

An ACI Technical Publication

SYMPOSIUM VOLUME



Evaluation of Concrete Bridge Behavior through Load Testing – International Perspectives

Editors:
Eva Lantsoght and Pinar Okumus

SP-323



American Concrete Institute
Always advancing



Documents
Document Sharing Hub

Evaluation of Concrete Bridge Behavior through Load Testing – International Perspectives

Editors:
Eva Lantsoght and
Pinar Okumus



American Concrete Institute

Always advancing

SP-323

First printing, May 2018

Discussion is welcomed for all materials published in this issue and will appear ten months from this journal's date if the discussion is received within four months of the paper's print publication. Discussion of material received after specified dates will be considered individually for publication or private response. ACI Standards published in ACI Journals for public comment have discussion due dates printed with the Standard.

The Institute is not responsible for the statements or opinions expressed in its publications. Institute publications are not able to, nor intended to, supplant individual training, responsibility, or judgment of the user, or the supplier, of the information presented.

The papers in this volume have been reviewed under Institute publication procedures by individuals expert in the subject areas of the papers.

Copyright © 2018
AMERICAN CONCRETE INSTITUTE
38800 Country Club Dr.
Farmington Hills, Michigan 48331

All rights reserved, including rights of reproduction and use in any form or by any means, including the making of copies by any photo process, or by any electronic or mechanical device, printed or written or oral, or recording for sound or visual reproduction or for use in any knowledge or retrieval system or device, unless permission in writing is obtained from the copyright proprietors.

On the cover: Load test of a cable stayed bridge over Wisłok River, Poland. Photograph by Dr. K. Wilde.

Printed in the United States of America

Editorial production: Susan K. Esper

ISBN-13: 978-1-64195-007-7

Preface

Load testing of concrete bridges is a practice with a long history. Historically, and particularly before the unification of design and construction practices through codes, load testing was performed to show the travelling public that a newly built bridge was safe for use. Nowadays, with the aging infrastructure and increasing loads in developed countries, load testing is performed mostly for existing structures either as diagnostic or proof tests. For newly built bridges, diagnostic load testing may be required as a verification of design assumptions, particularly for atypical bridge materials, designs, or geometries. For existing bridges, diagnostic load testing may be used to improve analysis assumptions such as composite action between girders and deck, and contribution of parapets and other nonstructural members to stiffness. Proof load testing may be used to demonstrate that a structure can carry a given load when there are doubts with regard to the effect of material degradation, or when sufficient information about the structure is lacking to carry out an analytical assessment.

In recent years, both researchers and practicing engineers worldwide have been refining load testing methods to balance accuracy, cost, effort, and time, and have been addressing increasingly complex structures and situations. To exchange international experiences among a global group of researchers and compare load testing methods used internationally, ACI Committee 342 organized two sessions titled “Evaluation of Concrete Bridge Behaviour through Load Testing – International Perspective” at the 2017 ACI Fall Convention in Anaheim, CA. This Special Publication contains several technical papers from experts who presented their work at these sessions, in addition to papers submitted for publication only.

This Special Publication combines contributions from different regions of the world, and in particular from Denmark, Germany, the Netherlands, Poland, Spain, Sweden, and from different regions in the United States. The technical papers consider both theoretical and practical aspects of load testing, discuss different levels of bridge behaviour assessment such as visual inspections, modelling, and load testing. They introduce the reader to the codes and guidelines that may only be available in some countries. The impact of differences in live loads, design codes, reserve capacities, age of structures, construction practices between Europe, and North America on assessment of concrete bridges is reflected by case studies. Recent developments with regard to codes and standards around the world for load testing are discussed, and open questions for future developments are highlighted by the authors.

The wide variety of concrete bridge structures investigated included short-span reinforced concrete slab bridges, older reinforced concrete earth-filled arch bridges, bridges that have been damaged and/or retrofitted, and modern prestressed concrete bridges with new materials. Reasons why load testing is required also vary and include apparent damage, opportunities created by decommissioned bridges, necessity to carry super heavy vehicles, use of unique materials or geometry, and absence of design plans. Results of testing bridges under static or dynamic service loads create knowledge on expected service

performance and allow load ratings, while testing decommissioned bridges to near collapse or collapse reveals true capacity and the level of conservatism in design assumptions. Several papers highlight vehicles or rigs designed specifically for reuse in standardized load testing. Others use recent technology such as 3-D scanning or digital image correlation to collect data, in addition to traditional methods such as strain gauges. As such, this Special Publication provides a global perspective on strategies for assessing the in-service performance of concrete bridges, and an overview of the state-of-the-art with regard to load testing internationally.

Overall, in this Special Publication, authors from different backgrounds and geographical locations share their experiences and perspectives on load testing and its impact on understanding concrete bridge behaviour. The coeditors, Dr. Pinar Okumus and Dr. Eva Lantsoght, are grateful for the contributions of the Special Publication authors and sincerely value the time and effort of the authors in preparing the papers in this volume.

Eva Lantsoght and Pinar Okumus
Co-Editors

TABLE OF CONTENTS

SP-323—1

Assessment of Slab Bridges through Proof Loading in the Netherlands

Authors: Eva O. L. Lantsoght, Cor van der Veen, Ane de Boer, and Dick A. Hordijk

SP-323—2

Load Testing of Highly Skewed Concrete Bridges

Authors: Mauricio Diaz Arancibia and Pinar Okumus

SP-323—3

Rating of Concrete Road Bridges with Static Proof Load Tests

Authors: Anna Halicka, Dick A. Hordijk, and Eva O.L. Lantsoght

SP-323—4

Bridge Load Testing and Monitoring for Super-Heavy Permit Loads

Authors: Brett Commander and Jesse Sipple

SP-323—5

Rating of Prestressed Concrete Adjacent Beam Bridges without Plans

Authors: Carlos V. Aguilar, David V. Jáuregui, Brad D. Weldon, and Craig M. Newton

SP-323—6

Bridge Load Testing In Germany

Authors: Gregor Schacht, Frederik Wedel, and Steffen Marx

SP-323—7

Diagnostic Load Testing Of Concrete Bridges, Principles and Example

Authors: Joan Ramon Casas, Piotr Olaszek, Juliusz Ciesla, and Krzysztof Germaniuk

SP-323—8

Assessment and Loading to Failure of Three Swedish RC Bridges

Authors: Jonny Nilimaa, Cristian Sabau, Niklas Bagge, Arto Puurula, Gabriel Sas, Thomas Blanksvärd, Björn Täljsten, Anders Carolin, Björn Paulsson, and Lennart Elfgrén

SP-323—9

High Magnitude Loading of Concrete Bridges

Authors: Jacob W. Schmidt, Philip S. Halding, Thomas W. Jensen, and Svend Engelund

SP-323—10

Torsional Effects on Load Tests to Quantify Shear Distribution in Prestressed Concrete Girder Bridges

Authors: Benjamin Z. Dymond, Catherine E. W. French, and Carol K. Shield

SP-323—11

Diagnostic Test for Load Rating of a Prestressed SCC Bridge

Authors: E. S. Hernandez and J.J. Myers

SP-323—12

Extending the Life of Aged, Reinforced Concrete Arch Bridges through Load Testing and Monitoring

Authors: Jeffrey Weidner, John Prader, Nathaniel Dubbs, Franklin L. Moon, A. Emin Aktan, John Taylor, and Clifford J. Skeens

ASSESSMENT OF SLAB BRIDGES THROUGH PROOF LOADING IN THE NETHERLANDS

Eva O. L. Lantsoght, Cor van der Veen, Ane de Boer and Dick A. Hordijk

Synopsis: A large subset of the Dutch bridge stock consists of reinforced concrete slab bridges, for which assessment often results in low ratings. To prioritize the efforts of the bridge owner, more suitable assessment methods for slab bridges are necessary. Research efforts over the past years resulted in the development of several methods, at levels requiring increasing costs, time, and effort for increasing accuracy. The last option, when an analytical assessment is not possible due to uncertainties, is to use proof load testing to evaluate the bridge directly. To develop recommendations for the proof load testing of reinforced concrete slab bridges for the Netherlands, different methods are combined: pilot proof load tests on bridges with and without material damage, a collapse test, tests on beams taken from an existing bridge and new beams with similar dimensions cast in the laboratory, and an extensive literature review. The result of this study is a set of recommendations that describe how to prepare and execute a proof load test, and how to analyze the results. This paper summarizes the research program about proof load testing from the Netherlands and gives an overview of the currently developed recommendations and topics for further research.

Keywords: field testing; flexure; measurements; proof load testing; reinforced concrete; shear; slab bridges

ACI member **Eva O. L. Lantsoght** is a full professor at Universidad San Francisco de Quito, a structural engineer at Adstren and a researcher at Delft University of Technology. She is a member of ACI 445-0D Shear Databases and of ACI-ASCE 421, Design of Reinforced Concrete Slabs, and an associate member of ACI 342, Evaluation of Concrete Bridges and Bridge Elements, ACI 437, Strength Evaluation of Existing Concrete Structures, and ACI-ASCE 445, Shear and Torsion.

Cor van der Veen is an associate professor at Delft University of Technology, Delft, The Netherlands. He received his M.Sc. and Ph.D. from Delft University of Technology. He is a member of various National Committees. His research interests include (very) high strength (steel fiber) concrete, concrete bridges and computational mechanics.

Ane de Boer is a senior advisor at Rijkswaterstaat, the Ministry of Infrastructure and the Environment, Utrecht, The Netherlands. He received his M.Sc. and Ph.D. from Delft University of Technology. He is a member of some National Committees, *fib* COM3 and member of an IABSE Working Committee. His research interests are remaining lifetime, existing structures, computational mechanics, traffic loads and composites.

Dick A. Hordijk is a full professor at Delft University of Technology, Delft, the Netherlands. He received his M.Sc. and Ph.D. in civil engineering from Delft University of Technology, Delft, the Netherlands, in 1985 and 1990, respectively. His research interests include concrete fracture mechanics, assessment of existing structures, structural application of new concrete types and forensic engineering.

INTRODUCTION

Existing bridges in the Netherlands

The expansion of the Dutch road network after the Second World War included the construction of a large number of bridges. These bridges are now approaching the end of their originally devised service life of 80 years. To evaluate if extension of the service life of these bridges is possible, and if they are suitable for carrying the current traffic, assessment of these structures is required.

A large subset of the Dutch bridge stock consists of reinforced concrete slab bridges. Of the bridges built during the 1960s and 1970s, 50% are reinforced concrete slab bridges. In later periods, the majority of bridges were prestressed prefabricated girder bridges. The reinforced concrete slab bridges are typically short span bridges in or over the highway, and can be a single span or multiple spans. A first assessment, based on the code provisions for the design of new structures (Vergoossen et al., 2013), showed that about 600 of these bridges do not fulfil these code requirements. In many cases, the shear capacity was insufficient. This observation resulted from the fact that the applied live loads according to NEN-EN 1991-2:2003 (CEN, 2003) are heavier than those used in the past, and that the shear provisions described in NEN-EN 1992-1-1:2005 (CEN, 2005) generally lead to lower shear capacities than the previously governing national code NEN 6720:1995 (Code Committee 351001, 1995).

Assessment based on Levels of Approximation

The *fib* Model Code 2010 (fib, 2012) introduced the Levels of Approximation concept for the first time. A low Level of Approximation is a conservative method that can be used, for example, for preliminary design. If the assessment requires a more accurate answer, a higher Level of Approximation is necessary. This higher Level of Approximation will require more time to set up the model, and increased computational time and effort as compared to a lower Level of Approximation. Belletti et al. (2015) recommend different Levels of Approximation for the shear and punching provisions (Belletti et al., 2015), as used in the *fib* Model Code 2012 (fib, 2012). Figure 1 illustrates the concept of using Levels of Approximation.

For the assessment of the shear capacity of reinforced concrete slab bridges, a method with Levels of Approximation was developed as well (Lantsoght et al., 2017a), called Levels of Assessment. In total, four Levels of Assessment were identified:

1. Level of Assessment 1 – Quick Scan method: the Quick Scan method (Lantsoght et al., 2013b; Lantsoght et al., 2016f) is a fast, spreadsheet-based calculation that mimics a hand calculation. The provisions of the Eurocodes NEN EN 1991-2:2003 (CEN, 2003) and NEN-EN 1991-2-2:2005 (CEN, 2005) are followed and extended with recommendations based on slab experiments (Lantsoght et al., 2013c; Lantsoght et al., 2014b; Lantsoght et al., 2015a; Lantsoght et al., 2015c). The result is a Unity Check: the ratio of the shear stress caused by the loads (self-weight, superimposed dead load, and live loads) to the shear capacity. If the Unity Check is smaller than or equal to one, no further refinement of the assessment method is required and the code requirements for the bridge are fulfilled. If the Unity Check is larger than 1, a higher Level of Assessment is necessary.

2. Level of Assessment 2 – Linear finite element models: at Level of Assessment 2, the shear stresses caused by the loads (self-weight, superimposed dead load, live loads consisting of distributed lane loads and concentrated truck loads) is determined by making a linear finite element model of the superstructure. The resulting peak shear stress is then distributed transversely over a width of $4d_l$, with d_l the effective depth to the longitudinal reinforcement (Lantsoght et al., 2013a; Lantsoght et al., 2014a). This averaged shear stress is then compared to the shear capacity from the provisions of NEN-EN 1992-1-1:2005 (CEN, 2005) to determine the Unity Check based on Level of Assessment 2. If the Unity Check is smaller than or equal to 1, the structure fulfills the code requirements. If the Unity Check is larger than 1, the assessment needs to be continued at a higher Level of Assessment.
3. Level of Assessment 3 – nonlinear finite element models and probabilistic analyses. Recommendations (Rijkswaterstaat, 2016) were developed for the use of nonlinear finite element models, and the required safety formats have been described (Belletti et al., 2010; Belletti et al., 2013). The result of such a calculation is an estimate of the full behavior of the bridge from initial loading to the ultimate limit state. As such, the model gives insight in the capacity beyond the code provisions. If the applied loading results in an action that is lower than the determined capacity, the structure fulfils the requirements. This Level of Assessment can also include probabilistic calculations (Lantsoght et al., 2015b; Lantsoght et al., 2016e). Recommendations for this application can be found in the Probabilistic Model Code (JCSS, 2001; Vrouwenvelder et al., 2002).
4. Level of Assessment 4 – field testing. If the calculations at lower Levels of Assessment show that the structure does not have sufficient capacity to carry the prescribed loads, but when there are reasons to expect additional capacity, the assessment can be based on a field test. A proof load test, in which the bridge is loaded with a load representative of the factored live loads, can lead to an experimental approval of the bridge. A diagnostic load test, in which the bridge is loaded with a lower load and the structural response is measured, can be used to update the models developed with Level of Assessment 2 or 3. Field testing only brings new information when the uncertainties are large (Lantsoght et al., 2017b), and when it has been defined prior to the test what are the objectives, and what needs to be measured to fulfill these objectives. Load testing is significantly more expensive than the other Levels of Assessment, and therefore a solid preparation of a load test is always required. This preparation should explore the possible benefits of a load test, and weigh these benefits against the cost of testing and/or strengthening or replacing of the considered bridge.

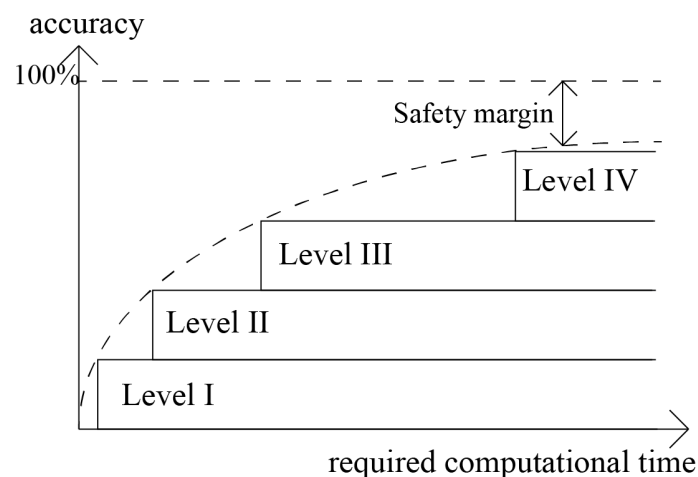


Figure 1 — Principle of Levels of Approximation (fib, 2012).

RESEARCH SIGNIFICANCE

Load testing is not a new practice, and extensive research from the past about load testing is available. However, many load testing guidelines and codes prescribe practices that are based on rules of thumb. The presented research combined field testing, a collapse test, and additional laboratory testing to develop evidence-based recommendations for the determination of the target proof load, for the loading protocol, and for the threshold values of the measurements to be used during proof load testing. As such, these results and the experience gathered in the Netherlands can be valuable for the international community of bridge engineers, to improve the current practice of proof load testing.

LITERATURE REVIEW

Two different types of load tests exist: diagnostic load tests and proof load tests. In a diagnostic load test (Fu et al., 1997; Matta et al., 2008; Moses et al., 1994; Olaszek et al., 2014; Russo et al., 2000; Sanayei et al.,

2016; Velázquez et al., 2000) a low load is applied and the structural response is measured. These measurements then lead to updating of the analytical model, often a finite element model. The updated model can subsequently be used to improve the assessment of the structure. The field measurements can give information about the transverse flexural distribution, the stiffness of the structure and the effect of non-structural members such as parapets and barriers on the stiffness of the system, and the effect of frozen bearings on the distribution of forces.

In a proof load test (Anay et al., 2016; Arangjelovski et al., 2015; Casas and Gómez, 2013; Faber et al., 2000; Olaszek et al., 2012; Saraf et al., 1996; Stewart and Val, 1999), the bridge is loaded with a load that is representative of the factored live load. If the bridge can carry this load level without signs of distress, the test shows that the structure fulfills the code requirements. Since a proof load test involves high loads, the applied sensors become important to evaluate if further loading may lead to irreversible damage to the structure. For this purpose, the measurements from the sensors are compared to thresholds, the so-called “stop criteria.” If a stop criterion is exceeded, further loading is not permitted and the proof load test needs to be terminated, even when the target proof load has not been achieved yet. The structure may then perhaps fulfil lower requirements, depending on how high the load was at exceedance of the stop criterion. This concept for proof load testing is shown in Figure 2, in which G_1 is the effect of the permanent loads, $effR_u$ is the capacity of the structure, F_{lim} is the load for which a stop criterion is exceeded, $ext.F_{lim}$ is the load above the permanent loads G_1 for which a stop criterion is exceeded, G_{dj} is the additional permanent loads, Q_d are the live loads, $ext.F_{target}$ is the proof load that should be applied in the test in addition to the already present load G_1 , and F_{target} is the total target proof load. Additionally, the measurements can be interesting for comparison to the output of a calculation carried out at Level of Assessment 3.

The prescribed stop criteria depend on the considered failure mode. The existing codes for proof load testing of concrete bridges from France (Cochet et al., 2004), Poland (PN-S-10040/99, 1999), and the Czech Republic and Slovakia (Frýba and Pirner, 2001) include stop criteria for flexure that are based on a maximum deflection and crack width. Other guidelines, such as the AASHTO LRFR Manual for Bridge Evaluation (AASHTO, 2016) mention in a more general way that the proof load test shall not result in non-linear behavior of the structure and/or irreversible damage.

A full literature review on the topic was developed as part of this research program (Lantsoght, 2016; Lantsoght et al., 2017h).

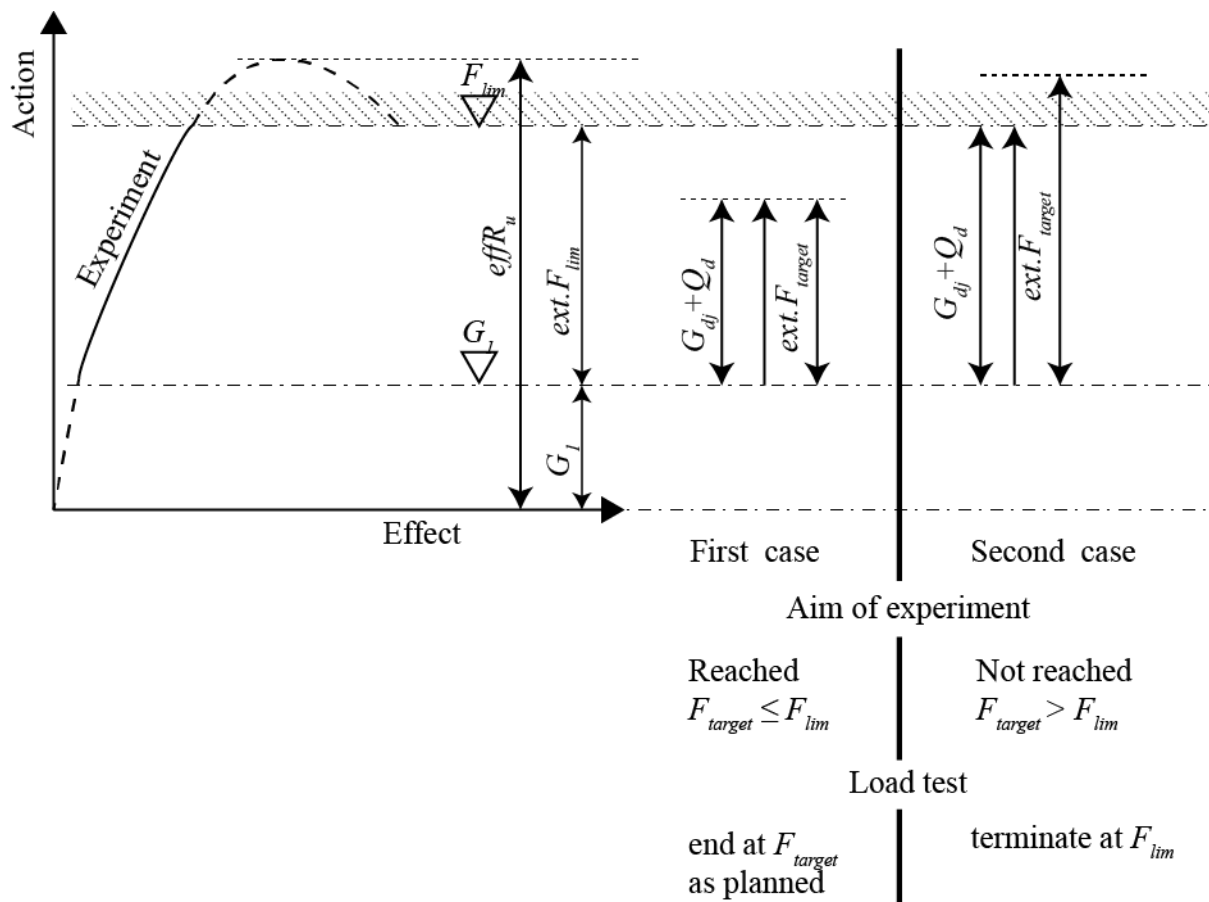


Figure 2 — Safety philosophy for proof load testing, originally proposed in the German guidelines for proof load testing (Deutscher Ausschuss für Stahlbeton, 2000).

