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Advanced Analysis and Testing Methods for Concrete Bridge Evaluation and Design

SP-342

Editors:
Benjamin Z. Dymond and Bruno Massicotte



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Advanced Analysis and Testing Methods for Concrete Bridge Evaluation and Design

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The cover photo depicting testing of a reinforced concrete bridge to failure was courtesy of co-editor Bruno Massicotte.

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PREFACE

Advanced Analysis and Testing Methods for Concrete Bridge Evaluation and Design

In recent years, both researchers and practicing engineers worldwide have been refining state-of-the-art and emerging technologies for the strength evaluation and design of concrete bridges using advanced computational analysis and load testing methods. Papers discussing the implementation of the following topics were considered for inclusion in this Special Publication: advanced nonlinear modeling and nonlinear finite element analysis (NLFEA), structural versus element rating, determination of structure specific reliability indices, load testing beyond the service level, load testing to failure, and use of continuous monitoring for detecting anomalies. To exchange international experiences among a global group of researchers, ACI Committees 342 and 343 organized two sessions entitled “Advanced Analysis and Testing Methods for Concrete Bridge Evaluation and Design” at the Spring 2019 ACI Convention in Québec City, Québec, Canada. This Special Publication contains the technical papers from experts who presented their work at these sessions. The first session was focused on field and laboratory testing and the second session was focused on analytical work and nonlinear finite element modeling. The technical papers in this Special Publication are organized in the order in which they were presented at the ACI Convention.

Overall, in this Special Publication, authors from different backgrounds and geographical locations share their experiences and perspectives on the strength evaluation and design of concrete bridges using advanced computational analysis and load testing methods. Contributions were made from different regions of the world, including Canada, Italy, and the United States, and the technical papers were authored by experts at universities, government agencies, and private companies. The technical papers considered both advanced computational analysis and load testing methods for the strength evaluation and design of concrete bridges.

The co-editors, Dr. Benjamin Dymond and Dr. Bruno Massicotte, are grateful for the contributions from the Special Publication authors and sincerely value the time and effort of the authors in preparing the papers in this volume. Furthermore, the Special Publication would not have been possible without the effort expended by the 24 experts who peer reviewed the papers in this volume.

Co-Editors

Benjamin Dymond and
Bruno Massicotte

TABLE OF CONTENTS

SP- 342-1:

Inelastic Shear Distribution in Prestressed Concrete Girder Bridges 1-19
Authors: Benjamin Z. Dymond, Catherine E. W. French, and Carol K. Shield

SP- 342-2:

Monitoring and Assessment of a Prestressed Concrete Segmental Box Girder Bridge 20-39
Authors: Marc Savard and Jean-François Laflamme

SP- 342-3:

Field Testing to Failure of a Skewed Solid Concrete Slab Bridge 40-59
Authors: Fabien Lagier, Bruno Massicotte, David Conciatori, and Jean-François Laflamme

SP- 342-4:

Instrumentation, Monitoring and Load Testing of the Champlain Bridge 60-79
Author: Dominic Lavigne

SP- 342-5:

Load Rating Reinforced Concrete Bridges without Plans: State-of-the-Practice..... 80-97
Authors: Rémy D. Lequesne and William N. Collins

SP- 342-6:

Estimation of Steel Rebar Strength in Existing Concrete Bridges 98-113
Authors: Alessandro P. Fantilli and Bernardino Chiaia

SP- 342-7:

Seismic Performance of Unreinforced Concrete Railroad Bridge Piers 114-128
Authors: Qiang Gui and Zhongguo John Ma

SP- 342-8:

Evaluation of the Orientation of Concrete Finishing Machines in Skewed Bridges..... 129-142
Authors: Faress Hraib, Li Hui, Brandon Gillis, Miguel Vicente, and Riyadh Hindi

SP- 342-9:

Non-Linear Evaluation of Strengthening Techniques for the Champlain Bridge 143-161
Authors: Denis Mitchell, Bruno Massicotte, William D. Cook, and Emre Yildiz

SP- 342-10:

Numerical Modeling Methodology for Strength Evaluation of Deep Bridge Bent Caps 162-177
Authors: Anish Sharma and Serhan Guner

SP- 342-11:

Seismic Simulation of Bridges Considering Bending and Torsion Interaction 178-196
Authors: Yang Yang and Ruili He

