extra.

For the safety of the executing personnel, a safety engineer was present on site, who give a safety briefing to all other personnel involved. During the proof load test, nobody was allowed to go under the tested span. A communication line was maintained throughout the entire experiment between the operators of the load and the measurement engineers. For every manipulation of the load, the operators waited for input of the measurement engineers, who analyzed the response of the structure in real-time and verified the stop criteria after each load cycle. To guarantee the safety of the traveling public on the bridge, the bridge was closed down for the entire duration of the preparation of the test, execution of the test, and removal of all material from the test site. All traffic was rerouted for this period of time.

4. Required proof load magnitude from a probabilistic perspective

4.1. Determination of limit state function and random variables

The general expression for the limit state is, as given before and as known from the literature discussed in §2.3, $g = R - S < 0$ with R the resistance and S the load. Applying this concept to the bending moment capacity (the limit state for which the assessment showed insufficient capacity) of viaduct De Beek gives the following limit state:

$$g = m_R - m_S < 0 \tag{19}$$

The mean values of the bending moment capacity and sectional bending moment need to be determined to set up the parameters of the distributions. In the first span, the bending moment capacity was determined as m_R = 673 kNm/m based on mean values of the material parameters.

The sectional bending moment caused by the self-weight, superimposed dead load and distributed and concentrated live loads from NEN-EN 1991-2:2003 [38] was determined by using the linear finite element model introduced previously. The acting bending moment for the load combination including the Eurocode live loads on the critical position equals m_s = 385 kNm/m (without load factors). In accordance with the JCSS Probabilistic Model Code, lognormal distributions are used for the probability density functions of m_R and m_S. The bending moment capacity uses a mean of 1.2 and a coefficient of variation of 0.15 (see also Table 5), and the sectional bending moment can be considered the case of moment in plates, so that the mean equals 1.0 and the coefficient of variation is 20%.

A MathCad sheet is used to solve the convolution integral from Eq. (14) and to determine the reliability index β for Eq. (19). The probability density function of m_R and m_S as well as the cumulative distribution functions are given in Fig. 7. Prior to the load test, the reliability index is β_b = 3.02. During the proof load test, a maximum load of 1751 kN was applied, which causes a moment (including the self-weight) of 597 kNm/m. As can be seen, this moment is 1.55 times the moment caused by the Serviceability Limit State load combination. The expression from Eq. (15) is then used to determine the reliability index during load testing, and the expression from Eq. (16) determines the reliability index after load testing. The results are that the reliability index during testing reduces to β_d = 1.95 and that the reliability index after testing has increased to β_a = 3.23, which is in between the values from Table 4 for RBK Reconstruction and RBK Design for $\alpha\beta$. The probability density function of m_s and the updated function of m_R are shown in Fig. 8. The maximum value during the proof load test was determined by finding the required load on the proof load tandem to achieve the same sectional moment as the factored loads as explained in §2.2. Note that the load factors from the RBK Design Level, see Table 4, were used,

but that the achieved reliability index after load testing was slightly smaller than the RBK Design Level $\alpha\beta$ = 3.44.

4.2. Results for different proof load magnitudes

In a next step, the value of the proof load is varied to find out which loads would be required based on the solution of the convolution integrals to achieve the reliability index specified by the different safety levels from Table 4. The results of this analysis are given in Table 7. No values for the RBK Disapproval, Usage, and Reconstruction levels could be found, since the value of β_b was already larger than the required value of $\alpha\beta$. From a probabilistic point of view, it would thus only be interesting to test with a load that corresponds to the RBK Design level.