PROOF LOAD TESTING RESEARCH IN THE NETHERLANDS

Pilot proof load tests

Over the past decade, a number of pilot proof load tests have been carried out in the Netherlands (Lantsoght et al., in press-b). The first pilot test was carried out on the viaduct Heidijk in 2007 (Dieteren and den Uijl, 2009), a continuous reinforced concrete slab bridge. This viaduct suffered material damage caused by alkali silica reaction (ASR), which resulted in an insufficient shear capacity upon assessment, as in the Netherlands the shear capacity of elements with ASR damage is assumed to be 75% of the capacity according to the code provisions (den Uijl and Kaptijn, 2004). The load in this test was applied by using a loading frame, ground anchors, and jacks. The maximum applied load was 640 kN (144 kip), which resulted in the conclusion that the viaduct could carry vehicles of 30 metric tons (66 kip).

The second test was carried out in 2009 on viaduct Medemblik, which is a girder bridge with damage caused by reinforcement corrosion. For this case, the BELFA vehicle from Germany (a special vehicle used for load testing) (Bretschneider et al., 2012) was used. The maximum applied load was 545 kN (123 kip) and the conclusion was that the viaduct could carry one lane of vehicles of 30 metric tons (66 kip).

The first two pilot proof load tests were carried out with minimal involvement of Delft University of Technology. In the subsequent tests, the involvement of Delft University of Technology became more important, and the goals of the proof load tests shifted from the evaluation of isolated cases to a broader research program.

The first pilot proof load test that had partial involvement of Delft University of Technology was the test on the viaduct Vlijmen Oost in 2013 (Fennis et al., 2014), see Figure 3a. This viaduct is a continuous reinforced concrete solid slab bridge with material damage caused by ASR. The BELFA vehicle applied the load, and the maximum applied load was 900 kN (202 kip). A difficulty in this test was that one lane had to remain available for traffic, whereas only two lanes were available for testing (one lane with the BELFA and one lane with a crane to provide additional counterweight). As a result, the level of noise on the measurements was rather large.

In 2014, the Halvemaans Bridge (Fennis and Hordijk, 2014), see Figure 3b, was tested. This single-span reinforced concrete slab bridge from 1939 was evaluated for flexure. In the proof load test, a system with a load spreader beam, counterweights, and hydraulic jacks applied the load in a cyclic manner. The maximum applied load was 900 kN (202 kip), which showed that the bridge fulfils the requirements of the Dutch guidelines for the assessment of bridges (Richtlijn Beoordelen Kunstwerken, RBK, RTD 1006:2013 1.1) (Rijkswaterstaat, 2013) at the Reconstruction level.



Figure 3 — Overview of bridges tested by Delft University of Technology in proof load testing program:
(a) Viaduct Vlijmen Oost; (b) Halvemaans Bridge; (c) Viaduct Zijlweg; (d) Viaduct De Beek.

In June 2015, the viaduct Zijlweg (Koekkoek et al., 2015; Lantsoght et al., 2017b), see Figure 3c, was proof load tested. This continuous reinforced concrete slab bridge has material damage caused by ASR. As a result, there were concerns with regard to the shear capacity of the bridge. The first span was tested at two positions: a shear-critical and a flexure-critical position by using a combination of a steel spreader beam, counterweights, and hydraulic jacks. A cyclic loading protocol was used. The maximum loads were 1368 kN (308 kip) for the bending moment test and 1377 kN (310 kip) for the shear test. With these loads, the conclusion is that the viaduct fulfils the requirements of the RBK at the Design level.

In November 2015, the viaduct De Beek (Koekkoek et al., 2016; Lantsoght et al., 2017f; Lantsoght et al., in press-a), see Figure 3d, was proof load tested. This continuous reinforced concrete slab bridge has significant flexural cracking. It was found upon assessment that the flexural capacity of the bridge is insufficient. As a result, the two lanes available on the bridge were reduced to a single lane. Two positions in the first span were tested: a shear-critical and a flexure-critical position. Again, a combination of a steel spreader beam, counterweights, and hydraulic jacks, and a cyclic loading protocol loaded the bridge. The maximum loads were 1751 kN (394 kip) for the bending moment test and 1560 kN (351 kip) for the shear test. With these loads, it the first span fulfills the requirements of the RBK at the Design level for two lanes of traffic. However, testing the second span was not possible, because this would require closing of the highway. This span had an even higher Unity Check than the first span. For the second span, an assessment assuming plastic redistribution was carried out, for which it was found that two lanes of traffic can be permitted, provided that no corrosion damage has occurred which would further reduce the available amount of flexural reinforcement.

Collapse test

In addition to the pilot proof load tests, a collapse test was carried out on two spans of the Ruytenschildt Bridge (Lantsoght et al., 2016a; Lantsoght et al., 2016b; Lantsoght et al., 2016c; Lantsoght et al., 2016e; Lantsoght et al., 2017e) in August 2014. The Ruytenschildt Bridge, see Figure 4, was a five-span continuous reinforced concrete slab bridge. A new structure for functional reasons now replaces the previous Ruytenschildt Bridge. Therefore, the old bridge could be tested to collapse for research purposes. A steel spreader beam, counterweights, and hydraulic jacks, which permitted the use of a cyclic loading protocol, loaded the bridge. The maximum applied load in the first span was 3049 kN (685 kip), which was the maximum available counterweight. The cross-section did not fail, but showed signs of flexural distress. For the test in the second span, additional counterweight was ordered, and the maximum load was 3991 kN (897 kip). This load caused yielding of the flexural reinforcement, large cracking, and a settlement of the pier.

The results of the collapse tests give information for the verification of proposed calculation methods for reinforced concrete slabs, such as the Extended Strip Model (Lantsoght et al., 2016d; Lantsoght et al., 2017c; Lantsoght et al., 2017d). The results have also been used to evaluate the margin of safety (Lantsoght, 2017a) provided by the stop criteria from existing codes and guidelines such as the German guidelines (Deutscher Ausschuss für Stahlbeton, 2000) and ACI 437.2M-13 (ACI Committee 437, 2013), as well as stop criteria proposed as part of this research (Lantsoght et al., 2017g). As such, the results of the tests on the Ruytenschildt Bridge have been important for the development of the recommendations for proof load testing of reinforced concrete slab bridges.



Figure 4 — Ruytenschildt Bridge in 2014.

Laboratory testing

Two sets of laboratory experiments have been carried out and analyzed to develop recommendations for proof load testing. The experiments consist of testing reinforced concrete beams without shear reinforcement in a cyclic manner. The first set of experiments consists of five tests on three beams sawn from the Ruytenschildt Bridge (Lantsoght et al., 2016b). Four of the experiments resulted in a flexural failure, and one test resulted in a flexural shear failure. Figure 5 shows the observed failure modes. All experiments were carried out in a cyclic manner, so that they could be used to evaluate the margin of safety of the stop criteria from the German guidelines and from ACI 437.2M-13 (Lantsoght et al., 2016g). This analysis could then be used to develop and improve recommendations for stop criteria for proof load testing (Lantsoght, 2017c).

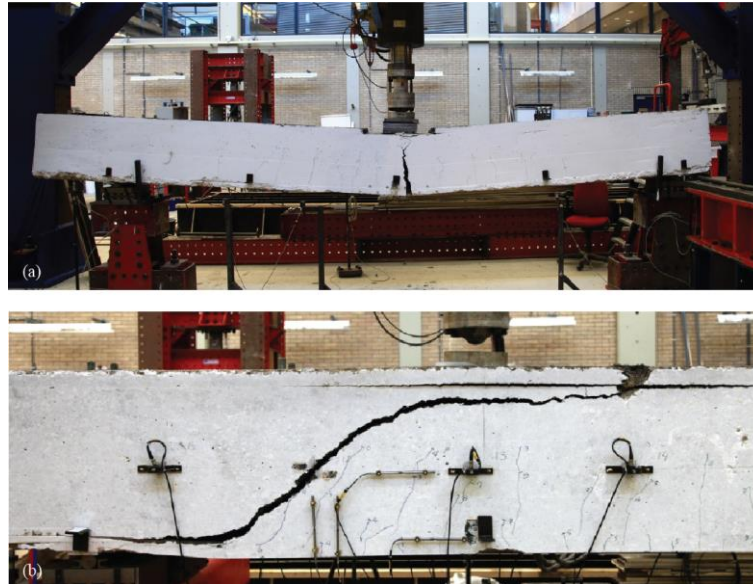


Figure 5 — Failure modes observed in the beams sawn from the Ruytenschildt Bridge: (a) flexural failure, RSB01; (b) shear failure, RSB03A (Yang, 2015).

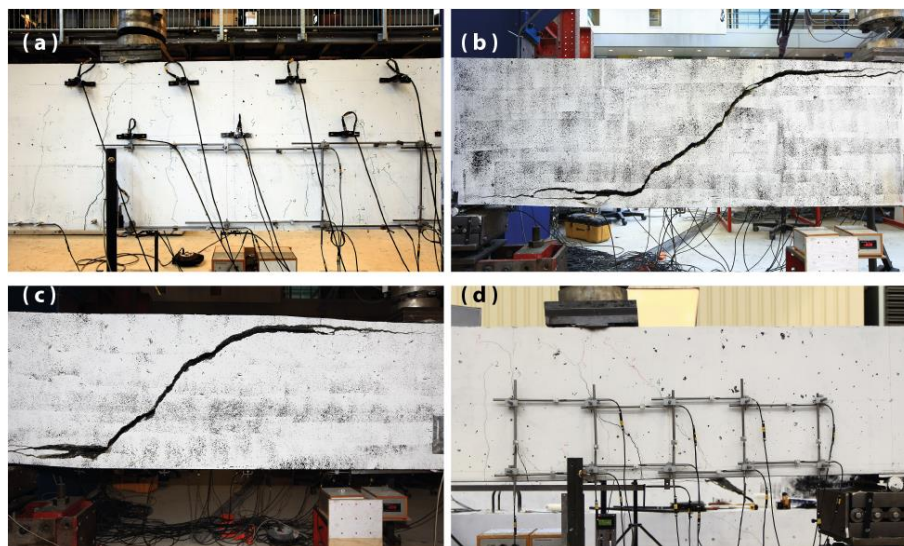


Figure 6 — Failure modes observed experiments on beams cast in the lab: (a) flexural failure of a beam without flexural cracks, P804A1; (b) shear failure of a beam with flexural cracks, P804A2; (c) shear failure of a beam without flexural cracks, P804B; (d) flexural failure of a beam with flexural cracks, P502A2.

The second set of experiments consists of four tests on two beams reinforced with plain bars, and cast in the laboratory (Lantsoght et al., 2017g). These tests studied the following parameters to develop recommendations for the loading protocol:

- loading speed
- number of cycles

- length of loading at the peak of a load cycle
- length of resting period between load cycles
- load levels

Additionally, the results were used to evaluate the margin of safety of the stop criteria from the German guidelines and from ACI 437.2M-13, as well as to develop new recommendations for stop criteria. Two experiments resulted in a flexural failure, and two experiments resulted in a shear failure. The presence of flexural cracks was found to have an effect on the margin of safety provided by the stop criteria. Therefore, this research resulted in the recommendation that for the stop criteria, four different cases need to be distinguished:

1. Flexural failure of an element without flexural cracks, see Figure 6a.
2. Flexural failure of an element with flexural cracks, see Figure 6d.
3. Shear failure of an element without flexural cracks, see Figure 6c.
4. Shear failure of an element with flexural cracks, see Figure 6b.

Note that the terms without and with flexural cracks refer to the state of the element at the beginning of the test. A test on a member that is precracked by a previous experiment is denoted as a test on a member with flexural cracks, whereas a test on a virgin specimen is denoted as a test on a member without flexural cracks.

RESULTING RECOMMENDATIONS FOR PROOF LOAD TESTING

Recommendations for the preparation of proof load tests

In the Netherlands, the procedure of using the Levels of Assessment is started when an inspection leads to the observation of material damage (such as damage caused by ASR), or when it is expected that the bridge does not fulfil the current requirements. These inspections are carried out once every six years. If regular assessment calculations show that the capacity is insufficient, linear or nonlinear finite element models are developed. These calculations are often complimented with the determination of the concrete compressive strength from core samples. If uncertainties about the structural behavior or the effect of material degradation remain, proof load testing of the bridge can become a viable assessment option.

The first step in the preparation of a proof load test is to consider if the structure is suitable for proof load testing. The engineer should gather the available information of the structure, and discuss the goals of the proof load test with the bridge owner. In order to approve a bridge experimentally through a proof load test, there must be uncertainties on the analytical assessment. The outcome of the proof load test then removes these uncertainties. The engineer should evaluate if proof load testing is the right method to address these uncertainties, or if diagnostic load testing can be sufficient for the case under consideration. Cases where proof load testing is recommended are when crucial information about the structure is missing (Aguilar et al., 2015), or when the effect of material deterioration on the capacity is unknown (Lantsoght et al., 2017b).

When it is determined that the structure is suitable for proof load testing, the test needs to be prepared. The first step in the preparation of a proof load test is an inspection of the bridge and its surroundings. In such an inspection, the engineer should identify if there are limitations in the access to the bridge site for the application of the load and the measurements. The engineer should note changes to the bridge as compared to the as-built drawings (widening of the superstructure, an increase in the thickness of the layer of asphalt, an overlay with high strength concrete to strengthen the bridge deck, or changes in lane layout). The effect of these changes on the assessment should be checked. The conditions of the joints and bearings must be inspected. If these conditions can have an effect on the distribution of forces in the structure, they should be taken into account for the assessment calculations and the preparatory calculations for the test. The inspection should also include a study of the cracks that are present on the bottom face of the deck and on the side faces. Inspection of the top face is only possible if no asphalt layer is present. The inspection of the cracks leads to the drawing of a map of the cracks. The position of the cracks determines the position of sensors that measure the increase in crack width during the proof load test.

After the inspection, preparatory calculations of the capacity are required. It is recommended (Lantsoght et al., in press-b) to use the mean material properties to estimate the expected value of the bending moment capacity, the shear capacity, and the punching capacity of the deck. If possible, the mean material properties should be determined by drilling concrete cores to find the mean concrete compressive strength and by taking samples of the reinforcement steel to find the average value of the yield strength of the steel. The engineer should calculate the expected deflections during the proof load test to select the correct measurement range for the sensors. The moment-curvature diagram and load-displacement diagram of the slab can be prepared based on hand calculations, to have a first idea of the expected behavior during the test. If it is not possible to approximate the behavior of the slab bridge during the experiment with simple hand calculations, then sometimes it may be necessary to model the bridge with nonlinear finite element models.

The next step in the preparation of a proof load test is to make a linear finite element model of the slab to determine the position and magnitude of the proof load. Since this model is only used to study the loading side of the equation, it is sufficient to use shell elements and to assume a Poisson's ratio of 0.2 and the Young's modulus of uncracked concrete. The model uses the following loads:

1. The self-weight of the superstructure. If the model is a simplified representation of the geometry, the model should include the equivalent load of the omitted parts. For concrete, a distributed load of 25 kN/m³ (159 lb/ft³) can be assumed.
2. The superimposed dead load. If asphalt is used, a distributed load of 23 kN/m³ (146 lb/ft³) can be assumed.
3. The live loads. The assessment of the bridges in the Netherlands is according to the loads prescribed by NEN-EN 1991-2:2003 (CEN, 2003) Load Model 1. This live load model uses a distributed load in all lanes, with a heavier distributed load in the first lane. Each lane holds a design tandem. The value of the applied load on the axles of the design tandem is different for each lane. Since the model uses shell elements, the concentrated loads can be distributed to mid-height of the slab, assuming a vertical load distribution of 45°, so that the resulting wheel print is larger than at the surface of the slab.

The load factors that are used to make the load combination are defined by the RBK (guidelines for the assessment of existing bridges) (Rijkswaterstaat, 2013) for all bridges that are owned by the Dutch Ministry of Infrastructure and the Environment. These bridges are highway bridges of Eurocode Consequences Class 3 as defined in NEN-EN 1990:2002 (CEN, 2002). For assessment, the minimum safety level from the RBK that a bridge has to fulfill is the “Usage” level. For this level, the load factors correspond to a reliability index of 3.3 and a reference period of 30 years. This required reliability index is high as compared to the target reliability index for the AASHTO LRFR (AASHTO, 2016) operating rating level with $\beta = 2.5$. For the pilot proof load tests, the evaluation includes all safety levels described in the RBK. Table 1 shows these safety levels, in which γ_{sw} is the load factor for the self-weight, γ_{sd} is the load factor for the superimposed dead load, and γ_{ll} is the load factor for the live loads. In the RBK, the same value for γ_{sw} and γ_{sd} is used. However, in combination with proof load testing, the geometry of the structure is not a random variable anymore, so that a reduction of the load factor for the self-weight to its model factor of 1.10 is possible.

Table 1 – Safety levels described by RBK and associated load factors as used in combination with proof load testing (Rijkswaterstaat, 2013)

Safety level	β	Reference period	γ_{sw}	γ_{sd}	γ_{ll}
RBK Design	4.3	100 years	1.10	1.25	1.50
RBK Reconstruction	3.6	30 years	1.10	1.15	1.30
RBK Usage	3.3	30 years	1.10	1.15	1.25
RBK Disapproval	3.1	15 years	1.10	1.10	1.25

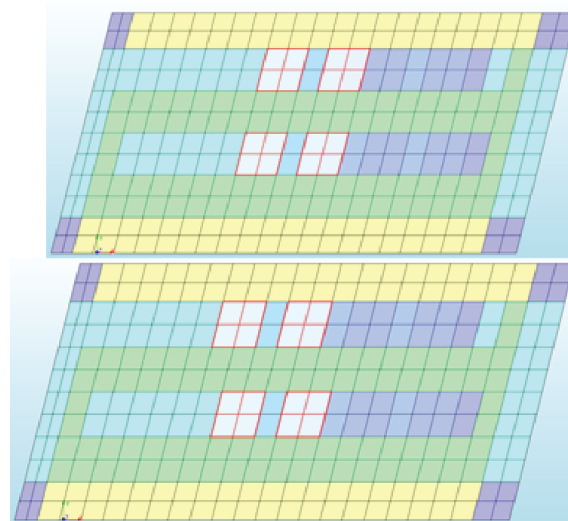


Figure 7 — Moving of the design tandem in its lane, for a one-lane bridge (viaduct Zijlweg).

With the prescribed finite element model, the critical position of the design tandems is determined. For the failure mode of bending, the design trucks are moved in their respective lanes until the position is found for which the resulting bending moment is the largest. This position is then the critical position for flexure. Figure 7 illustrates this procedure. Note that this procedure only considers the loading side of the load and resistance equation. If the capacity changes, for example because the height of the cross-section changes, or because the reinforcement layout changes, it may be better to determine the critical position as the cross-section with the highest Unity Check (ratio of considered load effect to capacity). When the studied failure mode is shear, it is assumed that the critical position on the slab is at a distance between the face of the support and the face of the first axle of $2.5d_l$, with d_l the effective depth to the longitudinal reinforcement (Lantsoght et al., 2013b).

After the determination of the critical position and the largest bending moment or shear caused by the loads prescribed by the code, the live loads are removed and replaced by the proof loads. For bridges with a small width (maximum three lanes), a single tandem can be used for proof load testing. The name of this tandem is the “proof load tandem.” The proof load tandem is placed at the critical position. Then, the load on the proof load tandem is increased until the same sectional moment or shear is found as with the loads prescribed by the code in the load combination prescribed by the RBK. The load factor on the proof load is $\gamma_{proof} = 1.0$. For the comparison of the sectional moment, the average moment over a width of $2d_l$ can be used and for the comparison of the sectional shear, the average shear over a width of $4d_l$ can be used (Lantsoght et al., 2013a). For skewed slabs, the critical position for shear of the proof load tandem needs to be taken in the obtuse corner (Cope, 1985).

