Pilot Proof-Load Test on Viaduct De Beek: Case Study

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1 **Case study: Pilot proof load test on viaduct De Beek**
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5
6 **Abstract**

7 For existing bridges, proof load testing can be a suitable assessment method. This paper

8 addresses the evaluation of a posted reinforced concrete slab bridge over the highway through

9 proof load testing, detailing the preparation, execution and analysis of the test. As the target

10 proof load and the required measurements for proof load testing currently are not well-defined in

11 the existing codes, this pilot case is used to develop and evaluate proposed recommendations for

12 proof load testing for a future guideline on proof load testing for the Netherlands. Moreover, the

13 pilot proof load test is used to study the feasibility of proof load testing for both shear and

14 flexure.

15

16 **CE database subject headings**

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

18 strength

19

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20 **Introduction**

21   Load testing is the non-destructive field testing of bridges (Cochet et al. 2004; Frýba and

22 Pirner 2001; NRA 2014). Two types of load testing can be distinguished. Diagnostic load testing

23 (Ataei et al. 2016; Bentz and Hoult 2016; Farhey 2005; Fu et al. 1997; Gokce et al. 2011;

24 Halding et al. 2017; Jauregui et al. 2010; Kim et al. 2009; Maguire et al. 2015; Matta et al. 2008;

25 Moen et al. 2013; Murià-Vila et al. 2015; Nguyen et al. 2016; Ohanian et al. 2017; Olaszek et al.

26 2014; Russo et al. 2000; Sanayei et al. 2012; Sanayei et al. 2016; Stroh et al. 2010; Velázquez et

27 al. 2000) uses lower load levels, and is used to verify assumptions made in analytical models. In

28 practice, these models are often linear elastic, three-dimensional finite element models (Bell and

29 Sipple 2009; Bridge Diagnostics Inc. 2012; Hernandez and Myers 2015). The structural response

30 in the analytical model can be compared to the structural response measured in the field, and the

31 analytical model and the resulting rating can be updated accordingly. Proof load testing uses

32 higher load levels. In a proof load test (Aguilar et al. 2015; Anay et al. 2016; Arangjelovski et al.

33 2015; Cai and Shahawy 2003; Casas and Gómez 2013; Faber et al. 2000; Fu and Tang 1995;

34 Moses et al. 1994; Olaszek et al. 2012; Olaszek et al. 2016; Saraf et al. 1996; Spaethe 1994;

35 Zwicky and Brühwiler 2015), a load is applied that demonstrates that the bridge can carry the

36 loads prescribed by the code satisfactorily, or that higher or lower load levels can be carried by

37 the bridge. Whether the bridge behavior is satisfactory is typically expressed based on

38 "acceptance criteria" or "stop criteria". These criteria, based on, among others, deflections, crack

39 widths and strains, identify the acceptable limits of the bridge's structural response. If these

40 limits are exceeded during a proof load test, and higher loads are applied, there is a risk for

41 irreversible damage to the structure. If a stop criterion is exceeded, further loading is not

42 permitted. The conclusion of the proof load test is then that the bridge satisfies a lower load level

43   (i.e. the last load level that was achieved prior to exceedance of a stop criterion) than the target

44   load level. Alternatively, when the target load level is achieved, but no stop criterion has been

45   exceeded yet, further loading can be used to demonstrate a larger load level.

46   Diagnostic load testing can be used to determine the transverse flexural distribution (He

47   et al. 2012), to determine the stiffness of a structure (Barker 2001; Zhang et al. 2011), and to

48   verify if a design or repair intervention is functioning appropriately (Nilimaa et al. 2015; Puurula

49   et al. 2015; Shifferaw and Fanous 2013). For structures with limited uncertainties, such as steel

50   bridges or concrete girder bridges, diagnostic load testing is recommended. Strain gages can be

51   placed over the girder height to determine the position of the neutral axis. The differences in

52   structural response in the analytical model and the response measured in the field can be

53   attributed to different contributions, such as the actual impact factor, the actual dimensions, the

54   unaccounted stiffness of elements such as curbs and railing, the actual lateral live load

55   distribution, the bearing restraint effect, and unintended composite action (Barker 2001). For

56   bridges with large uncertainties, on the other hand, proof load testing is necessary. These large

57   uncertainties can include the effect of material degradation on the structure's response

58   (Koekkoek et al. 2015a), the geometry and reinforcement layout for bridges without plans

59   (Aguilar et al. 2015; Anay et al. 2016; Shenton et al. 2007), or the load path at higher load levels

60   (Taylor et al. 2007). For bridge types such as reinforced concrete slab bridges (Saraf 1998),

61   placing strain gages over the height is more complicated, and measurements can only be taken

62   from the bottom of the slab, from the side faces, and, provided that it does not obstruct the

63   loading process and that no wearing surface covering the concrete cross-section is present, from

64   the top faces. This paper deals with a case study of proof load testing of a reinforced concrete

65 slab bridge for both flexure and shear, and how the results of this case study can be used to

66 develop and evaluate recommendations for proof load testing.