Inelastic Shear Distribution in Prestressed Concrete Girder Bridges

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

Synopsis: An experimental investigation was conducted on a full-scale prestressed concrete girder laboratory bridge to determine whether linear elastic shear distribution principles are conservative for load rating at ultimate capacity. A secondary goal was to determine whether existing web-shear cracks would be visible in an unloaded state. Two tests were conducted to failure (one near the end with a partial-depth diaphragm and one near the end without) to determine if the most loaded interior girder shed shear force to adjacent girders as it transitioned from uncracked to cracked to failure. Failure during each test was characterized by web-shear crushing and bridge deck punching at the peak applied load. Differences in the behavior of the two ends (with and without partial depth end diaphragm) affected the diagonal crack pattern, shear distribution, and loads at cracking and failure. The effect on loading was less than 10%. Inelastic shear distribution results indicated the girder carrying the most load redistributed shear to the other girders as it lost stiffness due to cracking. Use of linear elastic load distribution factors was conservative considering shear distribution at ultimate capacity. The visibility of web-shear cracks in an unloaded state was found to be a function of stirrup spacing.

Keywords: shear distribution, inelastic behavior, failure, concrete bridge, load testing, prestressed concrete

MOTIVATION AND BACKGROUND

Highway bridge owners regularly assign load ratings to bridge girders, which reflect the capacity of the component to carry traffic. Establishing girder load ratings requires an estimate of the member capacity (along with the amount of deterioration over time) and the live load demand. The capacity is calculated considering ultimate behavior and multiplied by a resistance factor (e.g., ϕV_n). The live load demand on an individual girder is estimated with distribution factors, which are typically derived based on linear elastic analysis and approximate how the traffic load distributes through the bridge system to an individual girder.

Engineers typically rely on the American Association of State Highway Transportation Officials (AASHTO) Specifications to assign load ratings and evaluate shear behavior. However, AASHTO requirements for shear have changed significantly over the years. As a result, some prestressed concrete girder bridges designed with previous AASHTO standards rate poorly for shear using current AASHTO standards, despite the fact that the girders may show no signs of distress under normal traffic loading conditions. Thus, the girders are often deemed to be in good condition, and therefore, the resulting shear rating may be neglected as outlined in Section 6A.5.8 of the AASHTO Manual for Bridge Evaluation (MBE) (2011), which states that “in-service concrete bridges that show no visible signs of shear distress need not be checked for shear when rating for the design load or legal loads.”

The primary goal of this research was to experimentally determine if an interior bridge girder shed shear force to adjacent girders as that beam transitioned from uncracked to cracked to failure. If shear force redistributed in the inelastic range of behavior after cracking and before failure, an inherent factor of safety may exist and use of linear elastic load distribution factors may be conservative when considering shear distribution at ultimate capacity. A secondary goal was to determine if initial web-shear cracking was visible in an unloaded state.

Load Rating with Elastic and Inelastic Principles

The methodology behind evaluation of existing bridges is transitioning from load factor rating (LFR), which aligned with the AASHTO Standard Specifications, to load and resistance factor rating (LRFR), which aligns with the AASHTO LRFD Specifications. While there are several differences between the rating factor (RF) equations for LFR and LRFR (e.g., nomenclature changes, separation of dead load by type), the general structure of the equation remains the same and is shown in Eqn. 1.

$$\text{Shear RF} = \frac{(\text{Resistance Factor}) * (\text{Shear Capacity}) - (\text{Load Factor}) * (\text{Dead Load})}{(\text{Load Factor}) * (\text{Live Load Shear Demand}) * (\text{Impact Factor})} \quad (1)$$

There is one key assumption present in both LFR and LRFR methodologies that is subtle and embedded in the calculation of the capacity and the live load. Calculation of a shear rating factor requires knowledge of the shear capacity at the *ultimate limit state* and knowledge of the live load shear demand on an individual girder estimated with distribution factors based on *linear elastic* analysis. Use of ultimate shear capacity and elastic distribution factors in load rating mixes principles related to elastic versus inelastic structural behavior.

Elastic and Inelastic Shear Distribution

The first load distribution principles for concrete slabs and beams published in the AASHTO Standard Specifications (1931) were developed by Westergaard (1930), confirmed by Newmark et al. (1946), and were based on elastic plate theory. The AASHTO Standard Specifications (2002) required use of the lever rule or “S-over” equations to calculate shear distribution factors. The lever rule assumes that the bridge deck is simply supported (hinged) over the interior girders in any cross section. At exterior girders, it is assumed that the deck panel is continuous with the overhang, which simulates a propped cantilever. These assumptions make the deck cross section statically determinate and the support reactions (i.e., distribution of shear among girders) can be readily calculated. The “S-over” equations were expressed in an *S/D* format, where *S* is the girder spacing in feet and *D* is a constant value for prestressed concrete girders of 7.0 and 5.5 for one lane loaded and two lanes loaded, respectively. Equations developed by Zokaie et al. (1991a; 1991b) are used for shear distribution factors in the AASHTO LRFD Specifications (2017). These equations are dependent on the girder spacing and were developed using linear elastic frame and shell finite element models loaded with the HS20 truck. The LRFD equations were calibrated against a database of constructed bridges to verify their applicability and generally produced results within five percent of those from a detailed finite-element analysis (Zokaie, 1991b).