The last step in the preparation of a proof load test is the determination of the sensor plan. Based on the map of cracks developed during the inspection of the cracks in the field, the cracks for monitoring during the test can be identified. The position of the proof load tandem determines the positions of the deflection measurements. The following minimum measurements are recommended for a proof load test:

- deflection in the longitudinal direction at five positions
- deflection in the transverse direction at three positions
- deflection of the supports on both sides of the tested span
- strain on the bottom of the slab for at least one position (i.e. under the proof load tandem)
- reference strain measurement to evaluate the effect of the environmental conditions
- crack width for at least one existing crack if bending cracks are present prior to the test

Other critical parameters, identified during the inspection, should be measured during the test. Measuring the applied load is necessary. If hydraulic jacks apply the load, load cells can measure its value. Figure 8 shows an example of a sensor plan.

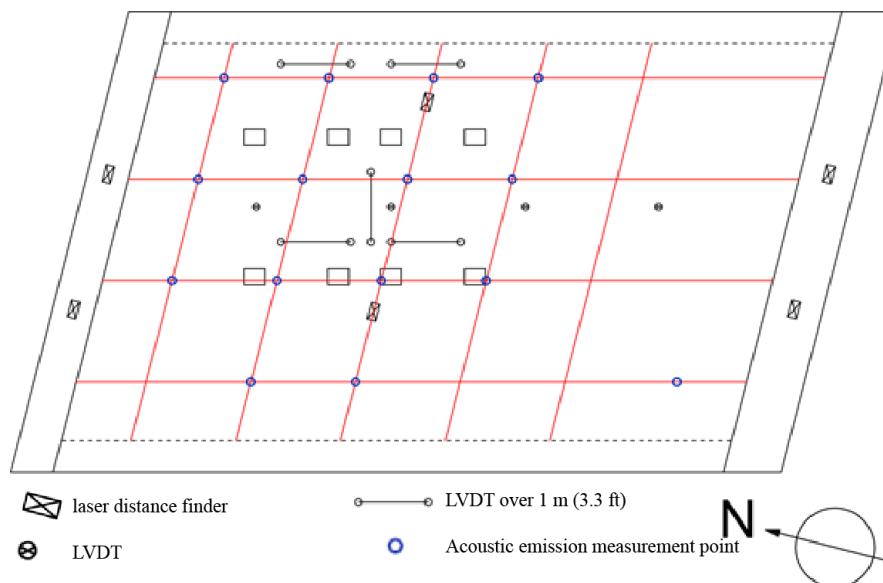


Figure 8 — Overview of resulting sensor plan as used for Viaduct Zijlweg.

For the sensor plan in Figure 8, LVDTs and laser triangulation sensors are used to measure the deflections, and LVDTs are applied over 1 m (3.3 ft) to measure averaged strains on the bottom of the slab. The crack width was measured as well on a longitudinal and transverse crack for the two positions of the proof load tandem that are shown in Figure 8. The positions of the LVDTs measuring crack widths are not shown on the sensor plan as they were drawn on the crack map.

The practical preparations prior to a proof load test focus on ensuring the safety of all parties involved. The safety of all personnel involved with the preparation and execution of the proof load test and its dismantlement is of the utmost importance. The safety of the traveling public and other local parties (press, neighbors ...) should be guaranteed, and these parties should receive clear information about the testing and its effect on the local traffic situation. A communications expert of the road authority should prepare this information. As part of the preparation stage, it is required in the Netherlands to deliver a safety report prior to the field test, and to have at least one safety engineer present on site at all times. The safety engineer educates every person entering the test site on the safety policies prior to starting his/her activities on the test site.

Recommendations for the execution of proof load tests

For the execution of proof load tests, two elements are important:

1. the loading protocol and the method of load application, and
2. the interpretation of the measurements and verification of stop criteria in real-time during the proof load test.



Figure 9 — BELFA vehicle on viaduct Vlijmen Oost.



Figure 10 — Use of steel spreader beam, hydraulic jacks, and counterweights in the test on the second span for the Ruytenschildt Bridge.

The first element is the application of the load and the determination of the loading protocol. The pilot proof load tests considered two load application methods: the use of a loading vehicle, see Figure 9, and the use of a steel spreader beam with counterweights and hydraulic jacks, see Figure 10. Both methods are suitable for proof load testing of reinforced concrete slab bridges, and the final choice for a certain proof load test will depend on practical considerations. The general requirements for the loading system are the following:

- the load must be applied in a safe manner, and sudden large deformations should not lead to collapse of the bridge and the loading system,
- the loading system must be suitable for the selected loading protocol and target proof load,
- it must be possible to keep the magnitude of the load constant,
- the loading system must be suitable for applying a cyclic loading protocol,
- the loading system must have the same wheel prints as the proof load tandem assumed to determine the critical position and target proof load.

The experiments on beams cast in the laboratory resulted in the recommend loading protocol. Early on during the pilot proof load tests, a cyclic loading protocol was chosen, to study different load levels, and to check per load level if the structural response remains the same for the repeated load cycles. If this is not the case, nonlinear behavior is taking place, and continuing the proof load test is not allowed. Figure 11 defines the terms load cycle, load level, and load step.

The loading protocol prescribes at least four load levels:

1. A low load level to check the proper functioning of all sensors.
2. A load level that corresponds to the Serviceability Limit State.
3. An intermediate load level, for example the RBK Disapproval level.
4. The target load level, which is the RBK Usage level for the assessment of existing bridges in the Netherlands, plus 5% extra.

The extra 5% on the RBK Usage level covers local variability of the tested span and the chance that another position may be the weakest spot in the slab. Further statistical analyses are necessary to determine the appropriate extra percentage on the target proof load. Note that for the pilot proof load tests the largest load level was the RBK Design level (plus a few percent extra), as the tests covered all load levels of the RBK. However, in general, a load level higher than the RBK Usage level is not necessary. This safety level is required for all existing highway bridges in the Netherlands.

Per load level, at least three cycles are recommended to check the linearity and reproducibility of the measured response. For load levels 3 and 4, after the Serviceability load level, an extra cycle is added in which the load is increased in small steps and kept constant for about two minutes. While the load is constant at these intermediate levels, the engineer checks the measurements to verify if the load can increase further. Unloading in the cycles does not occur to 0 kN (0 kip), but to a small value of the load to make sure that all sensors and jacks remain activated. The loading speed should be a constant value during loading and unloading, and should be a value between 3 kN/s (0.7 kip/s) and 10 kN/s (2.2 kip/s). Considering these recommendations, Figure 12 shows an example loading protocol.

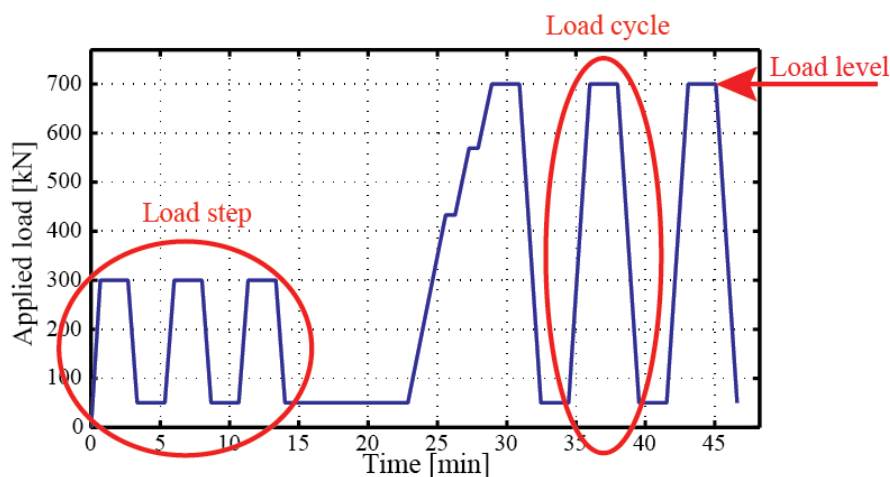


Figure 11 — Definition of load cycle, load level, and load step.

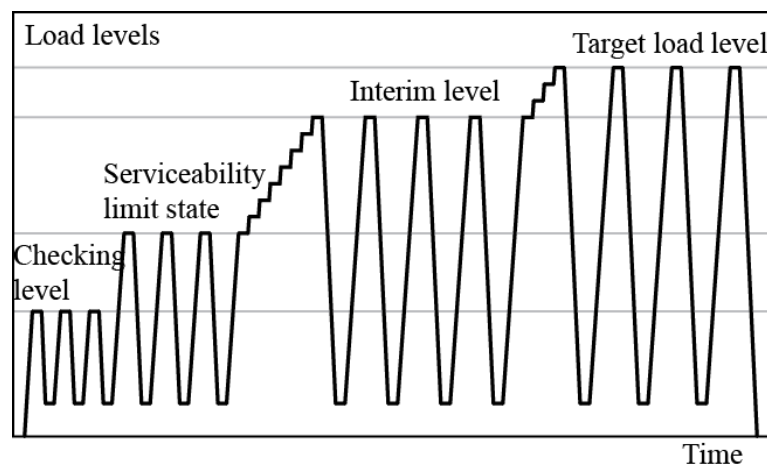


Figure 12 — Resulting recommended loading protocol.

During the proof load test, close monitoring of the structural response is necessary. The measurements should be followed in real-time during the test, and whenever the load is increased, it has to be verified first if no signs of distress are observed. Only after checking all stop criteria and all real-time plots, the engineers following the measurements can communicate to the load operator that they can execute the next load cycle or that they can increase the load. The German guidelines (Deutscher Ausschuss für Stahlbeton, 2000) prescribe stop criteria for proof load testing of plain and reinforced concrete buildings for the failure mode of flexure, and

ACI 437.2M-13 (ACI Committee 437, 2013) prescribes acceptance criteria for proof load testing of concrete buildings for the failure mode of flexure. Based on the pilot proof load tests and lab testing carried out in the Netherlands, a set of stop criteria for flexure and shear are proposed, see Table 2. Further research is necessary on shear-critical elements in the lab to confirm the stop criteria for shear. Moreover, testing of slabs instead of beams in the laboratory is necessary to study the limiting concrete strains and crack widths that can be used for the stop criteria for shear. In the proposal of Table 2, the stop criterion identified with “Concrete strains” is taken from the German guidelines (Deutscher Ausschuss für Stahlbeton, 2000) as:

$$\varepsilon_c < \varepsilon_{c,lim} - \varepsilon_{c0} \quad (1)$$

with ε_c the strain measured in the concrete during proof loading, $\varepsilon_{c,lim}$ a strain of 0.6 ‰ in general, which can be increased to 0.8 ‰ when the concrete compressive strength is larger than 25 MPa (3.6 ksi), and ε_{c0} the analytically determined short-term strain in the concrete caused by the permanent loads that act on the structure before the application of the proof load. At this moment, the stop criterion is the same for the failure mode of shear. However, further research should result in an improved formulation of the strain stop criterion for shear, possibly based on the Critical Shear Displacement Theory (Yang et al., 2016 ; Yang et al., 2017) and more laboratory testing.

Table 2 – Proposed stop criteria, subject to further research, based on Dutch pilot proof load tests and lab testing.

Failure mechanism	Previously cracked in bending moment or not?	
	Uncracked	Cracked
<i>Flexural failure</i>	Concrete strains $w_{max} \leq 0.5 \text{ mm (0.02 in)}$ $w_{res} < 0.3w_{max}$ Stiffness reduction $\leq 25 \%$ Deformation profiles Load-deflection graph	Concrete strains $w_{max} \leq 0.3 \text{ mm (0.01 in)}$ $w_{res} < 0.2w_{max}$ Stiffness reduction $\leq 25 \%$ Deformation profiles Load-deflection graph
<i>Shear failure</i>	Concrete strains $w_{max} \leq 0.4w_{ai}$ Stiffness reduction $\leq 25 \%$ Deformation profiles Load-deflection graph	Concrete strains $w_{max} \leq 0.75w_{ai}$ Stiffness reduction $\leq 25 \%$ Deformation profiles Load-deflection graph

The stop criteria for flexure for the maximum and residual crack widths, w_{max} and w_{res} are taken from the German guidelines (Deutscher Ausschuss für Stahlbeton, 2000). For the case of an element previously uncracked in bending, the crack width and residual crack width of a new crack are followed. For the case of an element with existing bending cracks, the increase in crack width of a crack near the load application is measured. For shear, a stop criteria for crack width is derived based on aggregate interlock theory (Lantsoght, 2017b). Further experimental validation of this proposed stop criterion is required. A crack between the load and the support, in the shear span, should be monitored. For a structure with bending cracks, monitoring an existing crack is possible. If no cracks are present at the start of the test, optical measurements are required to follow the development of cracks and the crack width can only be measured when the crack in the shear span has developed. The value of w_{ai} , which is the crack width for which the shear capacity provided by aggregate interlock drops below the inclined cracking load as determined in the RBK, is calculated as:

$$w_{ai} = \frac{0.03 f_c^{0.56} \left(1 + \rho_s n_e - \sqrt{2 \rho_s n_e + (\rho_s n_e)^2} \right) (978 \Delta_{cr}^2 + 85 \Delta_{cr} - 0.27) R_{ai}}{v_{RBK}} + 0.01 \quad (2)$$

with

$$\Delta_{cr} = \frac{25 d_l}{30610 \phi} + 0.0022 \leq 0.025 \text{ mm} \quad (3)$$

$$v_{RBK} = \max(1.13 k_{slab} k \sqrt{\frac{f_c}{f_y}}; 0.15 k_{slab} k (100 \rho_s f_c)^{1/3}) \quad (4)$$

$$k = 1 + \sqrt{\frac{200}{d_l}} \leq 2 \quad (5)$$

$$R_{ai} = 0.85 \sqrt{\left(\frac{7.2}{f_c - 40} + 1 \right)^2} - 1 + 0.34 \text{ if } f_c > 65 \text{ MPa} \quad (6)$$

with f_c the concrete compressive strength in [MPa], ρ_s the ratio of the longitudinal reinforcement, n_e the ratio of the moduli of elasticity of steel and concrete, Δ_{cr} the critical shear displacement, R_{ai} a correction factor for high-strength concrete with $f_c > 65$ MPa (9.4 ksi), d_l the effective depth to the longitudinal reinforcement in [mm], ϕ the bar diameter in [mm], v_{RBK} the shear capacity that corresponds to the inclined cracking load, $k_{slab} = 1.2$ for slabs supported by line supports and 1.0 for all other cases, k the size effect factor, and f_y the yield strength of the steel in [MPa]. In these expressions, units [MPa] and [mm] should be used.

The next stop criterion considers the stiffness reduction. For this evaluation the stiffness observed in the load-deflection diagram is compared for a given load cycle to the first load cycle. If in a given load cycle, the stiffness is less than 75% of the stiffness in the first load cycle, the proof load test should be terminated.

The deformation profiles that are studied as a stop criterion are the profiles of the deflection and/or the horizontal displacement across the different load levels. If the shape of these profiles changes, changes are taking place in the structure and further loading may not be permitted. An example of these profiles is shown in Figure 13, where it can be seen that at 175 kN (39 kip) the repeatability of the results is not obtained across the three load cycles anymore. The stop criterion would thus be exceeded at a load of 175 kN (39 kip), or 42% of the failure load, which allows for a sufficient margin of safety.

The last stop criterion is based on the load-deflection diagram. During the proof load test, the load-deflection diagram of the applied load and the deflection under the load is followed in real-time. When signs of distress, such as nonlinear behavior, are observed in this diagram, the stop criterion is exceeded and further loading is not permitted.

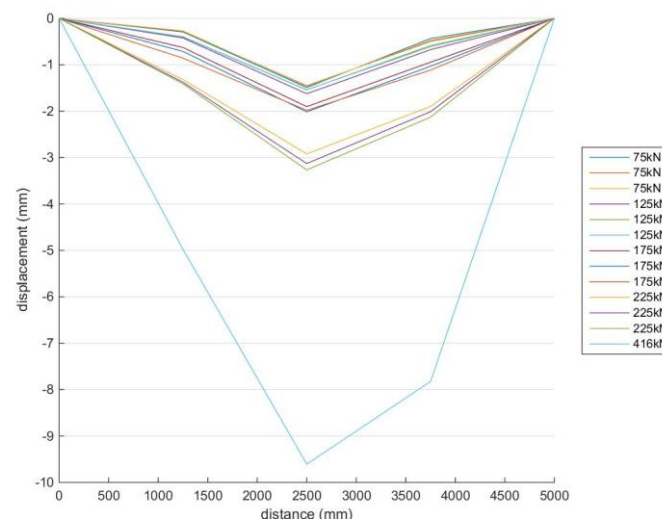


Figure 13 — Deflection profiles measured in RSB02B, a beam sawn from the Ruytenschildt Bridge and tested in the laboratory. Conversion: 1 mm = 0.04 in, 1 kN = 0.225 kip.

Recommendations for the evaluation of proof load tests

A proof load test is successful when the target proof load is applied and no signs for distress are observed during the test and no stop criterion is exceeded. After a successful proof load test, the direct conclusion is that the tested bridge fulfills the code requirements for the factored live loads, provided that the most critical position at the most critical span was tested. In that case, the report about the proof load test contains the post-processed data. In the post-processing stage, the measurements need to be corrected: the deflection measurements of the slab need to be corrected for the deflections of the supports to have the net deflections of the slab, and the strain measurements need to be corrected for the reference strain measurement to exclude the effect of temperature and humidity. Based on these corrected measurements, the verification of the stop criteria is added to the report. The report can include the following graphs:

- The load-deflection diagram showing all cycles, as well as the envelope of the load-deflection diagram. Figure 14 shows an example.
- The measured loading protocol.
- The deflection profiles, also analyzed for the stop criteria, should be plotted for each load cycle.

- The measured crack widths should be plotted as a function of the time and as a function of the applied load.
- The measured strains should be plotted as a function of the time and as a function of the applied load.

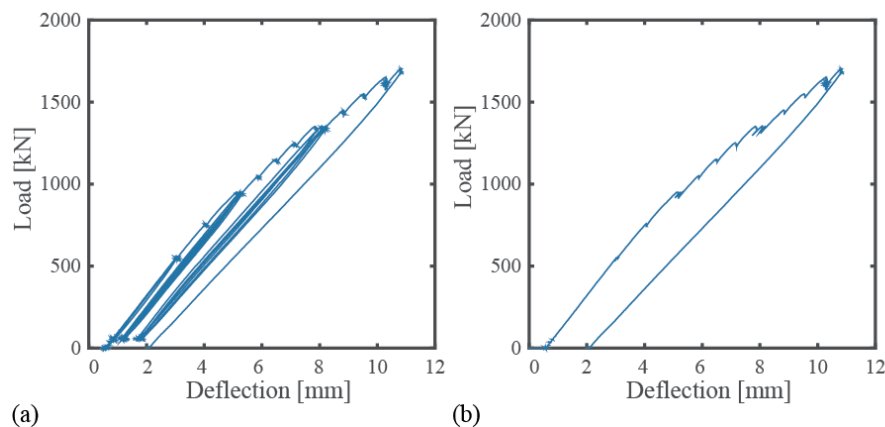


Figure 14 — Load-deflection diagram from Viaduct De Beek (bending moment test), as reported after post-processing: (a) load-deflection diagram showing all cycles; (b) load-deflection envelope. Conversion: 1 mm = 0.04 in, 1 kN = 0.225 kip.

The report should include the total maximum load applied during the proof load test. This value is the measured load on the load cells, plus the weight of the jacks and other elements of the test setup that load the deck directly. A graph showing the measurements of the load at each wheel print (when jacks are used) must show that the load was distributed equally over the wheel prints.

If another span of the bridge can be governing, or if an assessment based on another type of load (for example, a special type of vehicle) is required, then the finite element model that was developed to prepare for the proof load test must be updated with the responses measured during the proof load test. This approach is similar to updating a finite element model after a diagnostic load test (Barker, 2001; Bridge Diagnostics Inc., 2012). The finite element model is altered until the responses in the model correspond reasonably well with the responses measured during the proof load test. This updated model can then be used for the assessment of another span of the bridge, or for applying a different load.

The final report of the proof load test should be submitted to the bridge owner. The owner will make the decisions with regard to the removal of posting or changing of the lane layout based on the recommendations of the report, and takes the full responsibility for these decisions.

DISCUSSION AND FUTURE RESEARCH

Over the past decade, a number of pilot proof load tests have been carried out in the Netherlands, as well as a collapse test. Combined with additional testing of beams in the laboratory, the aim is to develop a guideline for the proof load testing of concrete bridges. At this moment, research resulted in significant steps in this direction, but some uncertainties prevent the publication of such guidelines covering all cases. More research is required before all proof load testing can be transferred to the industry. At this moment, for proof load testing of shear-critical bridges, the involvement of a research-oriented partner that has experience with shear testing is still required. Note that the current recommendations for proof load testing are based on the code requirements for the Netherlands. The application of these recommendations to other countries, such as the USA where AASHTO LRFR governs (AASHTO, 2016), is not straightforward, as the live load model for assessment and the requirements with regard to the target reliability index are different.

For the assessment of reinforced concrete slab bridges, the bridge owner is currently advised to follow the Levels of Assessment. As such, load testing is only considered after rigorous analytical studies based on the available information. Another type of load test that could be interesting, but which is not available yet, is a standard basic test which can be carried out with limited preparation and limited time on-site. In such a test, the application of a certain load and basic measurements should be sufficient to demonstrate that a given bridge can carry the prescribed factored live loads in a safe manner.