67 **Proof load testing**

68 *Current standards and guideline*

69 Existing codes for load testing of bridges focus on diagnostic load testing. Examples are

70 the French guidelines (Cochet et al. 2004), the Irish guidelines (NRA 2014) and the British

71 guidelines (The Institution of Civil Engineers - National Steering Committee for the Load

72 Testing of Bridges 1998). Similar procedures are followed in Italy (Veneziano et al. 1984),

73 Switzerland (Brühwiler et al. 2012), and the Czech Republic and Slovakia (Frýba and Pirner

74 2001). The Manual for Bridge Rating through Load Testing (NCHRP 1998) and the Manual for

75 Bridge Evaluation (AASHTO 2016) deal with diagnostic load testing and proof load testing.

76 These manuals do not qualitatively describe stop criteria for proof load testing, but mention that

77 the test should be terminated when the bridge exhibits the onset of non-linear behavior or other

78 visible signs of distress. None of the existing codes for proof load testing allow for the testing of

79 non-ductile failure modes, such as shear in concrete bridges.

80 For proof load testing of concrete structures, building codes are available. The German

81 guidelines (Deutscher Ausschuss für Stahlbeton 2000) are originally developed for reinforced

82 and plain concrete buildings, but are also applied to concrete bridges (Schacht et al. 2016b). For

83 buildings, ACI 437.2M-13 (ACI Committee 437 2013), prescribing a slightly different required

84 proof load than ACI 318-14 (ACI Committee 318 2014), is available. Since these codes are

85 specialized for concrete structures (and buildings in particular), they contain detailed stop criteria

86 (nomenclature used in the German guidelines) or acceptance criteria (nomenclature used in ACI

87 437.2M-13 (ACI Committee 437 2013)). The stop criteria are only valid for flexure-critical

88 positions, and proof load testing for shear is not permitted. Testing for shear is a current topic of

89 research (Schacht et al. 2016a).

90 *Goals of proof load testing and examples*

91   The main goal of a proof load test is to demonstrate experimentally that a bridge can

92 withstand the factored live loads given in the code. As such, a proof load test does not give an

93 estimate of the ultimate capacity of a bridge; only a lower bound of the capacity: the capacity is

94 known to be larger than the load effect induced by the proof load. However, because of the high

95 load levels involved in proof load tests, the risks for structural damage is larger. Adequate

96 preparation to guarantee the structural safety of the bridge and the safety of the personnel is thus

97 important (Cai and Shahawy 2003).

98   Some states and countries have developed special vehicles for proof loading. Examples

99 of these vehicles include the two proof loading vehicles of Florida that can be loaded with ballast

100 blocks (90 tons maximum each) (Shahawy 1995), and the BELFA ("Belastungsfahrzeug",

101 German for loading vehicle) from Germany (ifem 2013), which can apply a maximum load of

102 150 tons.

103   In the state of New Mexico, a large number of bridges without plans exist (Aguilar et al.

104 2015), for which a rating method based on diagnostic and proof load tests, combined with other

105 non-destructive testing techniques has been developed. Similar testing has also been carried out

106 in New York state (Hag-Elsafi and Kunin 2006), in Delaware (Shenton et al. 2007), and on

107 bridges owned by the US Army (Varela-Ortiz et al. 2010), which are subjected to different live

108 loads (military vehicles).

109   Another type of uncertainty that can require proof load tests, is uncertainty related to the

110 effect of material deterioration and degradation on the structural performance of existing bridges.

111   An example is the proof load testing of a deteriorated bridge in Michigan (Juntunen and Isola

112   1995), where a proof load test with an 82-ton two-unit vehicle successfully showed that the load

113   restriction of 45 tons did not need to be reduced because of the extensive deterioration in the

114   bridge. A later analysis, however, showed that in the proof load test, composite action between

115   the old beams and the newly applied overlay had occurred. This composite action is lost over

116   time, but was still sufficient for the structure to keep the 45 ton two-unit vehicle limit.

117   ***Previous proof load tests in the Netherlands***

118   In the Netherlands, a large number of reinforced concrete slab bridges were built in the

119   decades following the Second World War (Lantsoght et al. 2013b). These bridges are reaching

120   the end of their originally devised service life. To assess these structures, and to investigate their

121   structural safety under the current live loads that are larger than those at the time of their design,

122   an assessment is necessary. In Europe, no separate live loads models are defined for the

123   assessment of existing bridges. Therefore, all assessment, including assessment through proof

124   load testing, needs to be carried out based on the live load model which consists of design

125   tandems and distributed lane loads. In North American practice, the target proof load can be

126   calculated as a multiple (reference value = 1.4) of the truck used for assessment. In Europe, the

127   target proof load needs to represent the full live load model.

128   During the last decade, a number of proof load tests on reinforced concrete slab bridges

129   have been carried out in the Netherlands. An overview of the program of pilot proof load tests

130   can be found elsewhere (Lantsoght et al. in press). In this paragraph, only the main reasons for

131   selecting the pilot cases, and main conclusions from the load tests are given. The first test was

132   carried out on the viaduct Heidijk (Dieteren and den Uijl 2009), to see if this bridge with material

133   degradation caused by alkali-silica reaction can carry a truck of 30 ton on a shear-critical

134 position. The load was applied through a loading frame and hydraulic jacks with a hand pump. It

135 was found that the 30 ton truck can be successfully carried. A second test was on the viaduct

136 Vlijmen-Oost (Koekkoek et al. 2015b), also affected by alkali-silica reaction. The BELFA

137 vehicle (Bretschneider et al. 2012) was used on a shear-critical position and on a critical position

138 for bending moment. It was concluded that the bridge fulfills the current code requirements. In a

139 next test, an existing slab bridge with insufficient flexural capacity according to the assessment

140 calculations was tested: the Halvemaans Bridge (Fennis and Hordijk 2014). This test was the

141 first test in which the load was applied by using a load spreader beam and hydraulic jacks.