4.3. Comparison with MBE approach

Since all the previous calculations are based on the Eurocode live loads, the MBE approach will now be applied to the Eurocode design tandems. Considering two lanes, with a tandem of 300 kN axles in the first lane, and a tandem of 200 kN axles in the second lane, results in L_R = 1000 kN. Note that this value is significantly larger than the rating vehicles used with the MBE, but no rating vehicles are prescribed in the Eurocodes. Considering that the load test is executed as a static test, IM = 1.0. The target proof load is then:

$$L_T = X_{pA} L_R \left(1 + IM\right) = 1.4 \times 1000 kN = 1400 kN$$

This target proof load corresponds with a sectional moment of m = 453 kNm/m. The reliability index prior to testing is still β_b = 3.02, and with a target proof load of 1400 kN the reliability index after testing becomes β_a = 3.04. Therefore, from a reliability-based standpoint, the approach from the MBE determines a load that is not large enough to significantly update the probability density function of the resistance and change the reliability index. Since the Eurocode

live load model is used for this case, a comparison to the target reliability indices used for deriving X_{pA} cannot be made.

4.4. Sensitivity study of coefficient of variation

A sensitivity study is carried out to identify the effect of the assumptions with regard to the coefficient of variation on the resistance and load effects. All previous calculations are based on the coefficients of variation as prescribed by the JCSS Probabilistic Model Code, see Table 5. However, these recommended values could be subject to discussion. For the bending moment capacity of concrete members, it could be said that the only variable is the yield strength of the steel. The coefficient of variation of steel is 7% [48, 61]. However, this value should be used cautiously, since it is derived for modern steel types, but could not be valid for the QR24 steel used in viaduct De Beek. Additionally, the mean value of 1.2 given in Table 5 can be considered rather high for modern steel types. For the loading side of the equation, m_s, a high coefficient of variation of 20% is prescribed for moments in plates. In the derivation of the Dutch code NEN 8700:2011 [34], a coefficient of variation of 10% [62] for the load effect was used. Moreover, the coefficient of variation for stresses in 3D models according to the JCSS Probabilistic Model Code, see Table 5, is only 5%. Since part of the model used solid elements, the coefficient of variation can be expected to lie somewhere between 5% and 20%. Finally, it can be remarked that the recommendations from the JCSS Probabilistic Model Code are general recommendations, valid for all types of structures (buildings, bridges, new structures, existing structures, different construction materials), and that for existing structures the reference period is shorter, so that the coefficient of variation is also smaller. With these considerations in mind, a sensitivity study was carried out to study the effect of the assumptions of the coefficient of variation on the reliability index before, during, and after the load test. The sensitivity study is

based on the results of viaduct De Beek, and the applied load of 1751 kN, resulting in m = 597 kNm/m. The results of the sensitivity study are given in Table 8. The original case, using the JCSS Probabilistic Model Code values for the distributions, is highlighted in grey. From the results in Table 8, it can be concluded that consensus about the required values for the coefficients of variation, perhaps adjusted for existing structures, is necessary. The reliability index prior to testing varies from infinity when small values for the coefficient of variation are used, to 2.29, which is lower the absolute lower bound of 2.5 for the loss of human life [37]. Similarly, the reliability index after load testing varies from infinity when small coefficients of variation are assumed, to 2.85. For small coefficients of variation, the effect of carrying out a load test is small.

5. Discussion

The result of the presented approach is that large loads are necessary in proof load testing if a reliability index needs to be proven in a proof load test. In the current practice of proof load testing with vehicles, it can typically only be demonstrated that a certain vehicle type can cross the bridge safely; a certain probability of failure cannot be derived from a proof load test unless large proof loads are applied. When using large proof loads, the reliability index during the load test needs to be determined to see if the risk of testing is acceptable. The results in this paper provide a new insight on the required proof load magnitudes to show that the reliability index of the tested bridge is sufficient, and aims at opening the discussion on whether proof load tests should have as a goal to proof a certain reliability index and probability of a bridge, or if it is sufficient to know that a certain type of vehicle can pass safely, taking a safety margin into account.