A first topic that needs further research and further experiments is the determination of stop criteria for shear. The development of limiting values for the strains and crack widths for the case of an element without flexural cracks and for the case of an element with flexural cracks is necessary. The recommended values should be verified with experimental results. At this moment, only three experiments for shear are available on beams. More experiments, also on slabs, are required to verify the stop criteria for shear.

Another topic that needs further research is the determination of the target proof load based on the concepts of structural reliability (Lantsoght et al., 2017f). It should be studied if the load factors that are used to derive the target proof load can be further reduced. The results of WIM and traffic measurements should be implemented where suitable. For the determination of the probability of failure, it is necessary to describe the probability density functions of the load and resistance. Figure 15 illustrates the effect of proof load testing on the probability density function of the resistance. The description of the probability density function of the load and the resistance requires an estimate of the variability of these properties, for example by defining the coefficient of variation. At this moment, no recommendations for the choice of the coefficient of variation are available. Further statistical analyses are also necessary to determine the extra percentage on the target proof load that corresponds to the RBK Usage level. At this moment, the target proof load plus 5% is applied, to cover local variability of the material properties, but further research should provide statistical support for this practice. Drilling more cores results in a better statistical description of the concrete strength, but statistical analyses are necessary to determine the effect of the distribution of the concrete strength on the proof load test.

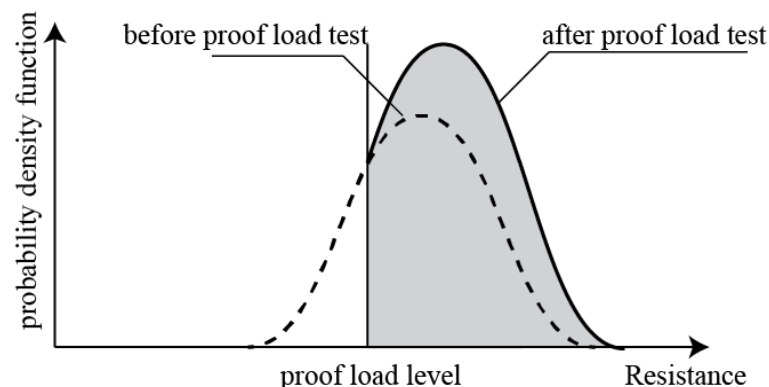


Figure 15 — Effect of proof load testing on probability density function of the resistance.

The calculations that are part of the preparation of a proof load test use the mean material properties. Standard protocols are available in the Netherlands for drilling concrete cores, their transportation, the testing, and the determination of the properties based on statistics (Rijkswaterstaat, 2014). However, for the determination of the properties of steel based on samples taken from an existing bridge, no recommendations are currently available.

The distribution width for the bending moment needs further study. Currently, the industry uses $2d_l$ (based on a rule of thumb). Based on the preparations for proof load testing of Viaduct De Beek, the recommended transverse distribution width for the peak bending moment was determined as 3 m (10 ft). Further research is needed to develop a recommendation for this width.

Currently, the recommendations imply the use of contact sensors. However, when access to the test site makes the application of contact sensors impossible, other measurement techniques become necessary. Additionally, the use of non-contact sensors can speed up the preparations on site, so that the bridge can be closed for a shorter amount of time. Currently, a proof load test takes two days of on-site preparations, one day of testing at two positions, and one day to dismantle all sensors. For this reason, research on the use of digital image correlation and radar interferometry is necessary. The use of a total station may be less suitable for the described proof load tests, since the time required by the total station to measure all points is relatively large. As such, the total station may be too slow to capture the behavior of the structure during the loading branch of a load cycle.

For the extension to wider viaducts, further research is needed. In the pilot proof load tests, only bridges with a width of maximum three lanes were tested. These bridges could be tested with a single proof load tandem. For wider bridges, perhaps more than one proof load tandem should be applied/

At this moment, the assessment of a span that has not been tested is based on updating the finite element model. However, a more direct method is necessary: if the Unity Check in one tested span can be reduced by a certain factor based on the results of the proof load test, what then becomes the reduction of the Unity Check in a span that has not been tested? This reduction should take into account the variability of (material) properties and the uncertainties involved with load testing.

The current research mostly considers reinforced concrete slab bridges. Additional checks and research are necessary to extend these recommendations to other types of reinforced concrete bridges and to prestressed concrete bridges.

SUMMARY AND CONCLUSIONS

A large number of the existing bridges in the Netherlands are approaching the end of their originally devised service life. These bridges require assessment. An initial assessment showed that there are causes for concern with regard to the reinforced concrete slab bridges. Therefore, research was carried out over the past few years to improve the assessment methods for reinforced concrete slabs and slab bridges, especially for shear. When the uncertainties on the capacity are large, analytical methods may not be sufficient, and field testing becomes interesting.

Field testing of bridges contains diagnostic load testing and proof load testing. A diagnostic load test has as its purpose to compare analytical predictions to the structural response in the field, to improve the analytical models and obtain a better assessment. In a proof load test, a load representative of the factored live loads is applied to the bridge. If the bridge can withstand these loads without signs of distress, the bridge fulfills the requirements of the code. Proof load testing can be suitable for reinforced concrete slab bridges when the uncertainties about the effect of material deterioration (such as alkali-silica reaction) on the capacity are large.

Pilot proof load tests since 2007 studied the possibilities for proof load testing on concrete bridges in the Netherlands, with a focus on reinforced concrete slab bridges. Viaduct Heidijk, viaduct Medemblik, viaduct Vlijmen Oost, the Halvemaans Bridge, viaduct Zijlweg, and viaduct De Beek all have been subjected to a proof load test. For the execution of these tests, in some cases a loading vehicle, the BELFA from Germany, was used, and in other cases a system with a steel spreader beam, counterweights, and hydraulic jacks was used. The collapse tests on the Ruytenschildt Bridge add valuable information to the campaign of proof load tests. The loading protocol and the stop criteria were studied in more detail with laboratory testing. Three beams sawn from the Ruytenschildt Bridge and two beams with plain bars cast in the lab were tested to failure, resulting in nine laboratory tests.

With the experience and insights from the pilot proof load tests, the collapse test, and the beam tests in the lab, recommendations have been formulated for the preparation, execution, and evaluation of proof load tests on reinforced concrete slab bridges. For the preparation of proof load testing, it is important to see if a proof load test is the right solution to answer the questions for the bridge under consideration. If this is the case, an inspection of the site and preparatory calculations of the capacity are required. In this stage, a linear finite element model determines the target proof load and its critical position. This information leads to the development of a sensor plan.

The pilot proof load tests and laboratory tests on beams resulted in a recommended loading protocol. During the proof load test, analysis in real time of the measurements and stop criteria is necessary. The currently available stop criteria are based on the information from the pilot proof load tests, the collapse test, and the laboratory testing. However, further experimental evidence is necessary to extend and to validate the proposed stop criteria for shear.

The evaluation stage of the proof load test consists of writing the report that contains the graphs of the most important structural responses measured during the field test. If the bridge can carry the target proof load without signs of distress, the proof load test is successful and the bridge fulfills the code requirements.

At this moment, further research with regard to the target proof load, the stop criteria for shear, the use of non-contact sensors, the transverse distribution width for flexure, the interpretation of material testing for steel samples, the extrapolation of results to other spans, the extension to wider viaducts, and the extrapolation of these recommendations to other reinforced concrete bridge types and prestressed concrete bridges is required before guidelines for proof load testing can be published and used by the industry in the Netherlands.

ACKNOWLEDGEMENTS

The authors wish to express their gratitude and sincere appreciation to the Dutch Ministry of Infrastructure and the Environment (Rijkswaterstaat) for financing this research work. The contributions and help during the pilot proof load tests of our current and former colleagues Albert Bosman, Sebastiaan Ensink, Sonja Fennis, Rutger Koekkoek, Patrick Van Hemert and Yuguang Yang of Delft University of Technology are gratefully acknowledged. The fruitful discussions with Frank Linthorst and Danny den Boef of Witteveen+Bos, responsible for logistics and safety during the pilot proof load tests, and with Otto Illing and the late Chris Huissen of Mammoet, responsible for applying the load during the pilot proof load tests, are also acknowledged.

REFERENCES

AASHTO, 2016, "The manual for bridge evaluation with 2016 interim revisions," 2nd ed. American Association of State Highway and Transportation Officials; Washington, D.C.

ACI Committee 437, 2013, "Code Requirements for Load Testing of Existing Concrete Structures (ACI 437.2M-13) and Commentary ", Farmington Hills, MA, 24 pp.

- Aguilar, C. V., Jáuregui, D. V., Newton, C. M., Weldon, B. D. and Cortez, T. M., 2015, "Load Rating a Prestressed Concrete Double-Tee Beam Bridge without Plans by Proof Testing," *Transportation Research Board Annual Compendium of Papers*, Washington DC, pp. 19.
- Anay, R., Cortez, T. M., Jáuregui, D. V., ElBatanouny, M. K. and Ziehl, P., 2016, "On-Site Acoustic-Emission Monitoring for Assessment of a Prestressed Concrete Double-Tee-Beam Bridge without Plans," *Journal of Performance of Constructed Facilities*, V. 30, No. 4.
- Arangjelovski, T., Gramatikov, K. and Docevska, M., 2015, "Assessment of damaged timber structures using proof load test – Experience from case studies," *Construction and Building Materials*, V. 101, Part 2, 12/30/, pp. 1271-1277.
- Barker, M. G., 2001, "Quantifying Field-Test Behavior for Rating Steel Girder Bridges," *Journal of Bridge Engineering*, V. 6, No. 4, pp. 254-261.
- Belletti, B., Hendriks, M. A. N., den Uijl, J. A. and Damoni, C., 2010, "Developing standardized guidelines for safety assessment of shear-critical rc beams based on nonlinear finite element modeling," *3rd fib International Congress*, Washington DC, pp. 13.
- Belletti, B., Damoni, c., Hendriks, M. A. N. and Den Uijl, J. A., 2013, "Nonlinear finite element analyses of reinforced concrete slabs: comparison of safety formats," *VIII International Conference on Fracture Mechanics of concrete and Concrete Structures, FraMCoS-8*, Van Mier, J. G. M., Ruiz, G., Andrade, C., Yu, R. C. and Zhang, X. X., eds., pp. 12.
- Belletti, B., Pimentel, M., Scolari, M. and Walraven, J. C., 2015, "Safety assessment of punching shear failure according to the level of approximation approach," *Structural Concrete*, V. 16, No. 3, pp. 366-380.
- Bretschneider, N., Fiedler, L., Kappahn, G. and Slowik, V., 2012, "Technical possibilities for load tests of concrete and masonry bridges," *Bautechnik*, V. 89, No. 2, pp. 102-110 (in German).
- Bridge Diagnostics Inc., 2012, "Integrated Approach to Load Testing," 44 pp.
- Casas, J. R. and Gómez, J. D., 2013, "Load Rating of Highway Bridges by Proof-loading," *KSCE Journal of Civil Engineering*, V. 17, No. 3, pp. 556-567.
- CEN, 2002, "Eurocode – Basis of structural design, NEN-EN 1990:2002 ", Comité Européen de Normalisation, Brussels, Belgium, 103 pp.
- CEN, 2003, "Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges, NEN-EN 1991-2:2003," Comité Européen de Normalisation, Brussels, Belgium, 168 pp.
- CEN, 2005, "Eurocode 2: Design of Concrete Structures - Part 1-1 General Rules and Rules for Buildings. NEN-EN 1992-1-1:2005," Comité Européen de Normalisation, Brussels, Belgium, 229 pp.
- Cochet, D., Corfdir, P., Delfosse, G., Jaffre, Y., Kretz, T., Lacoste, G., Lefaucheur, D., Khac, V. L. and Prat, M., 2004, "Load tests on highway bridges and pedestrian bridges (in French)," Setra - Service d'Etudes techniques des routes et autoroutes, Bagneux-Cedex, France.
- Code Committee 351001, 1995, "NEN 6720 Technical Foundations for Building Codes, Concrete provisions TGB 1990 - Structural requirements and calculation methods (VBC 1995) (in Dutch)," Civil engineering center for research and regulation, Dutch Normalization Institute, Delft, The Netherlands, 245 pp.
- Cope, R. J., 1985, "Flexural Shear Failure of Reinforced-Concrete Slab Bridges," *Proceedings of the Institution of Civil Engineers Part 2-Research and Theory*, V. 79, No. SEP, pp. 559-583.
- den Uijl, J. A. and Kaptijn, N., 2004, "Structural consequences of ASR: an example on shear capacity," *Heron*, V. 47, No. 2, pp. 1-13.
- Deutscher Ausschuss für Stahlbeton, 2000, "DAfStb-Guideline: Load tests on concrete structures (in German)," Deutscher Ausschuss für Stahlbeton, 7 pp.
- Dieteren, G. G. A. and den Uijl, J. A., 2009, "Evaluation proof load test Heidijk (in Dutch)," V. 2008-DWARS-MOLO, TNO Bouw en Ondergrond / TU Delft, 70 pp.
- Faber, M. H., Val, D. V. and Stewart, M. G., 2000, "Proof load testing for bridge assessment and upgrading," *Engineering Structures*, V. 22, pp. 1677-1689.
- Fennis, S. A. A. M. and Hordijk, D. A., 2014, "Proof loading Halvemaans Bridge Alkmaar (in Dutch)," Stevin Report 25.5-14-05, Delft University of Technology, Delft, The Netherlands, 72 pp.
- Fennis, S. A. A. M., van Hemert, P., Hordijk, D. and de Boer, A., 2014, "Proof loading Vlijmen-Oost; Research on assessment method for existing structures (in Dutch)," *Cement*, V. 5, pp. 40-45.
- fib, 2012, "Model code 2010: final draft," International Federation for Structural Concrete; Lausanne, 676 pp.
- Frýba, L. and Pirner, M., 2001, "Load tests and modal analysis of bridges," *Engineering Structures*, V. 23, No. 1, pp. 102-109.
- Fu, G., Pezze III, F. P. and Alampalli, S., 1997, "Diagnostic Load Testing for Bridge Load Rating," *Transportation Research Record*, V. 1594, pp. 125-133.
- JCSS, 2001, "JCSS Probabilistic Model Code - Part 3: Resistance Models".

Koekkoek, R. T., Lantsoght, E. O. L. and Hordijk, D. A., 2015, "Proof loading of the ASR-affected viaduct Zijlweg over highway A59," Stevin Report nr. 25.5-15-08, Delft University of Technology, Delft, The Netherlands, 180 pp.

Koekkoek, R. T., Lantsoght, E. O. L., Yang, Y. and Hordijk, D. A., 2016, "Analysis report for the assessment of Viaduct De Beek by Proof Loading," Stevin Report 25.5-16-01, Delft University of Technology, Delft, The Netherlands, 125 pp.

Lantsoght, E., 2016, "Literature review on load testing," Stevin Report nr. 25.5-16-07, Delft University of Technology, 102 pp.

Lantsoght, E., van der Veen, C. and de Boer, A., 2016a, "Shear and moment capacity of the Ruytenschildt bridge," *IABMAS 2016*, pp. 8.

Lantsoght, E., Yang, Y., van der Veen, C., de Boer, A. and Hordijk, D., 2016b, "Ruytenschildt Bridge: field and laboratory testing," *Engineering Structures*, V. 128, No. december, pp. 111-123.

Lantsoght, E., 2017a, "Measurements of Ruytenschildt Bridge test and analysis for stop criteria," V. Stevin Report nr. 25.5-17-02, Delft University of Technology, 60 pp.

Lantsoght, E., 2017b, "Development of a stop criterion for shear based on aggregate interlock," V. Stevin Report nr. 25.5-17-09, Delft University of Technology, 33 pp.

Lantsoght, E. O. L., de Boer, A., Van der Veen, C. and Walraven, J. C., 2013a, "Peak shear stress distribution in finite element models of concrete slabs," *Research and Applications in Structural Engineering, Mechanics and Computation*, Zingoni, A., ed. Cape Town, South Africa, pp. 475-480.

Lantsoght, E. O. L., van der Veen, C., de Boer, A. and Walraven, J. C., 2013b, "Recommendations for the Shear Assessment of Reinforced Concrete Slab Bridges from Experiments " *Structural Engineering International*, V. 23, No. 4, pp. 418-426.

Lantsoght, E. O. L., van der Veen, C. and Walraven, J. C., 2013c, "Shear in One-way Slabs under a Concentrated Load close to the support," *ACI Structural Journal*, V. 110, No. 2, pp. 275-284.

Lantsoght, E. O. L., De Boer, A. and van der Veen, C., 2014a, "Determination of distribution width for shear stresses at support in reinforced concrete slab bridges," *EuroC*, St. Anton am Arlberg, Austria, pp. 9

Lantsoght, E. O. L., van der Veen, C., De Boer, A. and Walraven, J., 2014b, "Influence of Width on Shear Capacity of Reinforced Concrete Members," *ACI Structural Journal*, V. 111, No. 6, pp. 1441-1450.

Lantsoght, E. O. L., van der Veen, C., de Boer, A. and Walraven, J., 2015a, "One-way slabs subjected to combination of loads failing in shear," *ACI Structural Journal*, V. 112, No. 4, pp. 417-426.

Lantsoght, E. O. L., Van der Veen, C., de Boer, A. and Walraven, J. C., 2015b, "Proposal for the extension of the Eurocode shear formula for one-way slabs under concentrated loads," *Engineering Structures*, V. 95, pp. 16-24.

Lantsoght, E. O. L., van der Veen, C., Walraven, J. and de Boer, A., 2015c, "Experimental investigation on shear capacity of reinforced concrete slabs with plain bars and slabs on elastomeric bearings," *Engineering Structures*, V. 103, pp. 1-14.

Lantsoght, E. O. L., van der Veen, C. and de Boer, A., 2016c, "Shear capacity of the Ruytenschildt Bridge," *fib Symposium 2016, Performance-based approaches for concrete structures*, Beushausen, H., ed. Cape Town, South Africa, pp. 10.

Lantsoght, E. O. L., van der Veen, C., de Boer, A. and Alexander, S., 2016d, "Bridging the gap between one-way and two-way shear in slabs," *ACI SP International Punching Symposium*, pp. 20.

Lantsoght, E. O. L., van der Veen, C., de Boer, A. and Hordijk, D. A., 2016e, "Probabilistic prediction of the failure mode of the Ruytenschildt Bridge," *Engineering Structures*, V. 127, 11/15/, pp. 549-558.

Lantsoght, E. O. L., van der Veen, C., de Boer, A. and Walraven, J., 2016f, "Using Eurocodes and AASHTO for assessing shear in slab bridges," *Proceedings of the Institution of Civil Engineers – Bridge Engineering*, V. 169, No. BE4, pp. 285-297.

Lantsoght, E. O. L., Yang, Y., Tersteeg, R. H. D., van der Veen, C. and de Boer, A., 2016g, "Development of Stop Criteria for Proof Loading," *IALCCE 2016*, Delft, The Netherlands, pp. 8 pp.

Lantsoght, E. O. L., 2017c, "Beams from Ruytenschildt Bridge: Analysis of stop criteria," Stevin Report 25.5-17-05, Delft University of Technology, 77 pp.

Lantsoght, E. O. L., De Boer, A. and Van der Veen, C., 2017a, "Levels of Approximation for the shear assessment of reinforced concrete slab bridges," *Structural Concrete*, V. 18, pp. 143-152.

Lantsoght, E. O. L., Koekkoek, R. T., Hordijk, D. A. and De Boer, A., 2017b, "Towards standardization of proof load testing: pilot test on viaduct Zijlweg," *Structure and Infrastructure Engineering*.

Lantsoght, E. O. L., van der Veen, C. and de Boer, A., 2017c, "Extended Strip Model for slabs subjected to load combinations," *Engineering Structures*, V. 145, pp. 60-69.

Lantsoght, E. O. L., van der Veen, C., de Boer, A. and Alexander, S. D. B., 2017d, "Extended Strip Model for Slabs under Concentrated Loads," *ACI Structural Journal*, V. 114, No. 2, pp. 565-574.