A few full-scale destructive tests of non-prestressed concrete girder bridges have been performed since 1970 (Burdette and Goodpasture, 1973; Jorgenson and Lawson, 1976; Miller et al., 1994; Bechtel et al., 2011; Zhang et al., 2013). A summary of 40 tests to failure on 30 concrete bridges by Bagge et al. (2018) focused on the lessons learned during these experiments. Results summarized by Bagge et al. indicated that the theoretical load-carrying capacity of bridges (and inherently individual girders within the system) based on methods traditionally used for design and assessment provided conservative estimates, which corroborated Bechtel et al. (2011) who indicated that bridges generally had greater capacity than those predicted with AASHTO design and rating techniques. This concept of greater than predicted capacity was further reinforced by Araujo and Cai (2006) who found that current bridge rating methods considerably underestimated the predicted flexural capacity of a prestressed concrete girder bridge using three-dimensional (3D) finite element model (FEM) results validated in the elastic range. The observed reserve strength relative to predicted capacity may be attributed to the fact that current design and rating procedures use elastic distribution and consider the resistance of individual members at the component level rather than at the system level, where load redistribution occurs during inelastic behavior (Bechtel et al., 2011). A few studies specifically testing prestressed concrete girder bridges to failure in the laboratory or field have been conducted (Burdette and Goodpasture, 1974; Dymond et al., 2016; Amir et al., 2016; Ensink et al., 2018; Murray et al., 2019). Furthermore, there is a dearth of research on live load shear distribution in prestressed concrete girder bridges in the inelastic range and near failure.

Description of Study

Two tests on a full-scale prestressed concrete girder laboratory bridge were conducted in two diagonally opposite quadrants, one with a partial-depth end diaphragm and one without. These tests were used to determine if an interior girder shed shear force to adjacent girders as the beam carrying the most load transitioned from uncracked to cracked to failure. This study was motivated by interest in investigating whether the use of linear elastic load distribution factors was conservative considering shear distribution at ultimate capacity. Instrumentation and visual inspection were used to investigate the shear distribution and web-shear cracking patterns. The visibility of web-shear cracks in an unloaded state was studied and compared to observations from other studies.

DESCRIPTION OF LABORATORY TESTS

Bridge Design and Test Setup

A full-scale prestressed concrete girder highway bridge was constructed in the Theodore V. Galambos Structural Engineering Laboratory at the University of Minnesota Twin Cities to be representative of end span structures designed in the 1960's, 1970's, and early 1980's. The girders were designed using the AASHTO Standard Specifications (1989) with 1991 Interim Revisions and the Precast/Prestressed Concrete Institute (PCI) Bridge Design Manual (1997). The laboratory bridge was constructed with the following features:

- Four 91.4 cm (36 in.) deep bridge girders, similar to the AASHTO Type II shape, spaced at 2.74 m (9 ft) center-to-center and supported by elastomeric bearing pads.
- 22.86 cm (9 in.) thick concrete deck on a 2.54 cm (1 in.) thick haunch.
- Bridge length of 9.75 m (32 ft), aspect ratio (length/width) of 0.94, and span length of 9.37 m (30 ft 9 in.) measured between bearing centerlines with zero degrees skew. These dimensions were similar to geometry of typical approach spans investigated by Dymond et al. (2018).
- Partial-depth end diaphragm on one end of the bridge, no diaphragm on the other end of the bridge.
- No lifting hooks were installed in the girders to avoid adding reinforcement near the ends of the girders where the shear demand was expected to be highest.

A shear span-to-composite height ratio (a/h) of 2.5 was used during testing to avoid developing arching action as described by Hawkins et al. (2005). This was equivalent to a shear span-to-composite depth ratio (a/d_v) of 3.1, where d_v was the effective depth at the critical section for shear. The bridge girders were designed using a capacity design approach. The maximum load that could be applied during testing was limited to the hydraulic testing limit of 1957 kN (440 kips). To avoid a flexural failure, the girder flexural capacity was designed to be larger than the maximum flexural demand from the applied load, assuming the largest AASHTO Standard flexural distribution factor considering both one and two lanes loaded. To ensure a shear failure within the range of available applied load, it was assumed that the girder shear capacity at ultimate was 30% greater than would be predicted using AASHTO Specifications, which was based on the findings of Hawkins et al. (2005) and confirmed with results from Runzel et al. (2007). Furthermore, the shear demand expected to fail a girder in the bridge was conservatively estimated using the smallest AASHTO Standard shear distribution factor considering both one and two lanes loaded.