142 Again, the load test was used to show that the bridge fulfills the requirements. In the summer of

143 2014, the Ruytenschildt bridge was tested to failure (Lantsoght et al. 2016a; Lantsoght et al.

144 2016b; Lantsoght et al. 2016c; Lantsoght et al. available online ahead of print) in two spans. The

145 last proof load test on a bridge with damage caused by alkali-silica reaction, the viaduct Zijlweg,

146 studied a shear- and flexure-critical position in the first span (Koekkoek et al. 2015a; Lantsoght

147 et al. in review). Upon assessment, it was found that the viaduct Zijlweg does not fulfill the

148 requirements of the code for shear. Through the proof load test, it could be shown that the

149 viaduct can carry the factored live loads of the code without signs of distress, and that it fulfills

150 the requirements for shear and bending moment. It should be emphasized that proof load testing

151 for shear is uncommon and typically not permitted, and that none of the existing codes or

152 guidelines prescribes stop criteria for shear.

153

154 **Description of viaduct De Beek**

155 *Restrictions on viaduct De Beek*

156   Viaduct De Beek, a reinforced concrete slab bridge, see Fig. 1a, lies in a local road, the

157 Beekstraat, over highway A67 close to Ommel in the province of Noord Brabant in the

158 Netherlands. The bridge was built in 1963 and is owned and managed by the Dutch Ministry of

159 Infrastructure and the Environment. An inspection and assessment for the current live loads in

160 2015 (Willems et al. 2015) led to the conclusion that the capacity of the viaduct is insufficient for

161 two lanes of unrestricted traffic. The assessment calculations (Iv-Infra 2015) determined that the

162 flexural capacity in the longitudinal and transverse direction is insufficient in all spans.

163 Originally, load posting was proposed, but for practical reasons it was decided to restrict traffic

164 to one lane by using barriers, see Fig. 1b. During the inspection of 2015, structural damage (wide

165 cracking) was observed at the bottom of the concrete deck, compromising the durability of the

166 structure.

167 *Geometry of viaduct De Beek*

168   The geometry of viaduct De Beek can be seen in Fig. 2. The viaduct has four spans, with

169 end spans of 10.81 m and central spans of 15.40 m. The width of the viaduct is 9.94 m, with a

170 carriageway width of 7.44 m, originally designed to carry one lane of traffic of 3.5 m wide in

171 each direction. The viaduct has a height that varies parabolically between 470 mm and 870 mm.

172 In the width direction, a curb with a height of 200 mm is available at the edge. The layer of

173 asphalt is measured to be between 50 mm and 75 mm.

Material properties of viaduct De Beek

175   Nine cores were drilled from the slab to determine the concrete properties. The

176 characteristic concrete compressive strength f_{ck} equals 44.5 MPa and the concrete tensile splitting

177 strength f_{ctm} = 4.4 MPa. The design concrete compressive strength is thus f_{cd} = 30 MPa.

178   Three samples of the steel were taken, from which it was concluded that steel QR 24 was

179 used. QR 22 and QR 24 are types of plain reinforcement that were used in the Netherlands

180 during the 1950s and 1960s. The measured average yield strength f_{ym} = 291 MPa and the tensile

181 strength f_{tm} = 420 MPa. The design yield strength can be taking as f_{yd} = 252 MPa. The

182 reinforcement drawing is given in Fig. 3. The main flexural reinforcement in the longitudinal

183 direction in span 1 consists of 6 layers of ϕ 25 mm with a 560 mm spacing, so that the

184 reinforcement is A_s = 5259 mm^2/m.

185 **Determination of target proof load**

186 *Practical application of the target proof load*

187   As mentioned previously, the live load model that is used for assessment of existing

188 bridges in Europe does not allow for a direct translation to a certain type of truck, unlike in North

189 America. Whereas in North America heavy dump trucks, special vehicles, and/or military

190 vehicles can be used for proof load tests, in Europe only the BELFA vehicle from Germany

191 (Bretschneider et al. 2012) is available with a maximum load of 150 metric ton. Regular vehicles

192 are not suitable. Other options for applying the target proof load in Europe include directly

193 applying dead weights on the deck (Olaszek et al. 2014), or by using an external structure

194 (Schwesinger and Bolle 2000).

Target proof load in North America

196   According to the Manual for Bridge Rating through Load Testing (NCHRP 1998) and the

197   Manual for Bridge Evaluation (AASHTO 2016), the target proof load is based on the load L_R of

198   the vehicle used for load rating at the legal load level, multiplied with a factor X_p and taking into

199   account the impact allowance *I*. The standard value of X_p equals 1.4. This value is adjusted as

200   follows:

201   • X_p needs to be increased by 15% if one lane load controls the response.

202   • For spans with fracture-critical details, X_p shall be increased by 10%.

203   • If routine inspections are performed less than every 2 years, X_p should be increased by

204   10%.

205   • If the structure is ratable, i.e. has no hidden details, X_p can be reduced by 5%.

206   • Additional factors including traffic intensity and bridge condition may also be

207   incorporated in the selection of the live load factor X_p.