- Lantsoght, E. O. L., Van der Veen, C., De Boer, A. and Hordijk, D. A., 2017e, "Collapse test and moment capacity of the Ruytenschildt Reinforced Concrete Slab Bridge " *Structure and Infrastructure Engineering*, V. 13, No. 9, pp. 1130-1145.
- Lantsoght, E. O. L., van der Veen, C., Hordijk, D. A. and de Boer, A., 2017f, "Reliability index after proof load testing: viaduct De Beek," *ESREL 2017*, Protoroz, Slovenia, pp. 8.
- Lantsoght, E. O. L., Yang, Y., van der Veen, C., de Boer, A. and Hordijk, D. A., 2017g, "Beam experiments on acceptance criteria for bridge load tests," *ACI Structural Journal*, V. 114, No. 4, pp. 1031-1041.
- Lantsoght, E. O. L., van der Veen, C., Hordijk, D. A. and de Boer, A., 2017h, "State-of-the-art on load testing of concrete bridges," *Engineering Structures*, V. 150, pp. 231-241.
- Lantsoght, E. O. L., Koekkoek, R. T., van der Veen, C., Hordijk, D. A. and de Boer, A., in press-a, "Case study: Pilot proof load test on viaduct De Beek," *Journal of Bridge Engineering*.
- Lantsoght, E. O. L., Van der Veen, C., De Boer, A. and Hordijk, D. A., in press-b, "Proof load testing of reinforced concrete slab bridges in the Netherlands," *Structural Concrete*, pp. 29.
- Matta, F., Bastianini, F., Galati, N., Casadei, P. and Nanni, A., 2008, "Distributed Strain Measurement in Steel Bridge with Fiber Optic Sensors: Validation through Diagnostic Load Test," *Journal of Performance of Constructed Facilities*, V. 22, No. 4, pp. 264-273.
- Moses, F., Lebet, J. P. and Bez, R., 1994, "Applications of field testing to bridge evaluation," *Journal of Structural Engineering-ASCE*, V. 120, No. 6, Jun, pp. 1745-1762.
- Olaszek, P., Świt, G. and Casas, J. R., 2012, "Proof load testing supported by acoustic emission. An example of application," *IABMAS 2012*.
- Olaszek, P., Lagoda, M. and Casas, J. R., 2014, "Diagnostic load testing and assessment of existing bridges: examples of application," *Structure and Infrastructure Engineering*, V. 10, No. 6, Jun 3, pp. 834-842.
- PN-S-10040/99, 1999, "Bridges. Concrete, reinforced concrete and prestressed structures. Requirements and testing. (in Polish)."
- Rijkswaterstaat, 2013, "Guidelines Assessment Bridges - assessment of structural safety of an existing bridge at reconstruction, usage and disapproval (in Dutch), RTD 1006:2013 1.1," 117 pp.
- Rijkswaterstaat, 2014, "Guidelines for coring, transporting, and testing of concrete cores and the determination of the compressive and splitting tensile strength (in Dutch)," RTD 1021:2014, Rijkswaterstaat, 29 pp.
- Rijkswaterstaat, 2016, "Guidelines for Nonlinear Finite Element Analysis of Concrete Structures," *RTD 1016:2012*, 65 pp.
- Russo, F. M., Wipf, T. J. and Klaiber, F. W., 2000, "Diagnostic Load Tests of a Prestressed Concrete Bridge Damaged by Overheight Vehicle Impact," *Transportation Research Record*, V. 1696, pp. 103-110.
- Sanayei, M., Reiff, A. J., Brenner, B. R. and Imbaro, G. R., 2016, "Load Rating of a Fully Instrumented Bridge: Comparison of LRFR Approaches," *Journal of Performance of Constructed Facilities*, V. 2016, No. 30, pp. 2.
- Saraf, V. K., Nowak, A. S. and Till, R., 1996, "Proof load testing of bridges," *Probabilistic Mechanics & Structural Reliability: Proceedings of the Seventh Specialty Conference*, Frangopol, D. M. and Grigoriu, M. D., eds., Worcester, MA, USA, pp. 526-529.
- Stewart, M. G. and Val, D. V., 1999, "Role of load history in reliability-based decision analysis of aging bridges," *Journal of Structural Engineering-ASCE*, V. 125, No. 7, pp. 776-783.
- Velázquez, B. M., Yura, J. A., Frank, K. H., Kreger, M. E. and Wood, S. L., 2000, "Diagnostic load tests of a reinforced concrete pan-girder bridge," V. Research Report 7-2986-2, The University of Texas at Austin, Austin, TX, USA, 121 pp.
- Vergoossen, R., Naaktgeboren, M., 't Hart, M., De Boer, A. and Van Vugt, E., 2013, "Quick Scan on Shear in Existing Slab Type Viaducts," *International IABSE Conference, Assessment, Upgrading and Refurbishment of Infrastructures*, Rotterdam, The Netherlands, pp. 8.
- Vrouwenfelder, T., Holicky, M. and Markova, J., 2002, "JCSS Probabilistic Model Code Example Applications," 19 pp.
- Yang, Y., 2015, "Experimental Studies on the Structural Behaviours of Beams from Ruytenschildt Bridge," Stevin Report 25.5-15-09, Delft University of Technology, Delft, 76 pp.
- Yang, Y., Den Uijl, J. A. and Walraven, J., 2016, "The Critical Shear Displacement theory: on the way to extending the scope of shear design and assessment for members without shear reinforcement," *Structural Concrete*, V. 17, No. 5, pp. 790-798.
- Yang, Y., Walraven, J. and den Uijl, J. A., 2017, "Shear Behavior of Reinforced Concrete Beams without Transverse Reinforcement Based on Critical Shear Displacement," *Journal of Structural Engineering*, V. 143, No. 1.

LOAD TESTING OF HIGHLY SKEWED CONCRETE BRIDGES

Mauricio Diaz Arancibia and Pinar Okumus

Abstract: Recurrent service problems and uncertainties in load distribution have been frequently reported by Departments of Transportation for skewed bridges. Service problems, such as deck cracking or excessive bridge racking can lead to bridge deterioration, and indicate the need of a better understanding of the structural response of high skew bridges to service loading. This paper presents the instrumentation and load testing of a three-span, medium span length, prestressed concrete bridge with 64° of skew to understand service, analysis and design problems associated with skew.

The instrumentation plan for the bridge was developed based on service problems observed in concrete bridges with high skew such as deck cracking and displacements, as reported by the literature and by regular bridge inspections. Complete understanding of skew related responses required both short-term testing and long-term load monitoring. Structural responses of the key areas of the bridge to live and temperature loads and shrinkage were measured. The effects of certain bridge details on live load distribution were determined using finite element models validated through short-term load testing data. The evolution and magnitude of bearing movements and deck strains were captured for long periods of opposite thermal tendencies.

Keywords: bearing displacements, deck cracking, load distribution, load testing, shrinkage, skew, temperature

INTRODUCTION

Highly skewed bridges experience several construction and service performance problems. They have regularly been reported by State Departments of Transportation, and have also been described in the published literature¹. Skew-related problems are observed, even at the component level. For instance, double-tee prestressed concrete bridge girders suffer significant end cracking during prestress release due to torsion and formwork restraint caused by the skewed geometry². At the system level, bridge movements and deck cracking are likely the ones causing the most detrimental effects on the structure. Temperature loads and shrinkage on skewed bridges result in superstructure horizontal movements or a tendency to rotate, which typically leads to premature failure of expansion joints and bearings, and misalignment of parapets between bridge and approach slab. On the other hand, characteristic diagonal acute corner deck cracking observed on skewed bridges may cause corrosion of the deck reinforcement and early deck replacement. In addition to these performance problems, the geometry of skewed bridges yields to modified load paths that can change how superimposed loads are distributed to structural components.

Bridge load testing is an excellent means to evaluate skewed bridge behavior. In general, load testing has been used to rate bridge load carrying capacity under short-term loading (load rating), understand and assess bridge response under long-term loading, or to generate data to validate numerical models that are later employed to study bridge performance. For instance, in 2005, Hag-Elsafi and Kunin³ developed a live-load testing program for load rating of a single-span, non-skewed, 70-ft long bridge with post-tensioned bulb-T beams built in 1961. A test-based rating was imperative since the bridge had no plans on record, leading to uncertainties on variables that affect load carrying capacity, which in turn prevented an analytical rating. Another example is the investigation on live load shear distribution of prestressed concrete girder bridges performed by French et al.⁴ They conducted full-scale laboratory and field bridge live load tests. Laboratory tests on a non-skewed bridge provided insights on shear distribution during the elastic range, and up to ultimate shear failure. Field tests were performed on six prestressed concrete girder bridges to determine shear distribution under service conditions. The effects of skew on shear distribution were also investigated, and tested for a bridge with 30° of skew. Data from load testing were used to validate finite element models for parametric studies on shear distribution. Testing showed that the shear redistributes prior to ultimate failure, and therefore, the use of linear elastic distribution factors is conservative when evaluating the shear ultimate capacity of the bridge. Parametric studies using finite element models, which did not consider the contribution of barriers, diaphragms and skew, showed that girder spacing, deck thickness and span length have the most significant effects on linear elastic shear distribution.

Problems on highly skewed bridges were also investigated using load testing. Fu et al.⁵ studied the causes of deck corner cracking on skewed bridges. They instrumented the deck of two girder type bridges. One of the bridges had steel I-beams and 49° of skew, while the other had AASHTO prestressed concrete beams and 46° of skew on the instrumented span. Deck strains and temperature, and ambient temperature and humidity were recorded for short-term loading that consisted of controlled truck loading, and for long-term loading, such as temperature and shrinkage. Using the data together with finite element models, they found that deck corner cracking is caused by thermal loading and shrinkage during concrete hydration, and that fatigue loading caused by traffic may widen the cracks. Huang et al.⁶ investigated transverse bending moment distribution on highly skewed steel girder bridges. They instrumented and load tested a two-span, deck-on-steel bridge with a skew angle of 60°. Load distribution was evaluated by comparing load test results with live load distribution factors obtained from AASHTO. The effect of deck stiffness and diaphragms on load distribution was explored using a finite element model validated with test results. Researchers concluded that for bridges similar to the one tested, AASHTO distribution factors are conservative for positive moments and un-conservative for negative moments.

Load testing was employed for this study to understand the behavior of highly skewed medium-span prestressed concrete bridges. The tested bridge was selected due to its high skew and distinctive mixed support fixity arrangement over the piers; a feature designed to minimize horizontal lateral movements. The investigation addressed the most common problems with skewed bridges: load distribution, superstructure horizontal movements and deck corner cracking, using short-term and long-term loading schemes. Short-term loading consisted of truck loading under controlled conditions and was used to generate shear and bending strain data. These data were used to validate finite element models employed for parametric studies dealing with the effects of certain bridge details on shear and bending live load distribution to girders. Long-term loading was composed of temperature loads, shrinkage and traffic load. Under these loads, bearing horizontal movements and deck strains at critical locations were continuously monitored for approximately 5 and 7 months, respectively, to gain a better understanding of how highly skewed bridges move and how deck cracking develops.

BRIDGE DESCRIPTION

The tested structure is identified as bridge B-40-870 in the Highway Structures Information System of Wisconsin Department of Transportation (WisDOT)⁷. It carries the Interstate Highway 94 East-Bound traffic over the Hank Aaron State Trail in Milwaukee County, Wisconsin. All three spans are deck-on-prestressed concrete-girders, continuous for live load and superimposed loads. The bridge has a skew of 64°. General characteristics and geometry of the bridge are presented in Table 1 and Figure 1, respectively.

Table 1—Bridge B-40-870 general features

Skew	Span lengths	Ave. deck width	Girder type	Abutment type	Bearing types	Year built
64°	109.2 ft – 125.4 ft – 85.8 ft [33.3 m – 38.2 m – 26.2 m]	94.9 ft [28.9 m]	54 in. wide flanged prestressed girder	Semi-retaining	Elastomeric (abutments and piers), metallic (piers)	2016

The bridge has fifteen prestressed concrete girders on each span, with varying or flared spacing due to a slight horizontal curvature of 4300 ft [1311 m] of radius. The cast-in-place deck is 8-in [203-mm] thick, fully composite with girders by means of #4 [#13] shear connectors at stirrup spacing. Transversely, girders are connected through steel diaphragms along spans, and through full depth cast-in-place concrete diaphragms over piers. The concrete diaphragms are not in contact with pier caps or girder bearing pads. The specified compressive strength of concrete for deck and substructure is 4000 psi [27.6 MPa]. Pretensioned concrete girders have a specified concrete strength of 8000 psi [55.2 MPa], with 270000 psi [1861.6 MPa] ultimate strength strands. Mild steel reinforcement has a specified yield strength of 60000 psi [413.7 MPa].

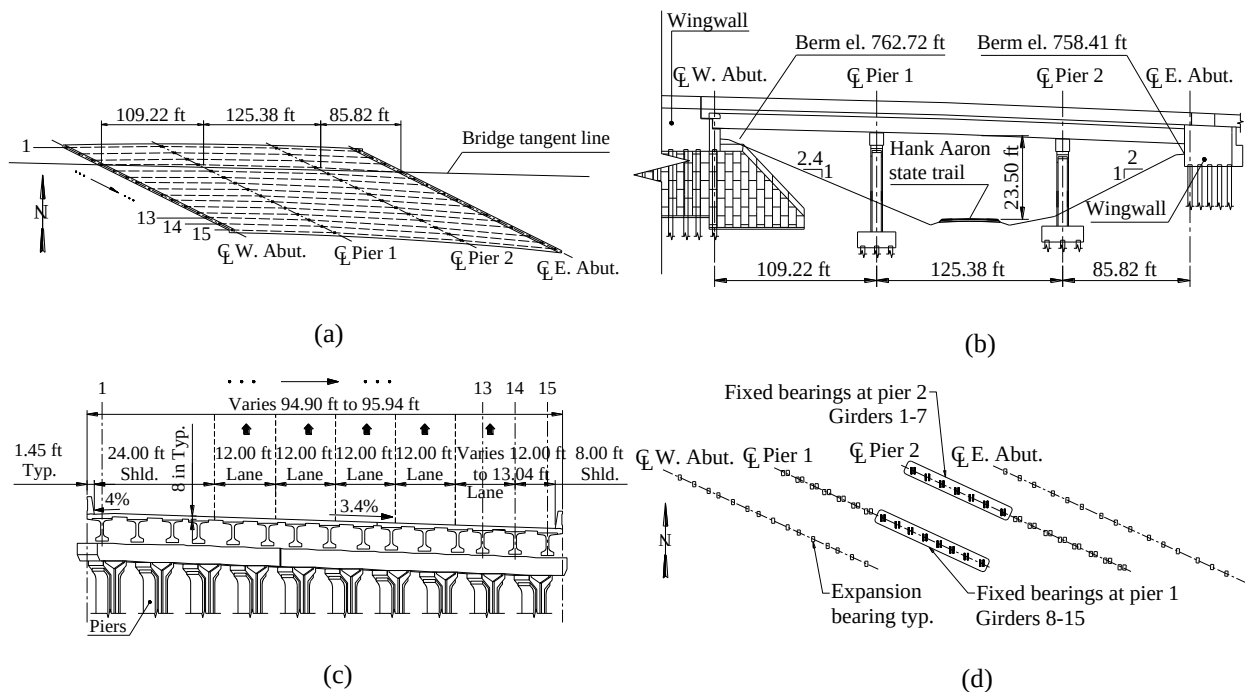


Figure 1—Bridge geometry: (a) plan view; (b) elevation (looking north); (c) cross section (looking east); (d) bearing layout shown on plan view [1 ft = 0.3048 m]

The bridge has a unique configuration of bearing fixity. Bearings over each pier are a mixed arrangement of both expansion and fixed bearings. Bearings over the abutments are expansion bearings only, as shown in Figure 1d. This special bearing arrangement was designed to keep thermal and shrinkage displacements at expansion joints and at parapet expansion joint cover plates under the limits specified on section 28.3.4 of the WisDOT Bridge Manual (BM)⁸.

LOAD TEST SET-UP

The load test was conducted to understand the short- and long- term behavior. Short-term results, shear and bending strains on girders, were generated by loading the bridge with trucks moving at crawl speed. These results were used for finite element model validation. Long-term data consist of bearing movements and deck strains at critical locations for a skewed bridge¹. Long-term loading includes temperature, shrinkage and traffic on the bridge under service.

Short-term loading

Loading to collect short-term data consisted of two dump trucks, positioned in series or parallel, moving at crawl speeds (i.e., less than 5 mph [8 km/h]) to simulate a pseudo-static load. Dimensions and axle weights of the two trucks used are shown on Figure 2. The middle and rear axles were assumed to equally share the measured weight of both axles together.

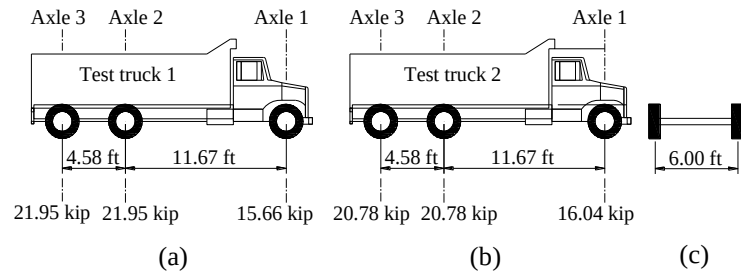


Figure 2—Load test trucks dimensions and axle weights: (a) test truck 1; (b) test truck 2; (c) front view showing axle width [1 ft = 0.3048 m, 1 kip = 4.4482 kN]

The short-term test had four load cases (Figure 3). In all load cases, the trucks moved from west to east. Each load case was run twice to ensure the results were repeatable. During some runs, the trucks came to a full stop at predefined positions to achieve actual static loading by avoiding any dynamic effects coming from motion of the trucks, surface irregularities or bumps on the road. Table 2 presents the details of truck stops.

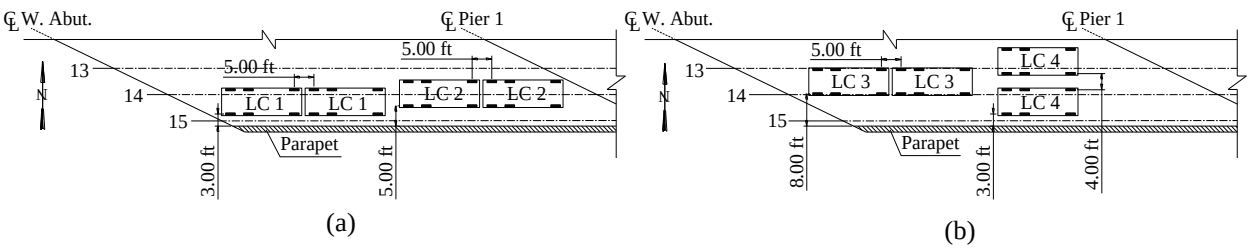


Figure 3—Load test load cases: (a) load cases 1 and 2; (b) load cases 3 and 4 [1 ft = 0.3048 m]

Table 2—Truck stops per load case

Load case	Run	Stops		
		Span 1 at mid-span (west)	Span 2 at mid-span	Span 3 at mid-span (east)
1	1	Yes	Yes	-
	2	-	-	-
2	1	Yes	Yes	-
	2	-	-	-
3	1	-	Yes	-
	2	Yes	Yes	Yes
4	1	Yes	Yes	-
	2	-	-	-

Instrumentation for short-term loading

Sensors for short-term loading consisted of uniaxial resistance strain gages attached on the surface of prestressed concrete girders on the west span (span 1). Strain gages measuring bending strains were installed in pairs on the bottom face of the three girders (girders 13, 14 and 15 in Figures 1a and 4) near the south side, and at three points along their span (at 15 ft [4.6 m], 40 ft [12.2 m] and 80 ft [24.4 m] from the west end of girders). Two gages were used at each gage location for redundancy against potentially malfunctioning sensors.

A second set of strain gages was used on the west end of the same girders (girders 13, 14 and 15) to measure shear strains at girder ends. These sensors are located near the obtuse corner where maximum shear response is expected on a skewed bridge. The gages were placed at mid-height of girder webs, where strains reach measurable values (i.e., close to the location of maximum shear strain). For this purpose, three strain gages were arranged in a 45-degree rosette layout, with the central one aligned with the horizontal axis of the girder. This strain gage configuration allows strains in any direction to be determined, in addition to the vertical shear strain determined only from the diagonally-oriented gages. The layout of bending and shear strain gages is presented in Figure 4 shown on plan view.

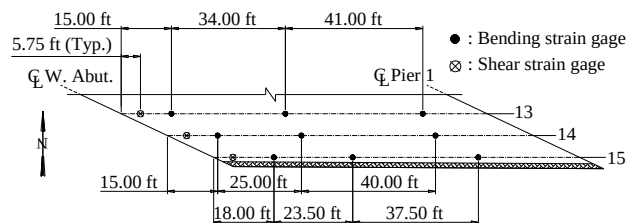


Figure 4—Bending and shear strain gage layout shown on plan view of south-west corner [1 ft = 0.3048 m]

Instrumentation for long-term loading

Long-term monitoring was performed with displacement sensors and vibrating wire embedment strain gages. The displacement sensors were wireless devices that have two components; the displacement sensing element and the wireless transmitter (Figure 5). The displacement sensing element had a mechanical stroke length of 3 in [76.2 mm]. It is connected (i.e., wired) to the wireless transmitter that sends displacement readings, at pre-set time intervals, to a solar powered data collector that allows storage of and remote access to the information. Displacement sensors also measure temperature.

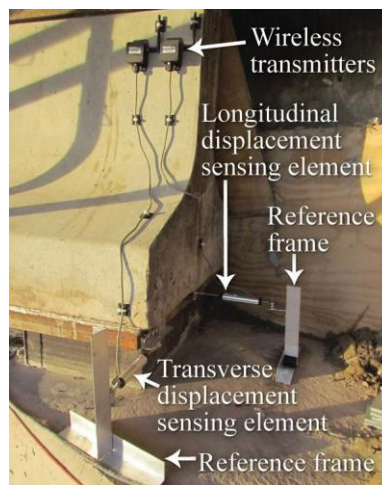


Figure 5—Displacement sensors as installed on the girder bearing at the south-east corner over the abutment

Five displacement sensors were installed on the bridge to understand the global displacement behavior of the bridge in the horizontal space. Girder bearings near the north-west and south-east corners over the abutments had two displacement sensors each (Figure 5). One sensor was placed on the girder near the south-west corner over the abutment. Having two sensors at a particular bearing, one in the longitudinal and the other in the transverse bridge

directions, and under the assumption of small rotations on the horizontal plane enabled the computation of the relative change of bearing position with time. This was not possible with the single sensor near the south-west abutment corner, however, this sensor still provided important bearing transverse displacement measurements. The acute corners of the bridge were chosen to be instrumented for displacements because they may experience significant movements¹. Figure 6 presents the layout of displacement sensors.