The determination of the coefficient of variation is crucial for the preparation of proof load tests. Consider for example the effect of changing the coefficient of variation of the resistance model, keeping all other parameters equal, as illustrated in Fig. 9. If the coefficient of variation is small, the left tail of the probability density function of the resistance will be small, and a larger proof load will be necessary to demonstrate a certain reliability index after proof load testing. The risk of collapse or damage during the proof load test, expressed by the reliability index during the test, may be acceptable. However, for the case where the coefficient of variation is larger, the left tail of the probability density function of the resistance will be more smeared out. With a lower proof load, the same reliability index after proof load testing can then be demonstrated. However, the calculated risk of collapse or damage during the proof load test, expressed by the reliability index during the test, will become much larger. Along the same lines, a study of the probability of failure in bending moment as compared to failure in shear for the Ruytenschildt Bridge [63] showed that, while a deterministic calculation indicated shear failure before bending moment failure, a probabilistic analysis resulted in a different conclusion. If the difference in coefficient of variation between the failure modes is taken into account (i.e. the uncertainties on a shear failure are larger), then the probability of a failure in bending moment becomes larger relative to the probability of failure in shear.

Before the step towards reliability-based methods can be taken for the determination of the target proof load, the required coefficients of variation that need to be used on the resistance and load effects should be determined. As shown with the sensitivity analyses in Table 8, the assumptions with regard to the coefficients of variation significantly affect the results for the resulting reliability index. Similarly, the assumptions with regard to the coefficients of variation thus influence the required target proof load. Researchers and engineers specialized in the

assessment of existing bridges should come to a consensus on the distribution functions and coefficients of variation to be used for these structures, supported by experimental evidence.

6. Summary and conclusions

Load testing of existing bridges has been part of the engineering practice since the late 19th century. In this paper, proof load testing to demonstrate sufficient capacity of bridges is studied from a reliability-based perspective. The convolution integrals for the probability of failure before, during, and after load testing are taken from the literature. These integrals are then used to find the required value of the target proof load for different safety levels, which each have different target reliability indices. These different safety levels are used in the Netherlands for the assessment of existing structures. This method was applied to the case of the viaduct De Beek, which showed that the required load to demonstrate a reliability index is significantly larger than the load prescribed by the Manual for Bridge Evaluation when the coefficients of variation recommended by the JCSS Probabilistic Model Code are used.

These observations need to be accompanied with the remark that the assumptions for the coefficient of variation in the load and resistance functions significantly influence the results. Therefore, a better definition, and a separate definition for existing bridges taking into account the shorter reference period, of the coefficients of variation is necessary to move towards reliability-based methods to determine target proof loads. Furthermore, the discussion on whether proof load testing should lead to an updated reliability index after testing, or if it is sufficient to know that a certain vehicle can pass a bridge with a margin of safety, should be held among bridge engineers.

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List of notation

d_l	effective depth to the longitudinal reinforcement
f_{cd}	design concrete compressive strength
f_{ck}	characteristic concrete compressive strength
f_r	probability density function of the resistance
f_R	allowable resistance specified in the AASHTO LRFD code [25]
f_s	probability density function of the load
f_{tm}	average tensile strength
f_{yd}	design yield strength
f_{ym}	average yield strength
g	limit state function
k	size effect factor
k_{cap}	factor which takes into account higher capacity for slabs (1.2 for slabs; 1.0 for other elements)
k_0	factor which takes into consideration how the proof load test was terminated

1	m	moment
2	m_R	bending moment resistance
3	m_S	sectional bending moment
4	s_p	proof load
5	v_{Ed}	acting shear stress
6	$v_{Rd,c}$	shear capacity
7	v_{min}	lower bound of the shear capacity
8	$\mathrm{BIAS_R}$	bias factor on the resistance
9	C	capacity
10	D	dead load
11	D_w	load from self-weight of the concrete
12	D_s	superimposed dead load
13	F_R	cumulative distribution function of the resistance
14	F_s	cumulative distribution function of the load
15	I	impact load
16	IM	dynamic factor
17	L	live load
18	L_p	maximum proof live load
19	L_r	live load on the roof
20	L_R	comparable unfactored live load due to the rating vehicle for the lanes loaded
21	L_T	target proof load
22	M_{Ed}	acting bending moment caused by factored loads
23	M_{Rd}	design value of the bending moment capacity