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

209   follows:

210
$$X_{pA} = X_p \left(1 + \frac{\Sigma\%}{100} \right)$$
(1)

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

212
$$L_T = X_{pA} L_R (1 + I)$$
(2)

213   with $1.3 \le X_{pA} \le 2.2$.

214

215   *Application to Eurocode live loads and Dutch safety levels*

216   It has been suggested for Europe to use WIM data to determine the target proof load

217   (Casas and Gómez 2013), but these data are not available for most bridges. In the Netherlands,

218   different safety levels, associated with different reliability indices are defined for existing

219   structures in the national code NEN 8700:2011 (Code Committee 351001 2011) and the

220   Guidelines Assessment Bridges (Richtlijn Beoordeling Kunstwerken = "RBK") (Rijkswaterstaat

221   2013). An overview of these different levels is given in Table 1, together with the ultimate limit

222   state and the serviceability limit state from the Eurocode for design of new structures (CEN

223   2002). These different safety levels correspond to different load factors. The load factors that are

224   used to determine the proof load are given in Table 2. Note that here the load factor of the self-

225   weight, $\gamma_{sw} = 1.10$ for all safety levels (except the serviceability limit state). The reason why a

226   lower load factor for the self-weight is used is that, because the calculations involve an existing

227   structure, the dimensions of the structure are not a random variable anymore, but can be

228   considered deterministic (i.e., the actual dimensions of the structure). Only the model factor

229   remains, which equals 1.07 in NEN-EN 1992-2+C1:2011 (CEN 2011). This value is rounded off

230   to 1.10. The target proof load to approve the structure is calculated for each safety level.

231   According to the RBK (Rijkswaterstaat 2013), the recommended safety level for the assessment

232   of existing bridges is the RBK Usage level. For the pilot proof load test, higher loads have been

233   applied to study the behavior of the bridge under all safety levels.

234   The proof load needs to be equivalent to the loads from Load Model 1 of NEN-EN 1991-

235   2:2003 (CEN 2003), which consists of a design tandem in each lane and a distributed lane load.

236   The position of the proof load is determined as the most critical position for bending moment and

237   the most critical position for shear. The proof load is applied as a single proof load tandem, of

238   which the load magnitude needs to represent the design tandem in both lanes, and the distributed

239   lane loads.

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

241   On viaduct De Beek, the proof load test was carried out in span 1. The critical span for

242 the assessment, and the span with the largest cracking damage, is span 2. However, span 2 is

243 over the highway. Testing span 2 would require the closing of the highway for safety reasons,

244 which is practically impossible. Therefore, span 1 is tested, and the results are then interpreted in

245 the light of the assessment of span 2. As currently no methods are available to extrapolate results

246 from a load test on one span to another span, an assessment of span 2 based on plastic

247 redistribution will be presented later in this paper. Both a flexure- and shear-critical position are

248 tested.

249   The following procedure is used to determine the required magnitude and position of the

250 proof load for bending moment:

251   1. A linear finite element model of the bridge is developed. The loads that need to be

252   considered are the self-weight of the concrete, the weight of the asphalt layer, and the

253   live loads from Load Model 1 from NEN-EN 1991-2:2003 (CEN 2003).

254   2. The design tandems from Load Model 1 are moved in their respective lanes until the

255   position of the tandems that causes the largest bending moment, distributed over 3 m

256   in the transverse direction, is found. The corresponding position of the design tandem

257   in the first lane is the critical position of the proof load tandem.

258   3. The live loads from Load Model 1 are removed and replaced by the proof load

259   tandem at the critical position. The load on the proof load tandem is now increased

260   until the same bending moment (distributed over 3 m transversely) is found as for the

261   bridge subjected to the live loads from Load Model 1 at the critical position.

262   For viaduct De Beek the critical position is found at 3.55 m from the end support. This position

263   (shown as position "A") is sketched in Fig. 4. The required values of the proof load at the

264   different safety levels are then given in Table 3.

265   A similar procedure is used for the shear-critical position. The main difference is that the

266   critical position is predetermined as $2.5d_l$ for the face-to-face distance between the load and the

267   support (Lantsoght et al. 2013b). The distribution width in the transverse direction for the peak

268   shear stress is taken as $4d_l$ per wheel load (Lantsoght et al. 2013a). For viaduct De Beek, the

269   critical position for shear is at 1.1 m from the end support. The position of the proof load tandem

270   for the shear test is shown as position "B" in Fig. 4. An overview of the required values of the

271   proof load at the different safety levels is given in Table 3.

272   ***Resulting loading protocol***

273   The load is applied with four hydraulic jacks and a load spreader beam, see Fig. 5, so that

274   if a large deflection occurs, the load is removed from the bridge. The simulated tire contact area

275   (steel loading plate) is 230 mm × 300 mm. The loading speed was determined as 5.4 kN/s in the

276   bending moment test, and as 7.3 kN/s in the shear test. A cyclic loading protocol was chosen, as

277   it allows for checking the stop criteria after each cycle, and linearity. In the bending moment test,

278   the following loading steps, referring to the load levels from Table 1 and Table 2, see Fig. 6a,

279   were used:

280   1. A low load level of 550 kN to check the functioning of all sensors.

281   2. A load level of 950 kN, which is slightly lower than the serviceability limit state.

282   3. A load level of 1350 kN, which corresponds with the RBK Usage level

283   (Rijkswaterstaat 2013).