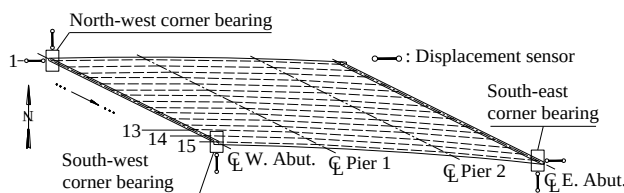


Figure 6—Displacement sensor layout shown on plan view

A total of twelve vibrating wire strain gages were installed in the concrete deck to understand its behavior due to long-term effects. Eleven were embedded in the deck, near the south-east corner of the bridge. The twelfth vibrating gage was used to measure shrinkage and temperature effects. This control gage was installed inside an unloaded, “stress free” concrete cylinder cast with the concrete used in the deck. The cylinder with the gage inside was placed near the bridge so that it is exposed to similar environmental conditions as the deck. Each of the vibrating wire gages was connected to a solar powered data logger that allowed remote access to the information. All vibrating wire gages also measure temperature.

Vibrating wire strain gages in the concrete deck were arranged in two lines: at deck mid-span between girders 14 and 15 (oriented in longitudinal and transverse directions), and along the edge of the top flange of girder 14 (oriented in transverse direction). All gages were attached to the top deck reinforcement, except for two along the latter line which were attached to the bottom reinforcement. Gages on the top layer and bottom layer deck reinforcement were at an average depth of 4 in [10.2 cm] and 6 in [15.2 cm] from the deck top surface, respectively. The layout of vibrating wire strain gages is shown on Figure 7. The acute corner region of the bridge was selected for instrumentation in order to better understand the development and progress of the typical diagonal concrete deck cracks that are regularly reported for skewed bridges.

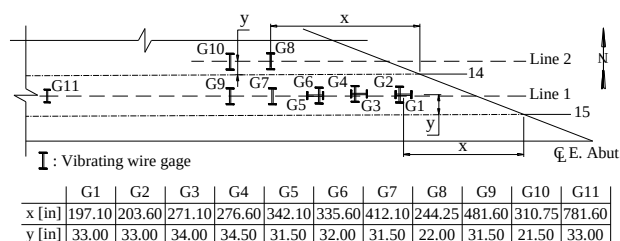


Figure 7—Deck vibrating wire strain gages layout shown on plan view [1 in = 0.0254 m]

LOAD TEST RESULTS

Results for short-term loading and Finite Element Modeling

Since data on the bridge under live load was only collected on three girders out of fifteen, the overall bridge load distribution was evaluated using a finite element model that was created and validated using test data. Finite element models used four-node shell elements for the deck and full depth diaphragms, and used frame elements for girders, parapets, partial depth diaphragms and steel in-span diaphragms. Full composite action was assumed between girders and deck by forcing deck and girder elements to connect at offset coincident nodes. All members were linear elastic.

Validation was achieved by gradually refining the finite element models. Table 3 summarizes changes made to the initial model (model v1). The final model (model v7) includes parapets, end, pier and intermediate diaphragms, stiffnesses of expansion elastomeric bearings, and material properties from cylinder compression tests where they were available. Translational and rotational stiffness of bearings were calculated using expressions on section 14.6.3 of AASHTO LRFD Bridge Design Specifications (BDS)⁹ and the upper-bound elastomer shear modulus from section

27.2.1 of the WisDOT BM⁸. The concrete modulus of elasticity for the deck, parapets, pier and end diaphragms was estimated using their 28-day compressive strengths. On other hand, the modulus of elasticity of each prestressed girder was evaluated recognizing the concrete gain of strength with age. In fact, there was an average difference of 203 days between the load test and 28-day cylinder break dates. Projected compressive strengths were calculated using expressions proposed by the ACI Committee 209¹⁰.

Table 3—Refinement steps of the finite element models

Improvements	Models for validation						
	v1	v2	v3	v4	v5	v6	v7 (final)
Parapets	x	✓	✓	✓	✓	✓	✓
End diaphragms	x	x	✓	✓	✓	✓	✓
Pier diaphragms	x	x	x	✓	✓	✓	✓
Intermediate diaphragms	x	x	x	x	✓	✓	✓
Bearing stiffnesses	x	x	x	x	x	✓	✓
Refined material properties	x	x	x	x	x	x	✓

The impact of model refinements on results is presented in Figure 8 as error normalized with respect to the error of the initial model (model v1). Error is assumed as the sum of strain deviations (in absolute value) from load test results at all sensor locations and for all load cases. Figure 8a shows that better estimates of material properties, and inclusion of secondary members such as parapets and end diaphragms play a significant role in bending behavior, and that including estimated bearing stiffnesses had a small contribution to the accuracy of results. In relation to shear behavior, Figure 8b shows a major impact of end diaphragms, followed in importance by bearing stiffnesses. Additional details of the finite element models will be published elsewhere.

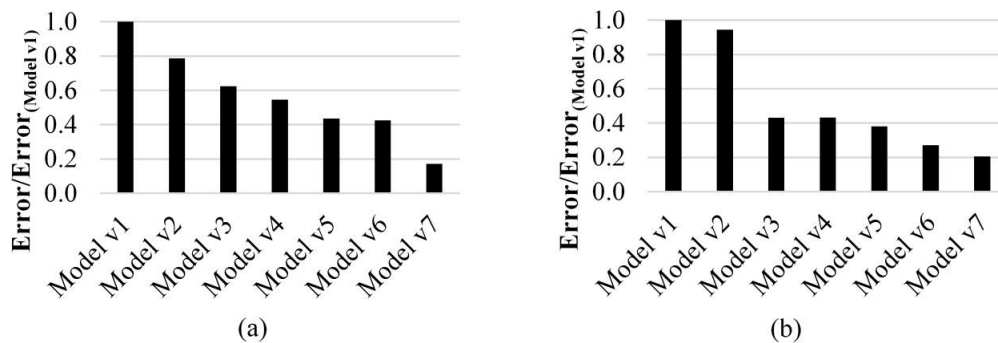


Figure 8—Error from refined models normalized with respect to the initial model: (a) bending; (b) shear

In general, results from truck loading and bridge finite element model (model v7) were in good agreement. Figure 9 compares strains obtained from testing (on the left) to ones obtained from finite element model results (on the right) as a function of approximate position of the front axle of the leading truck. Due to space constraints, in this paper, only results for the second run of the first load case (Figure 3 and Table 2), along the shear and bending strain gage lines at nominal distances of 5 ft [1.5 m], 15 ft [4.6 m], 40 ft [12.2 m] and 80 ft [24.4 m] from the west girder end (Figure 4), are presented as representative examples.

The validated model (model v7) was used to obtain live load distribution and to explore the effect of bridge details on load distribution. The models shown in Table 4 were built. Model 0 is the original bridge with as-built details.

Girder distribution factors for shear and bending moment were computed for all models and compared. These results were also compared with distribution factors obtained with the distribution factor method of AASHTO LRFD BDS⁹ with WisDOT BM⁸ exceptions, even though these factors do not apply to this bridge as bridge geometry is not regular (e.g., skew beyond 60°). Figure 10 presents the ratio of the AASHTO live load distribution factors to the corresponding factors obtained from the finite element models for shear and moment, on interior and exterior girders. Values of the ratio above 1.0 indicate AASHTO distribution factors are conservative.

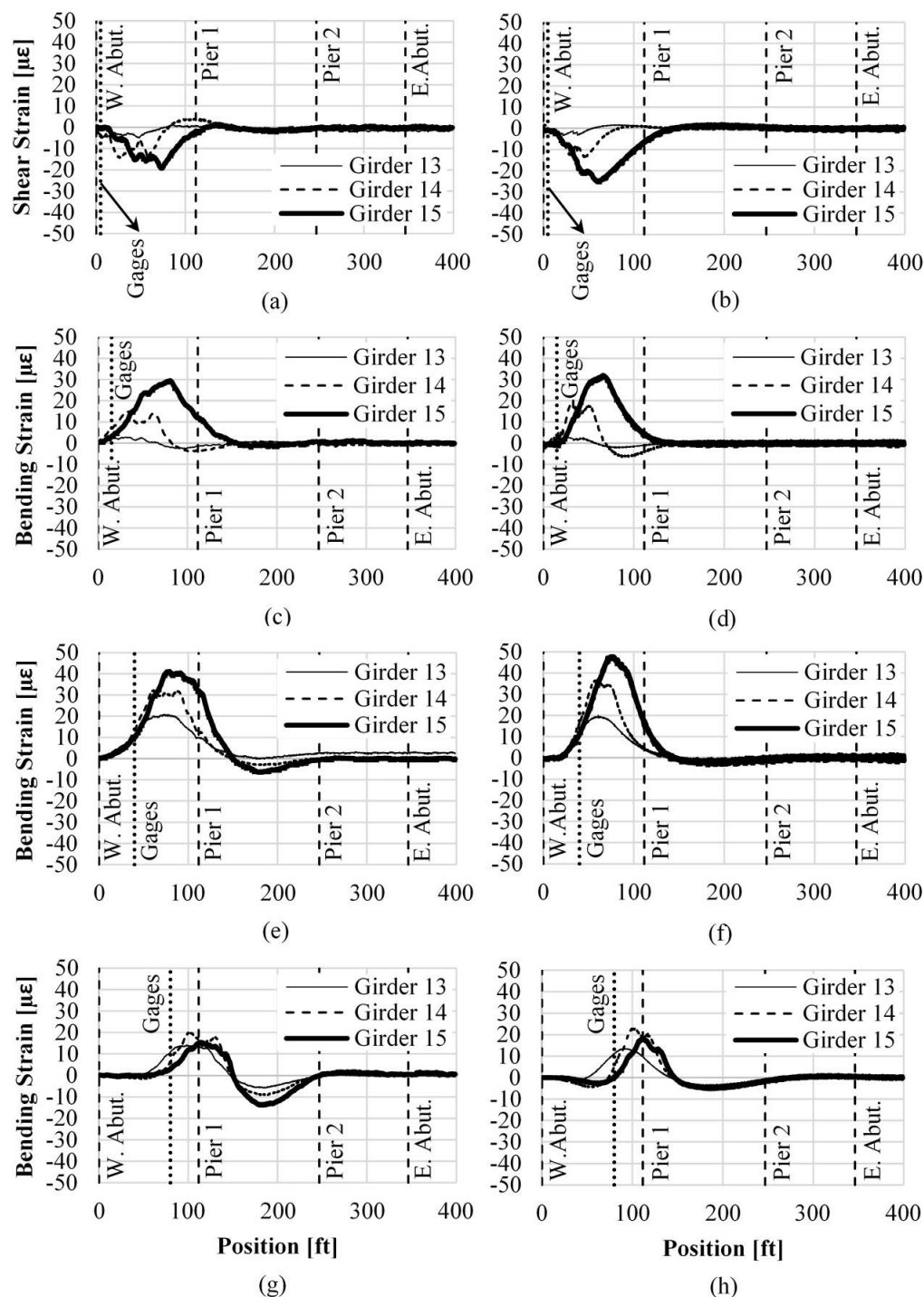


Figure 9—Shear and bending strains under short-term loading as a function of front axle position: from (a) test at 5 ft; (b) model at 5 ft; (c) test at 15 ft; (d) model at 15 ft; (e) test at 40 ft; (f) model at 40 ft; (g) test at 80 ft; (h) model at 80 ft [1 ft = 0.3048 m]

Results on Figure 10 reveal that the bridge details considered in Table 4 have a small influence on load distribution factors for both moment and shear. Changes in abutment diaphragms (Model 1 or 2) have a larger impact on live load distribution than others. For instance, for shear on interior girders of the exterior spans (Figure 10c), and especially on the third span, AASHTO load distribution factor was less conservative, which indicates a reduced ability to

distribute shear forces without an abutment diaphragm. Such results are not evident for shear on exterior girders or for moment. On the other hand, on the exterior spans for positive moment on the interior (Figure 10a) and exterior (Figure 10b) girders, full depth abutment diaphragms (Model 2) caused a more conservative AASHTO distribution factor, which indicates a better distribution of girder bending moments. In the case of shear force distribution, changes caused by full depth abutment diaphragms are negligible.

Table 4—Finite element model parameters

Model	Abutment Diaphragm	Diaphragm along the span	Pier Diaphragm
0	Partial depth concrete	Steel at third points of span	Full depth concrete
1	None	Steel at third points of span	Full depth concrete
2	<i>Full depth without encasing girder bearings</i>	Steel at third points of span	Full depth concrete
3	Partial depth concrete	None	Full depth concrete
4	Partial depth concrete	Steel at third points of span	None

Figure 10 also presents how AASHTO distribution factors compare to the ones obtained from finite element models. AASHTO distribution factors for moment are quite conservative for positive moment, and stand close to or at the verge of being un-conservative for negative moment on the interior and exterior girders, respectively. It should be noted that the distribution factors presented do not include moment reduction for skewed bridges following the WisDOT exception to AASHTO LRFD BDS⁹. If reduction factors were included, AASHTO factors will not be able to predict load distribution conservatively. In the case of distribution factors for shear, AASHTO stays moderately conservative for shear on interior girders, and it displays a similar trend for shear on exterior girders, with the exception of the interior span, where it lies on the limit.

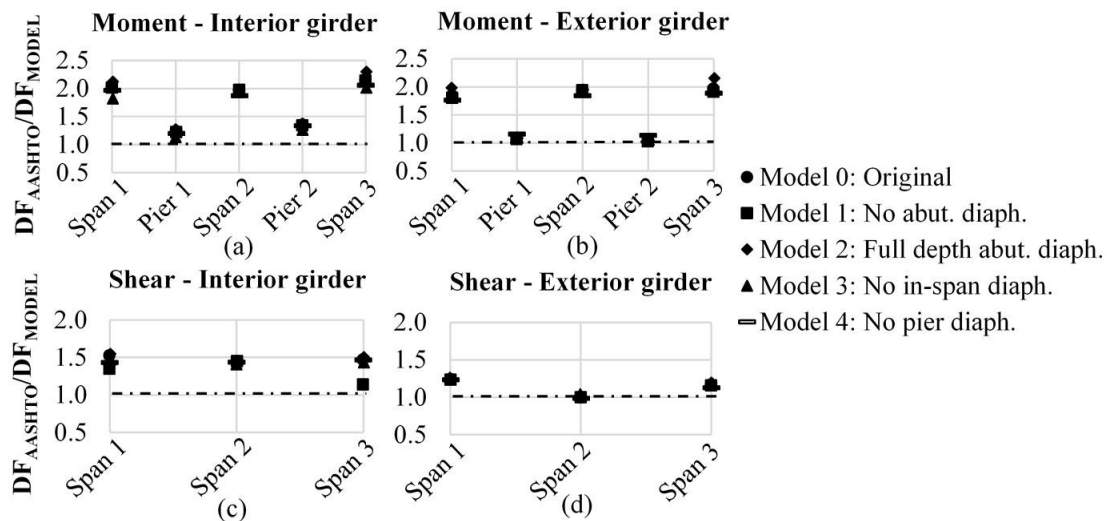


Figure 10—Comparisons of live load distribution factors from AASHTO LRFD BDS and from finite element models for: (a) moment-interior girder; (b) moment-exterior girder; (c) shear-interior girder; (d) shear-exterior girder

Results for long-term loading

Displacement sensors—These sensors measure displacements and temperature from the three corners of the bridge to understand the global displacement behavior of the bridge in the horizontal space under long-term loading. Sensors were attached to the top of the bearings of exterior girders. Data collection from displacement sensors started on November 13, 2016, shortly after construction, and will continue for a year. In this paper, results up to May 16, 2017 are presented on Figures 11-13. Reported displacement values are relative to bearing positions at the beginning of data recording. Since data recording started shortly after construction, displacements at the beginning of recording are expected to be virtually zero. Disturbances to sensors due to continuing construction resulted in loss of data for one month shown as gaps on the figures. The results after the data gap assume that no changes in displacement occurred during the period when no data were recorded. The displacement results are presented in two directions: in transverse (i.e., perpendicular to bridge axis) and longitudinal (i.e., along bridge axis) directions, and in parallel-to-expansion joint (racking) and normal-to-expansion joint directions.

Transverse and longitudinal displacements—Figures 11a-11b display the bearing displacements and the average internal temperature of the corresponding pair of sensors at the bridge acute corners. Figure 11c shows the transverse displacements in the south-west corner bearing and temperature, measured by a single sensor. In Figure 11c, potential impact of longitudinal displacements on transverse displacement measurements, which cannot be captured due to the lack of a second sensor, was neglected.

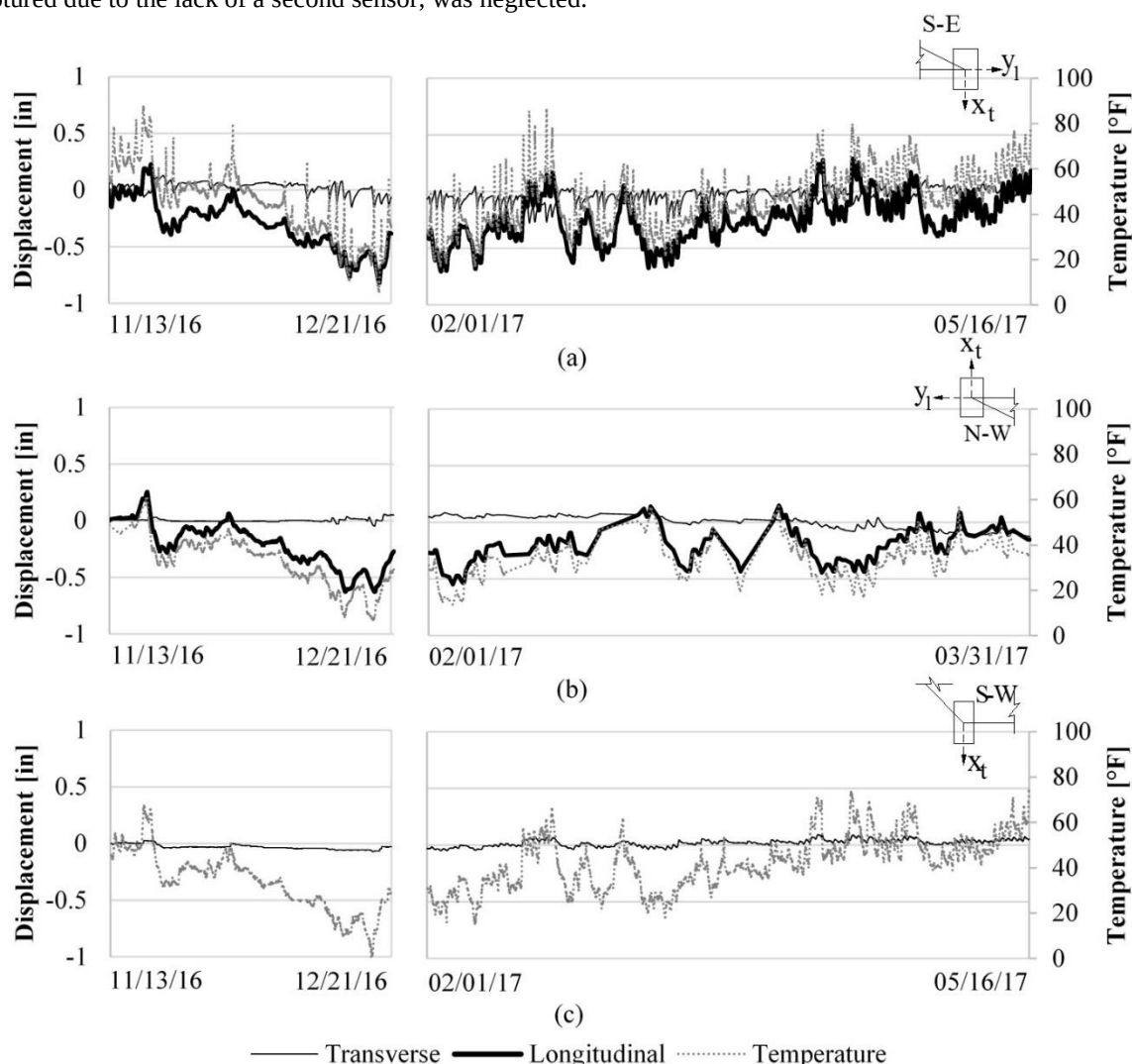


Figure 11—Bearing transverse and longitudinal displacements, and temperature on the: (a) south-east corner; (b) north-west corner; (c) south-west corner [1 in = 25.40 mm]

Results on Figures 11a and 11b show that longitudinal displacements were much larger than transverse displacements (up to 5 times) at both acute corner bearings. In all locations, transverse displacements were smaller than 0.28 in. It is also interesting to note that longitudinal displacements on the south-east bearing are larger than the ones observed on the north-west bearing, even though the expansion length for the latter is longer, as shown on Figure 1a. This can be explained by the horizontal curve of the bridge geometry, which gives rise to an effective expansion length for the south-east bearing more than 10 ft [3.0 m] longer.