1	OP	operating level capacity
2	P	permanent loads other than dead loads
3	P_f	probability of failure
4	P_{fa}	probability of failure after a proof load test
5	P_{fb}	probability of failure before a proof load test
6	P_{fd}	probability of failure during a proof load test
7	$P_{pl,bending}$	target proof load for bending moment
8	$P_{pl,shear}$	target proof load for shear
9	R	resistance
10	RL	rain load
11	R_n	determined strength, nominal member resistance
12	RF	rating factor
13	RF_0	rating factor at the operating level
14	S	load
15	SL	snow load
16	TLM	test load magnitude, from ACI 437.2M-13
17	X_p	target load factor from MBE
18	X_{pA}	adjusted load factor with adjustment for site conditions
19	α	sensitivity factor
20	α_S	sensitivity factor for the load
21	α_R	sensitivity factor for the resistance
22	β	reliability index
23	β_a	reliability index after load testing

β_b reliability index before load testing

β_d reliability index during load testing

φ_c condition factor

φ_s system factor

φ LRFD resistance factor

γ_D dead load factor

γ_{DC} dead load factor of structural and non-structural components

γ_{DW} dead load factor of wearing surface and superimposed loads

γ_{LL} live load factor

γ_P load factor for permanent loads other than dead loads

ρ_l reinforcement ratio of longitudinal reinforcement

Φ Gaussian function

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Table 1 – Adjustments to X_p, Table 8.8.3.3.1-1 from MBE [23]

Consideration	Adjustment
One-Lane Load Controls	+15%
Nonredundant Structure	+10%
Fracture-Critical Details Present	+10%
Bridges in Poor Condition	+10%
In-Depth Inspection Performed	-5%
Rateable, Existing RF ≥ 1.0	-5%
ADTT ≤ 1000	-10%
ADTT ≤ 100	-15%

Table 2 - Values for the coefficient of variation and bias for resistance and load effects as used for the derivation of the target proof load factor X_p.

	COV – prior to test	COV – after test	BIAS
Resistance	10%	0%	1.12 1.0 distress in test
Dead load	10%	0%	1.0
Live load	18% truck + members 14% truck only	18% 14%	1.79 one lane 1.52 two lanes
Impact	80%	80%*	1.00

*unless a moving load test is performed to investigate the impact

Table 3 – Overview of bridges tested as part of the proof loading research in the Netherlands [36, 38]

Bridge Name	Ref	Type	Reason for test
Viaduct Medemblik	[64]	RC girder bridge	corrosion damage
Viaduct Heidijk	[65]	RC slab bridge	ASR-damage
Viaduct Vlijmen-Oost	[66]	RC slab bridge	ASR-damage
Halvemaans Bridge	[67]	RC slab bridge	bending moment rating too low
Ruytenschildt Bridge	[63, 68-	RC slab bridge	Collapse test*

	70]		
Viaduct Zijlweg	[6]	RC slab bridge	ASR-damage
Viaduct De Beek	[55]	RC slab bridge	bending moment rating too low

* the Ruytenschildt Bridge was replaced for functional reasons, from which the opportunity arose to test the bridge to failure

Table 4 – Considered safety levels governing in the Netherlands [36, 38]

Safety level	β	**Ref period**	γ_{DC}	γ_{DW}	γ_{ll}	$\alpha\beta$
Eurocode Ultimate Limit State	4.3	100 years	1.25	1.35	1.50	3.44
RBK Design	4.3	100 years	1.25	1.25	1.50	3.44
RBK Reconstruction	3.6	30 years	1.15	1.15	1.30	2.88
RBK Usage	3.3	30 years	1.15	1.15	1.25	2.64
RBK Disapproval	3.1	15 years	1.10	1.10	1.25	2.48
Eurocode Serviceability Limit State	1.5	50 years	1.00	1.00	1.00	1.20