284   4. A maximum load of 1699 kN, which corresponds with the Eurocode Ultimate

285   Limit State level.

286   The applied maximum load at the jacks was 1699 kN. Adding the weight of the setup, results in

287   the maximum total applied load of 1751 kN, which is 6% above the calculated Eurocode

288   Ultimate Limit State level. The additional percentage takes into account local material

289   variability, and can be considered as a model factor for a proof load test.

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

291   1. A low load level of 250 kN to check the functioning of all sensors.

292   2. A load level of 750 kN, which is slightly lower than the serviceability limit state.

293   3. A load level of 1250 kN, which corresponds with the RBK Usage level

294   (Rijkswaterstaat 2013).

295   4. A maximum load of 1508 kN, which corresponds with the Eurocode Ultimate

296   Limit State level.

297   The maximum applied load, including the weight of the setup, was then 1560 kN, or the

298   calculated Eurocode ultimate limit state + 2%.

299

300   **Determination of required measurements and stop criteria**

301   *Current practice*

302   As mentioned earlier, the only codes and guidelines that contain stop criteria for concrete

303   structures (originally developed for concrete buildings) are ACI 437.2M-13 (ACI Committee 437

304   2013) and the German guideline (Deutscher Ausschuss für Stahlbeton 2000), and these stop

305   criteria are only valid for flexure. In ACI 437.2M-13 (ACI Committee 437 2013), the stop

306   criteria depend on the loading protocol, which can be monotonic or cyclic. As the loading

307   protocol for viaduct De Beek is cyclic, the focus here will be on the cyclic loading protocol. The

308   cyclic loading protocol of ACI 437.2M-13 consists of three load levels with two cycles per load

309   level. The first load level is the serviceability load level, and the final load level corresponds to

310   the target proof load. In ACI 437.2M-13, the stop criteria are defined as acceptance criteria –

311   criteria that need to be fulfilled for the acceptance of the structure after the proof load test. The

312   first acceptance criterion is that the structure should show no evidence of failure. The second

313   acceptance criterion is called the deviation from linearity index, I_{DL}, derived from the load-

314   displacement diagram. The angles α are determined based on the origin of the load-displacement

315   diagram and the maximum point in a load cycle. The acceptance criterion for the deviation from

316   linearity index is determined as:

317
$$I_{DL} = 1 - \frac{\tan(\alpha_i)}{\tan(\alpha_{ref})} \leq 0.25 \tag{3}$$

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

319
$$I_{pr} = \frac{I_{p(i+1)}}{I_{pi}} \leq 0.5 \tag{4}$$

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

321
$$I_{pi} = \frac{\Delta_r^i}{\Delta_{max}^i} \tag{5}$$

322
$$I_{p(i+1)} = \frac{\Delta_r^{(i+1)}}{\Delta_{max}^{(i+1)}} \tag{6}$$

323   The final acceptance criterion is related to the residual deflection Δ_r, measured at least 24 hours

324   after removal of the load. This value has to be smaller than or equal to 25% of the maximum

325   deflection or 1/180 of the span length.

326   The second set of stop criteria comes from the German guideline for load testing

327   (Deutscher Ausschuss für Stahlbeton 2000). This guideline uses a cyclic loading protocol of

328   three load levels with at least one cycle per level. The first stop criterion is based on the

329   measured strains in the concrete, ε_c:

330    $$\varepsilon_c < \varepsilon_{c,\lim} - \varepsilon_{c0} \tag{7}$$

331   The limiting strain $\varepsilon_{c,lim}$ is 0.8 ‰ if the concrete compressive strength is larger than 25 MPa,

332   minus the strain ε_{c0} caused by the permanent loads. The second stop criterion is based on the

333   measured strains in the steel reinforcement, ε_{s2}, which requires removal of the concrete cover:

334    $$\varepsilon_{s2} < 0.7 \frac{f_{ym}}{E_s} - \varepsilon_{s02} \tag{8}$$

335   The third stop criterion evaluates the crack width w for new cracks and the increases in crack

336   width Δw for existing cracks. New cracks can be maximum 0.5 mm, of which 30% is permitted

337   as residual crack width, and existing cracks can increase with maximum 0.3 mm, of which 20%

338   is permitted as residual crack width. The fourth stop criterion says that nonlinear behaviour

339   should not take place, and that the residual deformation is limited to 10% of the maximum

340   deformation.

341   ***Sensor plan for viaduct De Beek***

342   Since the proof load test on viaduct De Beek was a pilot test and part of a program of

343   proof load tests, the viaduct was heavily instrumented, so that the behavior of the viaduct could

344   be closely monitored during the experiment. Another goal was to analyze the measurements after

345   the test in order to come up with recommendations for proof loading of reinforced concrete slab

346   bridges and to evaluate the existing stop criteria for flexure. The following responses of the

347   bridge were measured:

348   1. The vertical deflections of the deck at different positions in the longitudinal and

349   transverse direction are measured with linear variable differential transformers

350   (LVDTs) and laser triangulation sensors.

351   2. The vertical deflections of the support beam are measured with LVDTs.

352   3. The strain in the reinforcement steel is measured at a few locations where the

353   concrete cover is removed, and strain gages are applied to the steel.

354   4. The strain in the concrete is measured at the bottom surface by applying LVDTs over

355   1 m.

356   5. The opening of existing cracks is followed by applying an LVDT over the crack.