Table 5 summarizes maximum values of transverse and longitudinal displacements ($\delta_{\max(+)}$ and $\delta_{\max(-)}$) since the beginning of data recording (construction), in both positive and negative directions according to the sign conventions shown in Figure 11. Table 5 also presents maximum changes in temperature ($\Delta T_{\max}$) between any two dates during data recording, and the corresponding change in transverse and longitudinal displacements ($\Delta\delta$) between those dates.

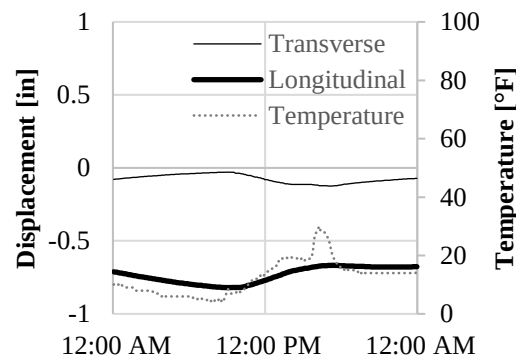
Table 5—Bearing displacements and temperature in transverse and longitudinal directions

Location	Period	Transverse direction				Longitudinal direction			
		$\delta_{\max(+)}$, in [mm]	$\delta_{\max(-)}$, in [mm]	$\Delta T_{\max}$, °F [°C]	$\Delta\delta$, in [mm]	$\delta_{\max(+)}$, in [mm]	$\delta_{\max(-)}$, in [mm]	$\Delta T_{\max}$, °F [°C]	$\Delta\delta$, in [mm]
S-E corner	1	0.13 [3.28]	-0.15 [-3.87]	-66 [-36]	0.01 [0.35]	0.23 [5.73]	-0.82 [-20.82]	-66 [-36]	-0.98 [-24.80]
	2	0.10 [2.53]	-0.28 [-7.06]	70 [39]	-0.27 [-6.80]	0.29 [7.39]	-0.70 [-17.90]	70 [39]	0.83 [21.20]
N-W corner	1	0.06 [1.47]	-0.05 [-1.24]	-54 [-30]	-0.02 [-0.58]	0.25 [6.46]	-0.63 [-15.93]	-54 [-30]	-0.81 [-20.53]
	2	0.09 [2.25]	-0.13 [-3.19]	44 [24]	0.02 [0.48]	0.15 [3.76]	-0.55 [-13.97]	44 [24]	0.69 [17.55]
S-W corner	1	0.07 [1.83]	-0.02 [-0.61]	-67 [-37]	0.08 [2.09]	N/A	N/A	N/A	N/A
	2	0.05 [1.24]	-0.09 [-2.33]	60 [33]	-0.09 [-2.35]	N/A	N/A	N/A	N/A

Changes in bearing displacements, $\Delta\delta$, caused by maximum temperature changes, $\Delta T_{\max}$, as displayed in Table 5 do not correspond to the difference between measured maximum displacements, $\delta_{\max(+)}$ and $\delta_{\max(-)}$. In other words, maximum displacements do not take place when temperature reaches a maximum or a minimum. The discrepancy is significant for transverse displacements on acute corner bearings, less significant on the obtuse corner bearing, and negligible for displacements in the longitudinal direction. These discrepancies show that transverse displacements are impacted by factors other than temperature.

Figures 11a and 11b show that the longitudinal displacements correlated well with temperature, indicating that temperature change is the main reason for longitudinal bridge displacements. In fact, the Pearson's correlation coefficient, which indicates for a pair of variables how strong their linear association is (i.e., ± 1 : perfect linear relationship, 0: no linear correlation)¹¹, between longitudinal displacements and temperature for both bearings and periods had a strong positive correlation with values above 0.90. On the other hand, correlation coefficients for temperature and transverse displacements on acute corner bearings were below 0.25. Such low coefficients show that transverse displacements and temperature on acute corners did not correlate linearly and that factors other than temperature may have a more important role on transverse displacements. In contrast to this finding, transverse displacements and temperature on the obtuse corner had a strong positive linear association with correlation coefficients of 0.90 for both periods. This may imply that transverse bearing movements also depend on location.

To inspect temperature effects on movement more closely, transverse and longitudinal bearing movements on a single day were investigated and are presented in Figure 12. During a single day, both longitudinal and transverse displacements had a significant linear correlation with temperatures. Pearson coefficient for transverse displacement was -0.90. Figure 12 suggests that temperature could influence transverse bearing movements, but such influence is not obvious when long term displacements are considered (Figure 11).

**Figure 12**—South-east bearing displacements and temperature on December 19, 2016 [1 in = 25.40 mm]

Parallel-to-expansion joint (racking) and normal-to-expansion joint bearing displacements—Results presented in the previous section seem to show that the unique support fixity arrangement (Figure 1d) led to small transverse displacements as intended by design. Figure 13 shows parallel-to-expansion joint and normal-to-expansion joint displacements, calculated from transverse and longitudinal displacements based on bridge geometry. Parallel-to- and normal-to- joint displacements at the south-west corner cannot be calculated since only a single sensor was used at this location.

When Figures 11a and 11b are compared with Figure 13, it can be seen that bearing displacements in parallel-to-joint and normal-to-joint directions are mainly caused by longitudinal movements of the bridge. In fact, given the high skew of the bridge (i.e. 64°), approximately 90% and 40% of the bearing longitudinal displacement translates into the parallel-to-joint displacement, and normal-to-joint displacements, respectively. In general, parallel-to-joint displacements are larger than normal-to-joint displacements.

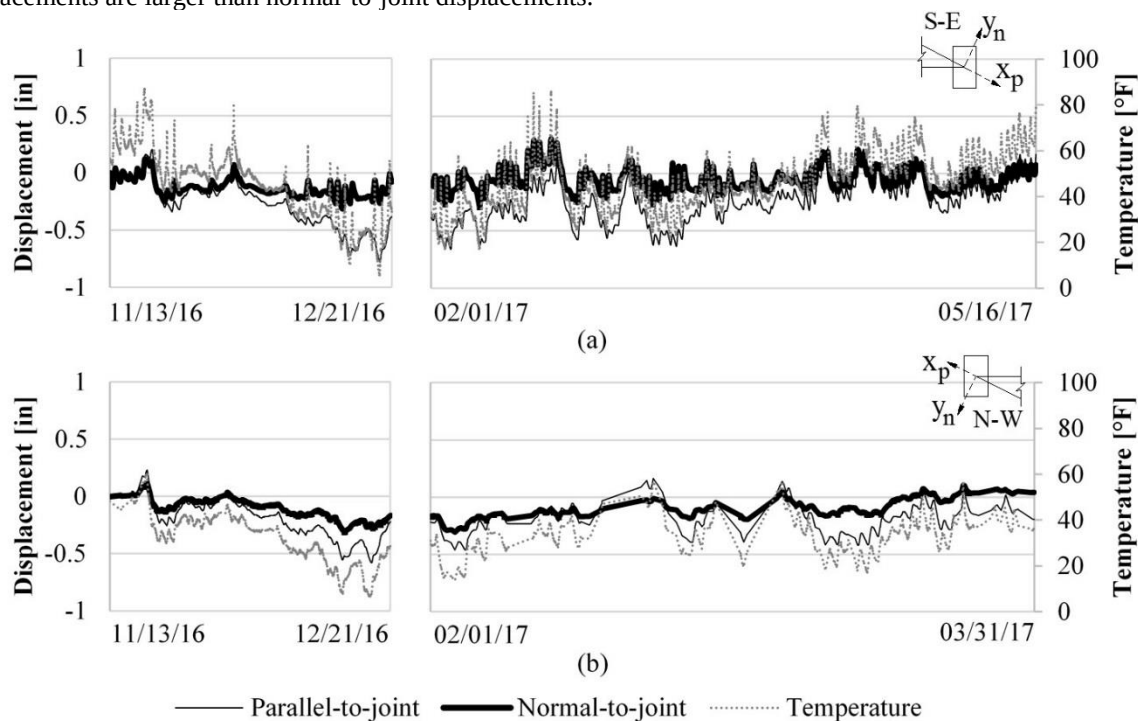


Figure 13—Bearing parallel-to-joint and normal-to-joint displacements, and temperature on the: (a) south-east corner; (b) north-west corner [1 in = 25.40 mm]

Parallel-to-joint displacements at both corners presented a strong positive linear relationship with temperature, with correlation coefficients beyond 0.90. Furthermore, normal-to-joint displacements at both corners exhibited a less strong, still positive, linear correlation, with coefficients between 0.76 and 0.96. As expected, these results confirm the major influence of longitudinal bearing movements on these displacement components. It was also noted that the correlation strength changes from warming to cooling periods. During the seasonal warming period, the linear correlation between displacements (i.e., for all components) and temperature is always less strong, indicating that temperature is more predominant during the cooling period.

Table 6 presents maximum values of parallel-to- and normal-to- expansion joint displacements ($\delta_{\max(+)}$ and $\delta_{\max(-)}$) recorded since the beginning of data acquisition, in both positive and negative directions according to the sign conventions shown in Figure 13. Table 6 also presents maximum changes in temperature ($\Delta T_{\max}$) between any two dates, and the corresponding change in parallel-to- and normal-to- expansion joint displacements ($\Delta\delta$) between those dates. Changes in parallel-to- and normal-to- expansion joint displacements caused by maximum temperature changes as shown in Table 6 are not equal to the difference between measured maximum positive and negative displacement values. For these displacement components, discrepancies are within the range of the ones found for longitudinal displacements, with smaller differences for the parallel-to-joint displacement direction.

Table 6—Bearing displacements and temperature in parallel-to- and normal-to- joint directions

Location	Period	Parallel-to-joint direction				Normal-to-joint direction			
		$\delta_{\max}$, in [mm]	$\delta_{\min}$, in [mm]	$\Delta T_{\max}$, °F [°C]	$\Delta\delta$, in [mm]	$\delta_{\max}$, in [mm]	$\delta_{\min}$, in [mm]	$\Delta T_{\max}$, °F [°C]	$\Delta\delta$, in [mm]
S-E corner	1	0.20 [5.17]	-0.78 [-19.77]	-66 [-36]	-0.91 [-23.03]	0.12 [3.16]	-0.31 [-7.97]	-66 [-36]	-0.36 [-9.22]
	2	0.23 [5.89]	-0.66 [-16.80]	70 [39]	0.68 [17.36]	0.30 [7.73]	-0.29 [-7.30]	70 [39]	0.55 [13.94]
N-W corner	1	0.23 [5.82]	-0.58 [-14.75]	-54 [-30]	-0.73 [-18.56]	0.11 [2.81]	-0.31 [-7.79]	-54 [-30]	-0.35 [-8.80]
	2	0.16 [4.14]	-0.46 [-11.72]	44 [24]	0.62 [15.85]	0.10 [2.66]	-0.31 [-7.79]	44 [24]	0.30 [7.54]

Temperature and transverse displacements seem to have a relationship that is disrupted by other factors, which resulted in a weak correlation between the two measurements. Inclusion of additional variables into analyses (e.g. shrinkage), additional sensors and instrumentation time could help understand global bridge movements. Nevertheless, it is clear from the small lateral bearing displacements that any lateral translation or rotation on the bridge are very small, and that the thermal movements of the bridge are predominantly due to longitudinal expansion or contraction.

Vibrating wire strain gages—Recording of deck strains began on October 18, 2016 (i.e., 85 hours after deck pour), and will be in progress for one year. Figures 14 and 15 present strain and temperature data up to May 13, 2017. Figures 14a, 14c, and 14d display total deck strains along lines 1 and 2. Figure 14b shows transverse deck strains along line 1 after the control gage strains were subtracted (i.e., gage 12 strains). Figure 15 shows temperature on the control gage, and the average of temperature from all deck sensors. Temperatures on deck sensors were similar.

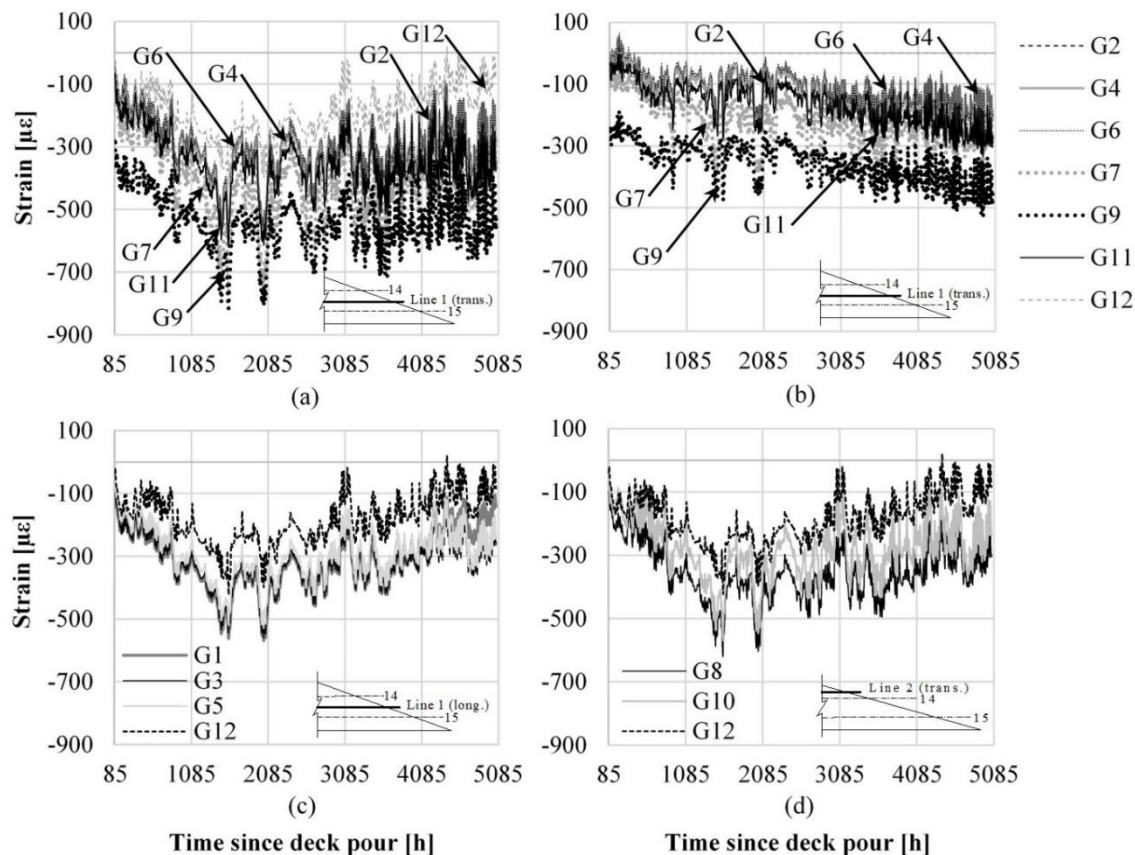


Figure 14—Deck strains: (a) total transverse strains on line 1; (b) transverse strains on line 1 minus strains from gage 12; (c) total longitudinal strains on line 1; (d) total transverse strains on line 2. Tension: (+); compression: (-)

Figure 15 shows that the average lowest temperature registered was of -4°F (-20°C) on December 19, 2016, and corresponded to a temperature drop of -77°F (-43°C) from the beginning of data collection. During this cooling period, the control gage (G12) exhibited a gradual increase in compressive strains ($-369\text{ }\mu\text{strain}$), typical of negative temperature changes and shrinkage in concrete (Figures 14a, 14c, and 14d). The drop in temperature for the control gage was -61°F (-34°C). On the other hand, it can be seen on Figure 15 that the average highest temperature was of 81°F (27°C) on April 15, 2017, and was reached after a change in temperature of 85°F (47°C) from the date of the lowest temperature. During the warming period, the control gage presented a gradual increase in tensile strains ($361\text{ }\mu\text{strain}$), which is expected for positive temperature changes (Figures 14a, 14c, and 14d). The increase in temperature for the control gage was 71°F (39°C).

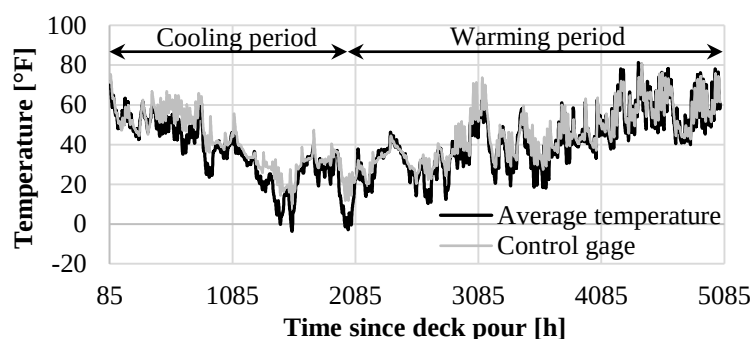


Figure 15—Average temperature of all gages in the deck and temperature in the control gage

Gages on lines 1 and 2 showed a similar behavior when compared to the control gage (Figures 14a, 14c, and 14d). During the cooling period, strains in the instrumented region of the deck dropped significantly. This tendency was more pronounced on transversely oriented gages (Figures 14a and 14d), which exhibited average maximum changes in strain of $-556\text{ }\mu\text{strain}$ and $-505\text{ }\mu\text{strain}$ for gages along lines 1 and 2, respectively. Longitudinal gages on line 1 displayed an average maximum change in strain of $-441\text{ }\mu\text{strain}$.

During the warming period, strains showed a constant increase in tension. Positive changes in strain (tension) were significant; however, not large enough to take the instrumented region into tension. As in the cooling period, transverse gages registered greater average maximum changes in strain, which were of $482\text{ }\mu\text{strain}$ and $465\text{ }\mu\text{strain}$ for gages on lines 1 and 2, respectively. The average maximum change in strain for the longitudinal gages on line 1 was of $411\text{ }\mu\text{strain}$.

When strain measurements corresponding to the same temperatures measured at the beginning of the cooling period and at the end of the warming period were compared, the difference was a significant compressive residual strain. Transverse gages along lines 1 and 2 presented average compressive residual strains of $183\text{ }\mu\text{strain}$ and $154\text{ }\mu\text{strain}$, respectively. Similarly, longitudinal gages measured an average compressive residual strain of $109\text{ }\mu\text{strain}$. Interestingly, the longitudinal gage 1 on line 1 had almost no residual strain, similar to the control gage, which might be an indicator for tensile strains offsetting the compression strains in this localized region.

Figure 14b shows interesting results for the transversely oriented gages along line 1. It indicates that after the removal of strains captured by the control gage, which are interpreted to be temperature and shrinkage strains, this region of the deck gradually and almost linearly accumulated compressive strains. In effect, almost all gages exhibited the same behavior. Results from other gages are not presented due to space limitations. Longitudinal gage 1 on line 1 was the exception and displayed a rapid increase in tensile strains approximately two months before the end of the warming period. Note that this gage is closer to the end of the bridge, and may be crossed by transverse or diagonal paths of high tensile strains that lead to typical deck acute corner cracking.

The results do not fully explain the accumulation of compressive strains over time. The limited number of monitoring points on the deck made it difficult to visualize global deck deformations. In addition, boundary conditions, exposed surface area of concrete, traffic loading and temperature gradient through the depth of the deck¹² are different for the control gage and gages in the deck. The deck also has mild reinforcement. These may lead to differences in shrinkage

and temperature strains. Therefore, corrections for shrinkage and temperature effects using readings of the control gage should be used with caution.

Figure 16 supplements Figure 14, and presents the total change in strain during the data recording period as a function of gage position along lines 1 and 2. It shows in general that changes in transverse strains along line 1 are significant and beyond $-150 \mu\text{strain}$, with a slight tendency to increase towards the pier. A similar trend may exist towards the abutment after gage 4, as suggested by the readings on gage 2. Similar to line 1, changes in transverse strains along line 2 are important. However, with only two gages along this line, it is difficult to distinguish a clear trend. It is only evident that the gage closer to the abutment experienced larger compressive strains. On the other hand, changes in longitudinal strains on line 1 were insignificant at the location of the gage closer to the abutment (gage 1). For locations beyond this gage, changes in longitudinal strains were comparable to changes in strain in the transverse direction, and showed no visible tendency to increase or decrease towards the pier.