Table 5 – Relevant recommendations for the model uncertainties from the JCSS Model Code [48]

Model type	**Distr**	**mean**	**COV**
load effect calculation			
moments in frames	LN	1.0	0.1
shear forces in frames	LN	1.0	0.1
moments in plates	LN	1.0	0.2
forces in plates	LN	1.0	0.1
stresses in 2D solids	N	0.0	0.05
stresses in 3D solids	N	0.0	0.05
resistance models concrete (static)			
bending moment capacity	LN	1.2	0.15
shear capacity	LN	1.4	0.25

Table 6 – Target proof loads for different safety levels for bending moment and shear in span 1 of viaduct De Beek.

Safety level	$P_{pl,bending}$ **(kN)**	$P_{pl,shear}$ **(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

Table 7 – Required proof load values for different safety levels

Safety Level	$\alpha\beta$	m **(kNm/m)**	P_{test} **(kN)**	β_b	β_d	β_a
RBK Disapproval	2.48	-	-	3.02	-	-
RBK Usage	2.64	-	-	3.02	-	-
RBK Reconstruction	2.88	-	-	3.02	-	-
RBK Design	3.44	655	1951	3.02	1.33	3.44

Table 8 - Results of sensitivity studies for viaduct De Beek

m_r		m_s	**Results**		
mean	**COV**	**COV**	β_b	β_d	β_a
1.2	5%	5%	∞	6.02	∞
1.2	7%	5%	∞	4.29	∞
1.2	10%	5%	7.03	3.00	∞
1.2	15%	5%	4.51	1.95	∞
1.2	5%	10%	6.90	6.02	6.66
1.2	7%	10%	6.26	4.29	6.14
1.2	10%	10%	5.14	2.98	5.64
1.2	15%	10%	4.35	1.95	5.34
1.2	5%	15%	4.63	6.02	4.81
1.2	7%	15%	4.63	4.29	4.55
1.2	10%	15%	4.24	2.98	4.23
1.2	15%	15%	3.47	1.95	3.96
1.2	5%	20%	3.50	6.02	3.75
1.2	7%	20%	3.50	4.29	3.61
1.2	10%	20%	3.50	2.98	3.43
1.2	15%	20%	3.02	1.95	3.23
1.0	5%	5%	8.21	2.37	∞
1.0	7%	5%	7.59	1.68	∞
1.0	10%	5%	5.13	1.15	∞
1.0	15%	5%	3.34	0.73	∞
1.0	5%	10%	5.02	2.37	5.16
1.0	7%	10%	4.43	1.68	5.06
1.0	10%	10%	4.12	1.15	5.02
1.0	15%	10%	3.08	0.73	5.02

1.0	5%	15%	3.61	2.37	3.64
1.0	7%	15%	3.40	1.68	3.59
1.0	10%	15%	3.15	1.15	3.57
1.0	15%	15%	2.65	0.73	3.59
1.0	5%	20%	2.82	2.37	2.84
1.0	7%	20%	2.74	1.68	2.82
1.0	10%	20%	2.59	1.15	2.81
1.0	15%	20%	2.29	0.73	2.85

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Fig. 1: Sketch of the probability density functions of the load and resistance, with the probability of failure at the intersection these functions: (a) before load testing; (b) during load testing; (c) after load testing, based on [43, 44]

Fig. 2: Flowchart of updating the functions after proof load testing.

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Fig. 6: Loading protocol for viaduct De Beek, span 1: (a) bending moment test, (b) shear test.

Fig. 7: Distribution functions of resistance moment and sectional moment for viaduct De Beek: (a) probability density functions; (b) cumulative distribution functions.

Fig. 8: Effect of load testing on probability density function of m_R, resulting in $f_R^*(x)$.

Fig. 9: Effect of coefficient of variation of resistance on functions before and after a proof load test: (a) COV_R = 5%; (b) COV_R = 25%.