357   6. The applied load is measured with load cells at the four wheel print positions of the

358   proof load tandem.

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

360   ***Measurements of viaduct de Beek***

361   Some interesting measurements and post-processing results of the bending moment test

362   are shown in Fig. 8. The first result that is studied is the load-deflection diagram, of which the

363   envelope is given in Fig. 8a. The maximum deflection during the proof load test was 11 mm.

364   From the results of the load-deflection diagram, the reduction of the slope over the applied load

365   cycles can be studied, see Fig. 8c. A 25% reduction of the slope is indicated in Fig. 8c with a red

366   line. It can be seen that during none of the load cycles this limit, which was proposed as a

367   possible stop criterion based on beam tests in the laboratory (Lantsoght et al. (in press)), is

368   exceeded.

369   Another element of post-processing is the determination of the deflection profiles in the

370   longitudinal and transverse directions. The longitudinal deflection profile is given in Fig. 8d,

371   from which it can be observed that the increases in deflection increase linearly with the load. The

372   supporting calculations can be found in the background report (Koekkoek et al. 2016).

373　　　　The measurements of the deflections and strains can be compared to the results of the

374　　linear finite element program. This comparison indicated that the stiffness of uncracked concrete,

375　　32.9 GPa can be used for the finite element model. However, it must be noted that in the

376　　simplified finite element model possible additional sources of stiffness (Barker 2001), such as

377　　the effect of curbs and railings and the bearing restraint stiffness of aged bearings, were not taken

378　　into account. The strain measurements showed good correspondence between the steel and

379　　concrete strains. The calculated strains also corresponded reasonably well with the measured

380　　strains, see Fig. 8b.

381　　　　For the shear position test, the most important measurements and post-processing results

382　　are shown in Fig. 9. The first result that is studied is the load-deflection diagram, of which the

383　　envelope is given in Fig. 9a. The maximum deflection during the proof load test was 7 mm. The

384　　reduction of the slope over the applied load cycles is shown in Fig. 9c. During none of the load

385　　cycles the limit of maximum 25% reduction of the slope is exceeded. The longitudinal deflection

386　　profile is given in Fig. 9d, from which it can be seen that under the applied loads the behavior

387　　was linear.

388　　　　The measurements of the deflections and strains can be compared to the results of the

389　　linear finite element program. From the deflection results, it was concluded again that a stiffness

390　　of uncracked concrete, 32.9 GPa can be used, see Fig. 9b.

391　　*Evaluation of stop criteria*

392　　　　In this section, the existing stop criteria that are developed for buildings for flexure are

393　　evaluated. The residual deformation after the test was determined. In the bending moment test

394　　the ratio of the residual to maximum deflection was 15%, which does not fulfil the stop criterion

395　　of the German guideline but fulfills the acceptance criterion of ACI 437.2M-13. In the shear test

396   the ratio of the residual to maximum deflection was 8%, which is below the limit of the German

397   guideline and ACI 437.M-13.

398   The stop criteria for the strains from Eq. (7) and (8) must be verified. The strain caused

399   by the self-weight of the concrete and the layer of asphalt is ε_{c0} = 163 µɛ. The limiting strain $\varepsilon_{c,lim}$

400   = 800 µɛ, so that the measured strain should be smaller than 637 µɛ. This maximum is exceeded

401   in the experiment, in the loading step leading up to the target load level, as can be seen in Fig.

402   8b. The stop criterion was exceeded at 97% of the target load. Loading to a higher load level than

403   the target load level could have resulted in permanent damage to the structure. The limiting steel

404   strain leads to a maximum strain of 857 µɛ, which is not exceeded during the experiment. The

405   stop criteria with regard to concrete and steel strains are not exceeded during the shear

406   experiment. This observation is not surprising, since the shear position activates less flexural

407   response.

408   The maximum measured opening of an existing crack during the bending test was 0.12

409   mm, after which the residual crack width was 0.03 mm. It is assumed that crack widths smaller

410   than 0.05 mm can be neglected. The conclusion is then that the studied crack fully closed after

411   the maximum load, and that no permanent damage was inflicted on the structure by the proof

412   load test. The maximum measured opening of an existing crack was 0.11 mm during the shear

413   test, after which the residual crack width was 0.01 mm. The studied crack fully closed after the

414   maximum load.

415   **Assessment of viaduct De Beek**

416   *Assessment of the tested span*

417   All assessments for viaduct De Beek are carried out based on the original two lanes of

418   traffic, to see if the current traffic restrictions (Fig. 1b) can be removed. All acting bending

419   moments m_{Ed} are determined based on a transverse distribution of 3 m. With the reinforcement

420   from Fig. 3, the moment capacity in span 1 is determined as m_{Rd} = 579 kNm/m. The factored

421   acting moment in the cross-section with the load factors of the RBK Usage level, which is used

422   for the assessment of existing highway bridges (Rijkswaterstaat 2013) is m_{Ed} = 463 kNm/m. As a

423   result, the Unity Check for bending moment equals UC = 0.8. The Unity Check is determined as

424   the ratio of the load effect over the capacity. This result does not correspond with the 2015

425   assessment of the bridge (Iv-Infra 2015), which resulted in the lane restrictions applied to the

426   bridge. The 2015 assessment combined a calculation of the UCs based on a linear finite element

427   model with a visual inspection. A comparison showed that the 2015 assessment did not consider

428   all reinforcement as shown in Fig. 3. Moreover, the proof load test showed that the viaduct can

429   carry the factored live loads of the Eurocode Ultimate Limit State.