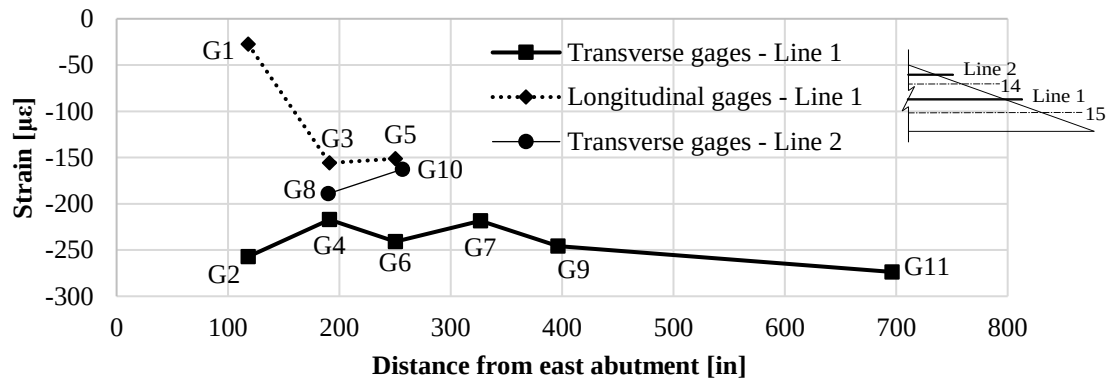


Figure 16—Total change in strain along gage lines 1 and 2 during data recording period.
Tension: (+); compression: (-)

CONCLUSIONS

Based on the results presented on this paper, the following conclusions were made:

- Refinements to the bridge finite element model during the validation process revealed that an acceptable representation of the behavior of concrete bridges may require more than the inclusion of only principal structural components. In general, refinement requirements depend on the level of accuracy that is deemed acceptable. It was found that parapets, end and intermediate diaphragms, and enhanced estimates of material properties (modulus of elasticity) are important for predicting girder bending behavior. Girder end shear response is highly influenced by end diaphragms, and to some extent by bearing stiffnesses.
- The validated finite element model showed that the bridge details such as end and intermediate diaphragms have a small influence on girder shear and bending distribution. However, the use of full-depth abutment diaphragms improves girder bending distribution on exterior spans, and that the absence of abutment diaphragms degrades shear distribution on interior girders of exterior spans.
- Evaluations of AASHTO distribution factors for this bridge showed that they are overly conservative for positive moments, even though the factors are not applicable to the bridge due to its high skew. The predictions of negative moments over piers calculated using AASHTO distribution factors essentially match the ones obtained from the finite element model. These distribution factors would be un-conservative if skew correction (reduction) factors for moment were used. Distribution factors for shear are conservative for interior girders, and on exterior girders of exterior spans. Distribution factors for exterior girders of the central span compared well with finite element model estimates.
- Transverse bearing (and therefore girder and superstructure) displacements were small and were below 0.28 in [7.1 mm] during the observation period. Conversely, longitudinal displacements were significant and, on the acute corners, could reach maximum values up to 5 times larger than maximum transverse displacements.

- Even though transverse bearing displacements were minimized, presumably due to the unique support fixity arrangement employed in design, the maximum parallel-to-joint (racking) bearing displacements were only slightly smaller than maximum longitudinal bearing displacements. Similarly, the maximum normal-to-joint bearing displacements were larger than maximum transverse bearing displacements. Parallel-to-joint displacements were consistently larger than normal-to-joint displacements. The high skew of the bridge and the small magnitude of transverse displacements explain such results.
- Longitudinal bearing displacements have a strong positive linear correlation with seasonal and daily changes in temperature. This indicates that longitudinal movements are mainly caused by temperature. On the other hand, transverse bearing displacements do not correlate linearly to seasonal changes in temperature, but they have a linear association with temperature on certain days. This shows that transverse displacements are affected by temperature, but other factors are generally more influential.
- Both parallel-to- and normal-to-expansion joint bearing displacements have strong linear correlations with temperature, with the former having a stronger correlation. This confirms the significant influence of longitudinal movements on these alternative displacement directions.
- Based on the small magnitude of transverse displacements and the evident relationship between significant longitudinal bearing displacements and temperature, movements of the bridge are likely primarily caused by longitudinal thermal contraction and expansion. Any tendency of the bridge to rotate or laterally translate were small.
- Vibrating wire strain gages installed on an acute corner of the deck showed that deck behavior is complex. It seems to be significantly affected by the interaction of the deck with other structural components and by loads other than uniform changes in temperature. This is observed when measurements of strain from the control gage in a stress free concrete cylinder were compared to the ones of gages in the deck. Main observations on the deck behavior are as follows:

During the cooling period, the concrete deck accumulates significant compressive strains, which is expected for the superposition of shrinkage and negative temperature changes. Negative changes in strain seemed more pronounced in the deck transverse direction.

During the warming period, significant positive (tensile) changes in strain can develop. Larger changes in strain may be found in the deck transverse direction. In general, changes in strain caused by increasing temperatures were not large enough to create tension in the instrumented region. In fact, compressive strains beyond 150 μ strain could be observed at the end of the observation period at almost all instrumented points.

The deck behavior can be different in locations close to the abutment and closest to the acute corner of the bridge than other locations near the acute corner. This was suggested by the longitudinal gage 1 on line 1 (closer to the abutment) that presented significantly smaller compressive strains at the end of the warming period. This gage also had a gradual increase in tensile strains when the control gage strains were subtracted.

ACKNOWLEDGEMENTS

This research project was funded through WisDOT and the USDOT. Neither WisDOT nor the USDOT assume any liability for the contents or the use thereof nor does this presentation reflect official views, policies, standard specifications or regulations of either department. The technical support during instrumentation of Michael G. Oliva, professor emeritus, the University of Wisconsin, Madison and staff of the Structural Engineering and Earthquake Simulation Laboratory of the Department of Civil, Structural and Environmental Engineering at the University at Buffalo is particularly acknowledged. Logistic support provided by field engineers, particularly Philip Ciha, Brian Koch of WisDOT; Matt Pokorny, Jason Zembroski of GCG; David Wagner of Collins Engineers, were instrumental in load testing. The opinions and conclusions presented in this paper are of the authors' and do not necessarily reflect the views of the parties acknowledged above.

REFERENCES

1. Diaz Arancibia, M.; Okumus, P.; and Oliva, G. M., “Review of Skew Effects on Prestressed Concrete Girder Bridges: Problems and Current Practices,” *Proceedings of the PCI Convention and National Bridge Conference*, Cleveland, OH, February 28-March 4, 2017, 50, p. 18.
2. Diaz Arancibia, M., and Okumus, P., “Causes of Excessive Detensioning Stresses in Northeast Extreme Tee (NEXT) Beams,” *PCI Journal*, V. 62, No. 3, 2017, pp. 31-45.
3. Hag-elsafi, O., and Kunin, J., “Load Testing for Bridge Rating: Dean’s Mill Road over Hannacrois Creek,” *Report No. 147*, Transportation Research and Development Bureau, New York State Department of Transportation, Albany, NY, 2006, p. 60.
4. French, E. W. C.; Dymond, Z. B.; and Shield, K. C., “Investigation of Shear Distribution Factors in Prestressed Concrete Girder Bridges,” *Report No. MN/RC 2016-32*, Research Services and Library, Minnesota Department of Transportation, Minneapolis, MN, 2016, p. 595.
5. Fu, G.; Feng, J.; Dimaria, J.; and Zhuang, Y., “Bridge Deck Corner Cracking on Skewed Structures,” *Report No. RC-1490*, Construction and Technology Division, Michigan Department of Transportation, Lansing, MI, 2007, p. 153.
6. Huang, H.; Shenton, W. H.; and Chajes, J. M., “Load Distribution for a Highly Skewed Bridge: Testing and Analysis,” *Journal of Bridge Engineering*, V. 9, No. 6, 2004, pp. 558-562.
7. Wisconsin Department of Transportation, “Maintenance & Inspection — Highway Structures Information System (HSI),” 2017, <http://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrcs/strct/hsi.aspx>.
8. Wisconsin Department of Transportation, “Bridge Manual,” 2017, Madison, WI.
9. AASHTO, “LRFD Bridge Design Specifications,” Customary U.S. Units, 7th Edition, American Association of State Highway and Transportation Officials, Washington, DC, 2014, p. 1716.
10. ACI Committee 209, “Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures (ACI 209R-92, Reapproved 1997),” *ACI Manual of Concrete Practice*, American Concrete Institute, Farmington Hills, MI, 1992, p. 47.
11. Walpole, E. R.; Myers, H. R.; Myers, L. S.; and Ye, K., *Probability and Statistics for Engineers and Scientists*, 9th Edition, Prentice Hall, Boston, MA, 2012, p. 812.
12. Phares, M. B.; Greimann, L.; and Liu, Z., “Evaluation of the Need for Longitudinal Median Joints in Bridge Decks on Dual Structures,” *Report No. TR-661*, Iowa Highway Research Board, Iowa Department of Transportation, Ames, IA, 2015, p. 97.

ACI student member **Mauricio Diaz Arancibia** is a graduate research assistant in the Department of Civil, Structural and Environmental Engineering at University at Buffalo, the State University of New York, Buffalo, NY.

ACI member **Pinar Okumus** is an Assistant Professor in the Department of Civil, Structural and Environmental Engineering at University at Buffalo, the State University of New York, Buffalo, NY. Her research interests include understanding the fundamental behaviors of reinforced concrete and prestressed concrete highway bridge members, inelastic analytical modeling and experimental verification of shear-dominated zones and extreme response, and accelerated bridge construction.

RATING OF CONCRETE ROAD BRIDGES WITH STATIC PROOF LOAD TESTS

Anna Halicka, Dick A. Hordijk, Eva O.L. Lantsoght

Synopsis: Nowadays, finite element analyses provide information about the performance of a structure, but they are more or less simplified. Therefore, load tests are the only way to find the “real” behavior of an existing bridge subjected to the rating process. In this paper, the *state-of-the-art* concerning load tests of concrete road bridges is presented, and the problems of the execution of such tests are specified. It is pointed out that only load tests accompanied with current finite element analyses may result in a proper assessment of the level of safety of the bridge. The authors’ procedure of complex assessment of such bridges combines *in-situ* examination of the structure, load testing, and finite element modeling.

The paper discusses the following topics:

- Aims and fundamentals of static diagnostic and proof load tests,
- The load application method according to different codes and specifications,
- The basis for proper assessment of the target load: reliability index, partial factors approach, global rating factor approach,
- Establishing the load allowable on the bridge, based on the applied proof load,
- The proposed procedure of assessment of existing concrete road bridges by load testing.

Keywords: codes of practice; field testing; finite element modeling; load testing; measurements; proof load testing; reinforced concrete bridges

Anna Halicka is an associate professor and head of Building Structure Unit in Faculty of Civil Engineering and Architecture at Lublin University of Technology, Lublin, Poland. Her research interests include assessment of existing concrete and masonry structures (especially durability and safety aspects), concrete composite structures (interface between two concrete parts and support zones), as well as reinforced and prestressed concrete tanks and silos).

Dick A. Hordijk is a full professor at Delft University of Technology, Delft, the Netherlands. He received his MS and PhD in civil engineering from Delft University of Technology, Delft, the Netherlands, in 1985 and 1990, respectively. His research interests include concrete fracture mechanics, assessment of existing structures, structural application of new concrete types, and forensic engineering.

ACI member **Eva O. L. Lantsoght** is a full professor at Universidad San Francisco de Quito, a structural engineer at Adstren and a researcher at Delft University of Technology. She is a member of ACI 445-0D Shear Databases and of ACI-ASCE 421, Design of Reinforced Concrete Slabs, and an associate member of ACI 342, Evaluation of Concrete Bridges and Bridge Elements, ACI 437, Strength Evaluation of Existing Concrete Structures, and ACI-ASCE 445, Shear and Torsion.

INTRODUCTION

Nowadays, software for structural analysis is widely available and commonly used. Finite element analyses provide insight in the structural performance of the modeled structure, but they are more or less simplified. Therefore, non-destructive load tests are the only way to find the “real” behavior of an existing bridge that needs to be rated. Unfortunately, the high cost of load tests may be the reason why bridge owners are reluctant to use such tests. Another drawback for using load tests is the risk of damage to the structure during the test.

The finite element analysis of a structure needs detailed input values and relationships to simulate the behavior of the structure from initial loading to its ultimate limit state. However, for design, some simplifications are usually introduced:

- Designers often use only two-dimensional schemas in analysis, whereas in reality, the bridge behavior is three-dimensional.
- A simplified static model (e.g. simply or continuously supported beams) is adopted and ideal supports conditions are assumed in the numerical model, which do not always correspond to the actual support stiffness.
- The actual bridge dimensions may differ from the designed ones.
- The location and weight of the non-structural elements (pavements, parapets, railings) are often not the same as in the design drawings, and these elements are sometimes neglected or simplified in the numerical model. However, their effect on the overall structural stiffness can be considerable (Barker, 2001).
- Most often, only a simple elastic-static analysis of the structure is performed using the modulus of elasticity and Poisson’s ratio of the designed concrete class. However, the actual concrete might be of a different class than the designed one. Additionally, undetected damage may be present. These differences between designed and actual class may be aggravated by the increase of the concrete strength with age on one hand and concrete deterioration, also connected with age, on the other hand.

These inaccuracies of numerical models result in the risk of an incorrect assessment of the bridge capacity. However, the main goal of design is to ensure that a structure fulfils the requirements of the code concerning the ultimate and serviceability limit states. It is not necessary for the designer to know the residual capacity of the structure at some future time. Anyway, for existing structures subjected to live loads larger than what they were designed for, or whose capacity is subject to discussion due to material degradation, the question about the residual capacity of the structure becomes relevant. Some of the above-mentioned inaccuracies are also characteristic for numerical models used for the assessment of bridges, although the incorrectness can be reduced: the dimensions may be measured, the weight and location of non-structural elements is known, and the concrete strength can be determined based on core samples.

The most important topic for the assessment of an existing bridge is the determination of the actual *load-deflection* relationship. Only load testing allows to verify and determine the generalized actual stiffness of the structure and the (linear) part of the *load-deflection* curve up to the proof load imposed onto the bridge. On that basis it is possible to predict the live load that may be permitted on the bridge. This allowable load usually causes effects in the structure that lie within the linear part of the structural behavior.

The correctness of analyses including load tests results in the safety level of the structure. The required safety level (expressed e.g. by the reliability index) should be determined in relation to bridge importance and

size. For example, a bridge in an important highway connection will require a higher safety level than a bridge in a rural road used by few vehicles. Additionally, the following questions should be answered:

- which level of accuracy of the analysis (especially numerical model) connected with proof loads should be obtained,
- which number of parameters must be measured in the field,
- which target proof load should be applied.

Information about load testing of concrete bridges can be found in the literature. The practical aspects were described (Cai and Shahawy, 2003; Cai et al., 2012; Enochsson et al., 2008; Gutermann et al., 2003; Lagoda and Lagoda, 2014; Lazinski and Salamak, 2010; Myers et al., 2012; Nowak and Saraf, 1996; Olaszek et al., 2012; Olaszek et al., 2014; Puurula et al., 2008; Ransom and Heywood, 1997; Taljsten et al., 2008; Yuefei et al., 2014), and the methods of bridge safety assessment were discussed (Cai and Shahawy, 2003; Casas and Gómez, 2013; Faber et al., 2000; Fu and Tang, 1995; Jeppsson, 2003; Moses et al., 1994; Nowak and Tharmabala, 1988; Stewart et al., 2001; Toprak et al., 2012; Yuefei et al., 2014). Some tests in the past have been continued up to damage of the bridge (Bagge et al., 2015; Haritos et al., 2000; Lantsoght et al., 2016).

In some countries there are codes and requirements dedicated to load tests of bridges: the Manual for Bridge Evaluation (MBE) (AASHTO, 2016), the DAfStb guideline from Germany (Deutscher Ausschuss für Stahlbeton, 2000), and the RIRB requirements from Poland (Research Institute of Roads and Bridges, 2008). In Switzerland codes SIA 505 269:2011 (SIA, 2011a) and SIA 505 269/2 (SIA, 2011b) are dedicated to all structures, not only to bridges.

In this paper, the problems of the execution of static proof load tests of concrete road bridges for bridge assessment and the determination of its load-carrying capacity are discussed on the basis of a *state-of-the-art* analysis and the authors' experience. A detailed assessment procedure for the *in-situ* examination of a structure and steps for load testing is proposed.

RESEARCH SIGNIFICANCE

The presented knowledge may be useful for bridge owners and for experts realizing the rating process. The paper provides guidance for the proof load tests and rating procedure that uses proof load tests. The guidelines for performing proof load tests and determining the target proof load are based both on European and American sources, so they are universal.

TYPES OF BRIDGE LOAD TESTS

The British National Steering Committee for the Load Testing of Bridges (The Institution of Civil Engineers - National Steering Committee for the Load Testing of Bridges, 1998) classifies bridge load tests as *supplementary load tests*, *proof loadings* and *proving load testing* (or *proof load tests* – term used in this paper, and conform the term used in the AASHTO MBE).

The aim of *supplementary load tests* is supplementing analytical methods and providing information of the actual bridge behavior under loading, including the load distribution through the particular structural members. Such load tests are usually complemented with non-destructive testing methods used for finding the concrete and reinforcement characteristics (Varela-Ortiz et al., 2013). The level of load should be sufficient to obtain measurable responses from the structure without causing any permanent structural damage. It is lower than the load applied on the structure under normal traffic. The obtained information is used to verify the initial numerical model. According to Casas et al. (Casas et al. 2009) the structural model is recognized as properly calibrated if the differences between measured and calculated deflections are not larger than 10% in prestressed concrete bridges, and 20% in concrete bridges. In the case of larger differences, the model should be modified.

Proof loadings are undertaken on newly constructed bridges before opening in order to validate the design model and assumptions. These tests are focused on confirming that the bridge satisfies the design requirements and that its capacity is not smaller than the designed one. The level of proof loadings is usually higher than used in supplementary tests and it is close to the serviceability limit state level. Such tests are demanded by codes in some countries e.g. Italy, Poland, Spain, Slovenia (Lagoda and Lagoda, 2014). In Poland, according to PN-S-10040/99 code (PN-S-10040/99, 1999) and RIRB Rules (Research Institute of Roads and Bridges, 2008), all new road bridges of a span length larger than 20 m (66 ft) must be load tested.

Proof load tests are usually undertaken for more realistic assessment of the behavior than that obtained from the theoretical analysis alone and for proving the safety of:

- bridges containing damaged or strengthened elements,
- bridges subjected to larger loads than assumed during design (due to increasing traffic and vehicle weight, or if a special transport is planned e.g. military),
- bridges with durability problems, for which it needs to be certified that the bridge can be used over the next years.

The load is increased in increments to some predetermined maximum (the so-called *target proof load*) or until the structure shows signs of deterioration or distress. The levels of target load are usually higher than used in

other testing and may reach over twice the nominal live load. On the basis of the target proof load, the magnitude of the load allowable on the bridge is deduced. The proof load test results may be also used for updating the numerical model and the verification of the ultimate limit state. The extrapolation to the ultimate limit state is adequate and safe only if the material parameters are determined and some pattern of bridge behavior at advanced levels of loading is known.

The second classification of bridge tests may be made according the manner of load application. The proof load may be:

- static: the load is applied at fixed points (the location and magnitude of the loads are fixed during one test scenario, but they may be changed during the next scenario),
- pseudo-static: trucks move at a low speed,
- dynamic: trucks move at full speed and vibrations are induced. Such proof loading allows to assess the dynamic characteristic of the bridge (frequencies) and as a result establish the critical (safe) velocities. The dynamic characteristics can then also be compared to those assumed during the design stage.

According to (Stewart et al., 2001), even the observation that the bridge has survived the current service load is, in itself, a proof load that provides information about the bridge's safety. Therefore service loads may be treated as a specific type of proof loads with an appropriate uncertainty.

FUNDAMENTALS FOR STATIC PROOF LOAD TESTS

A proof load test proves that the bridge is able to carry its full permanent load and some live load. The aim is to determine the magnitude of live load that can be carried by the bridge safely, or to demonstrate that the bridge fulfils the code requirements at a certain safety level. This target load, lower than the ultimate capacity of the bridge, can be used to evaluate if the bridge fulfils the required level of safety.

Before the proof load is applied, a detailed technical evaluation of the bridge's condition should be made. This inspection should be used for gathering the following information:

- the crack pattern and the size of the cracks,
- the location of weak concrete or losses of concrete,
- the concrete strength, based on core testing,
- carbonation depth, based on chemical tests of the concrete cover,
- presence of corrosion of reinforcement and the extent of section loss.

If rehabilitation measures (e.g. realkalisation, extraction of chloride ions, addition of reinforcement to complement corroded bars, and repair of the destroyed concrete cover) are needed, these should be performed.

The main topics of discussion with regard to static proof load tests are connected with the decisions concerning:

- the load application method and location,
- the target proof load magnitude, which should be based on the specified criteria ensuring no destruction of bridge during tests on the one hand and the demanded level of safety on the other hand,
- the specification of parameters to be measured and the measurement methods.

The load should be placed at a position so that the response in critical bridge members and critical zones in these members can be determined. For instance, to achieve the highest bending moment in simply supported structure, the load should be applied in the middle of the span and close to the support to achieve the highest shear force. It might be required to load the bridge at different load locations along the bridge width in order to show differences in the bridge behavior under load in the middle and edge lanes.

During proof load tests, no irreversible damage should be caused. Increasing the load above the limit of elastic performance is generally forbidden.

The behavior of the bridge is controlled by internal forces, which cannot be measured directly. Only their effects may be measured (e.g. deflection, strain of steel, or deformations and cracks in concrete). The limiting values of these effects, the so-called stop criteria, should be determined prior to the test. The bridge response should be continuously monitored and evaluated in real-time.

The target proof load value L_{targ} is the value of the load intended to be applied to the bridge during proof load tests in the assumed scenario and for the chosen location. It should be larger than the live load expected at future use in order to provide a margin of safety against some uncertainties such as the dynamic action of the load and unexpected overloads. This topic is discussed in a further section of this paper.

The deflection at the middle of the span is the value of primary interest. The resulting load-deflection relationship can be used for the calibration of the numerical model and to determine if the elastic limit was reached. The load-deflection relationship should be monitored continuously during the load test to ensure that the threshold of non-linear behavior is not exceeded. Digital levels, deflection inductive transducers, and surveyor's levelling instruments (optical or laser) may be used for the deflection measurement. The measurement of the support displacements should be carried out as well to find the net deflection of the superstructure (RIRB, 2008).