430   Using the rating factor from the Manual for Bridge Evaluation (AASHTO 2016) resulted

431   in RF = 1.32 > 1, so that the first span fulfills the requirements.

432   The shear capacity according to the RBK (Rijkswaterstaat 2013) was $v_{Rd,c}$ = 1.002 MPa.

433   For the RBK Usage level, the acting shear stress is v_{Ed} = 0.482 MPa when using averaging over a

434   distance of $4d_l$ (Lantsoght et al. 2013a), so that UC = 0.48. The first span thus fulfills the

435   requirements for shear, prior to taking into account the information from the proof load test.

436   *Assessment of span 2*

437   According to the reinforcement drawings, Fig. 3, less reinforcement is present in span 2

438   as compared to span 1 (4 layers of ϕ 25 mm bars with a spacing of 560 mm as compared to 6

439   layers of ϕ 25 mm bars with a spacing of 560 mm), while span 2 has a larger span length. The

440   moment capacity now is m_{Rd} = 335 kNm/m for the cross-section at the midspan. The bending

441   moment caused by the factored loads acting on this cross-section is m_{Ed} = 422 kNm/m, so that

442   UC = 1.26, which means that the cross-section does not fulfill the requirements for bending

443   moments under the RBK Usage loads (Rijkswaterstaat 2013). A further analysis of the cross-

444   section is thus necessary.

445   In a next step, the analysis is carried out with plastic redistribution. In this case, the Unity

446   Check for the hogging moment over support 2 is considered. The ultimate moment capacity at

447   support 2 equals m_{Rd} = 1022 kNm/m. Using plastic redistribution means that a plastic hinge will

448   form in the midspan cross-section when a moment of 335 kNm/m is achieved in this cross-

449   section. If higher loads are applied, redistribution of the moment diagram will occur, and higher

450   sectional moments will occur over the support. The moment m_{Ed} = 335 kNm/m is reached in the

451   midspan cross-section at 78% of the full factored RBK Usage loads. The moment at support 2 is

452   then m_{Ed} = 900 kNm/m. The midspan of the slab is now modeled as a plastic hinge over the full

453   width of the slab. With this model, the acting bending moments under the factored RBK Usage

454   live loads (Rijkswaterstaat 2013) are m_{Ed} = 960 kNm/m at support 2 and m_{Ed} = 335 kNm/m at

455   midspan. The amount of plastic redistribution that has taken place is 6.7%. With plastic

456   redistribution, UC = 0.94 over support 2 and UC = 1 at midspan. These results indicate that a

457   direct assessment of span 2 based on the test results does not lead to a recommendation for the

458   removal of the traffic restrictions. Only when plastic redistribution is allowed to take place, and

459   cracking and the reduction of the durability of the structure are acceptable by the owner, the

460   traffic restrictions can be removed.

461   The assessment for shear (Iv-Infra 2015) gave UC = 0.51 for the cross-section close to

462   the intermediate support in span 2. The second span thus fulfills the requirements for shear.

-21-

463 **Recommendations**

464 *Viaduct de Beek*

465   Based on the presented analyses, it was recommended to check the reinforcement in span

466 2 with a scanner or by removing the concrete cover locally to verify the spacing between bars.

467 The reinforcement layout presented in the plans is unexpected, since the longer middle spans are

468 provided with less reinforcement. The acting bending moment for the RBK Usage level in span 1

469 is 463 kNm/m and in span 2 422 kNm/m. The reduction of the span moment due to the support

470 moment is thus rather limited in the second span. It is also recommended to carry out an

471 additional inspection of the cracks in span 2, and to carefully check for signs of corrosion, which

472 would further reduce the flexural capacity. If the condition of span 2 is considered satisfactory in

473 terms of present corrosion, the current traffic restriction can be removed, provided that plastic

474 redistribution is allowed.

475 *Lessons learned for proof load testing*

476   The pilot proof load test shows that proof load testing can be carried out at flexure- and

477 shear-critical positions. The determination of the target proof load is currently carried out based

478 on equivalent sectional moments and shears. The presented method which uses a single proof

479 load tandem is valid for bridges of small width.

480   The analysis of the stop criteria shows that the concrete strain criterion of the German

481 guideline is suitable for the combination with proof load tests for flexure and shear. The criterion

482 for the steel strains cannot always be used, as not all bridge owners allow for the removal of the

483 concrete cover. The crack width criterion is useful, provided that cracks of less than 0.05 mm are

484 neglected. The residual deflection of 10% is rather conservative; the value of 25% from ACI

485 437.2M-13 could be more suitable. The other stop criteria from ACI 437.2M-13 could not be

486   evaluated, as these are directly associated with the loading protocol of ACI 437.2M-13, which

487   was not the same as the loading protocol used for viaduct De Beek. Stop criteria to evaluate

488   possible shear failure still need to be developed.

489   **Summary and Conclusions**

490   The viaduct De Beek is a reinforced concrete slab bridge with a traffic restriction that

491   reduces the use of the viaduct from one lane in each direction to a single lane, as the bending

492   moment capacity was found to be insufficient for the prescribed loads. The bridge was evaluated

493   in a pilot proof load test, which also served to study if proof load testing for shear is possible,

494   and if the existing stop criteria derived for buildings can be used in proof load tests for bridges.

495   As the stop criteria are a topic of research, a large number of sensors were applied on the viaduct

496   to closely monitor the structural response during the test.

497   A proof load test was carried out at a flexure- and shear critical position in the first span

498   of the viaduct. For both tests, the target proof load was achieved. The analysis of the

499   measurements showed that the structural response remained sufficiently close to the linear

500   behavior. However, some stop criteria from the German guideline were exceeded, which

501   indicates that further loading of the structure could have resulted in permanent damage to the

502   structure. Further research should focus on the development of stop criteria for shear.

503   The assessment with the Unity Checks showed that the capacity of span 1 is sufficient,

504   and was proven to be sufficient in the proof load tests, but the capacity of span 2 cannot directly

505   be proven to be sufficient. In an additional analysis, plastic redistribution was allowed. It was

506   found that if 6.7% of plastic redistribution is allowed to take place, the Unity Checks at the

507   support and in the midspan cross-section of span 2 can fulfill the requirements, provided that a

508   reduction of the durability is accepted.

509   **Notation List**

510   The following symbols are used in this paper:

511   d_l effective depth to the longitudinal reinforcement

512   f_{cd} design concrete compressive strength

513   f_{ck} characteristic concrete compressive strength

514   f_{ctm} characteristic tensile splitting strength of the concrete

515   f_{tm} average tensile strength of the steel

516   f_{yd} design yield strength of the steel

517   f_{ym} average yield strength of the steel

518   m_{Ed} design action moment on cross-section

519   m_{Rd} design resistance moment of cross-section

520     $v_{Rd,c}$ design shear resistance

521   A_s longitudinal reinforcement

522   E_s modulus of elasticity of reinforcement steel

523   I the AASHTO specifications impact allowance

524   I_{DL} deviation from linearity index

525   I_{pi} permanency index for the i-th load cycle

526   I_{pr} permanency ratio

527   K_a updating factor based on test results

528   K_b updating factor based on situation of considered structural member

529   L_R the comparable live load due to the rating vehicles for the lanes loaded

530   L_T target proof load

531     $P_{load,bending}$ required proof load for bending moment

532	$P_{load,shear}$	required proof load for shear
533	RF	rating factor
534	RF_T	updated rating factor based on proof load test results
535	UC	unity check
536	X_p	factor to determine target proof load, without adjustments
537	X_{pA}	target live load factor
538	α	angle of line between origin of load-displacement diagram and maximum value of
539		considered load cycle
540	α_i	angle of line between origin of load-displacement diagram and maximum value of
541		load cycle i
542	α_{ref}	angle of line between origin of load-displacement diagram and maximum value of
543		first load cycle
544	β	reliability index
545	γ_{as}	load factor for the superimposed dead load
546	γ_{ll}	load factor for the live load
547	γ_{sw}	load factor for the self-weight
548	ε_c	the theoretically determined strain in the finite element model under the
549		maximum proof load
550	$\varepsilon_{c,meas}$	strain measured during proof loading
551	$\varepsilon_{c,lim}$	limit value of the concrete strain : 0.6 ‰, and for $f_{cd} \geq 25$ MPa this can be
552		increased up to maximum 0.8 ‰.
553	ε_{c0}	analytically determined short-term strain in the concrete caused by the permanent
554		loads that are acting on the structure before the application of the proof load

555   ε_{s02} analytically determined strain (assuming cracked conditions) in the reinforcement

556   steel caused by the permanent loads that are acting on the structure before the

557   application of the proof load.

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

559   measurements

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

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

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

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

564

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789

790

791 **List of Figures**

792 **Fig. 1:** Viaduct De Beek: (a) overview photograph; (b) current traffic restrictions.

793 **Fig. 2:** Geometry of viaduct De Beek: (a) top view; (b) longitudinal direction (cut A-A'); (c)

794 transverse direction (cut C-C'). All dimensions in [mm].

795 **Fig. 3:** Reinforcement drawing of viaduct De Beek: (a) side view; (b) cross-section. Units: bar

796 diameters in mm, all other distances in cm.

797 **Fig. 4:** Position of proof load tandem: top view of viaduct.

798 **Fig. 5:** Test setup with load spreader beam, ballast, and jacks.

799 **Fig. 6:** Loading protocol, showing numbering of different loading cycles: (a) bending moment

800 test; (b) shear test.

801 **Fig. 7:** Sensor plan: (a) position of deflection measurements; (b) position of strain

802 measurements for the bending moment test; (c) position of strain measurements for the shear test

803 (position of strain gages not altered with respect to the bending moment test).

804 **Fig. 8:** Overview of measurements of the bending moment test: (a) envelope of the load-

805 displacement diagram at LVDT7; (b) comparison between calculated and measured strains in the

806 transverse direction; (c) slope of load-displacement diagram per load cycle; (d) longitudinal

807 deflection.

808 **Fig. 9:** Overview of measurements of the shear test: (a) envelope of the load-displacement

809 diagram at LVDT6; (b) comparison between calculated and measured deflections in the

810 longitudinal direction; (c) slope of load-displacement diagram per load cycle; (d) longitudinal

811 deflection.

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813 **List of Tables**

814 **Table 1**. Overview of different safety levels used in the Netherlands for the assessment of

815 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

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

819   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

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822   **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

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