Fig. 1 shows the bridge framing plan and location of the end diaphragm. The end diaphragm concrete was placed simultaneously with the bridge deck. Many full-depth end diaphragms *in-situ* are cracked at the interface of the girder and end diaphragm. These cracking patterns are particularly evident in semi-integral abutment bridges where the upward girder deflection due to solar radiation may be constrained by the end diaphragm. Use of an uncracked full-depth end diaphragm in the laboratory was not considered representative of full-depth end diaphragm behavior in the field. Cracked full-depth end diaphragms may have a stiffness value and behavior between an uncracked full-depth end diaphragm and a partial-depth end diaphragm so a partial-depth end diaphragm was selected for the study instead of a full-depth end diaphragm. A realistic design compressive strength for a 30-year-old bridge girder was targeted to be 51.7 MPa (7500 psi) considering concrete compressive strength gain over time (Wood, 1991; Dereli et al., 2010). This specified 28-day concrete compressive strength was the current expected strength of the younger girders in the field that were found to rate poorly for shear from the 1980's. Furthermore, the specified 28-day concrete compressive strength for the bridge deck was 4,000 psi, which was representative of replaced decks on these structures. Additional material properties and design details for the girders, deck, and partial-depth end diaphragm can be found in Table 1 and Fig. 2 through 6.

The laboratory bridge was tested to the inelastic range of behavior in two diagonally opposite quadrants, shown in Fig. 1. The first test occurred in the southwest quadrant closer to the end diaphragm. Load was applied directly over interior Girder 3 and Girders 2 and 4 were adjacent to the applied load. The second test occurred in the northeast quadrant away from the end diaphragm and included any damage incurred in the structure during the first test. Load was applied directly over interior Girder 2 and Girders 1 and 3 were adjacent to the applied load. In both tests, load was applied a distance of 2.92 m (9 ft 7 in.) from the centerline of the nearest girder support and 6.45 m (21 ft 2 in.) from the centerline of the farthest girder support. A rectangular patch load that was 30.5 cm (12 in.) wide longitudinally by 91.4 cm (36 in.) wide transversely was applied to the structure using a combination of two 489 kN (110 kip) actuators and one 978 kN (220 kip) actuator suspended from a steel load frame as shown in Fig. 7. The patch area was not the same as the AASHTO tire patch dimensions, but the applied load was much higher in magnitude than an AASHTO tire and was not meant to represent truck tires.

Behavior Captured with Instrumentation

Several types of structural behavior were captured using instrumentation during testing of the bridge including shear strains in the stirrups and shear strains in the girder webs. Stirrup strains were measured with foil strain gages installed at three vertical locations (evenly distributed through the depth of the girder web) on one leg of five stirrups at the end of the girder nearest the applied load. Shear strain on a vertical face, γ_{xy} , was calculated using rosette strain data from foil and vibrating wire strain gages (VWSG) installed at multiple positions along the length of the girders. The rosette gage locations were measured relative to the critical shear section, d_v , from the interior edge of the sole plate. The value of d_v for the composite cross section was approximately 94 cm (37 in.). Specifically, two types of rosette strain gages were installed at $0.5d_v$, d_v , and $2d_v$ at the end of the girders nearest the applied load and at d_v on the opposite end of the girders, farthest from the applied load. Sets of four VWSGs were installed on the surface to form a box-type rosette, shown in Fig. 8. The box-type rosette allowed for linear interpolation of strain between the two horizontal gages such that, when incorporated with the single vertical and diagonal gage, three directions of a 45-degree rosette strain measurement were captured at the center of the box configuration. This configuration assumed that the vertical strain did not vary significantly over a small longitudinal distance. The box-type rosette was configured on girder webs such that the diagonal strain gage was parallel to the principal compressive stress from the applied load to maximize the reading. Foil strain gage rosettes were installed on the surface at the same longitudinal positions as the VWSG rosettes but were located on the opposite face of the girder web. Data collected from the VWSG box-type rosettes and the foil rosettes were averaged to negate the effects of torsion across the width of the girder web and to calculate shear strain due to the vertical shear resultant as discussed by Dymond et al. (2018).

RESULTS

Inelastic Behavior of the Bridge

Load was applied to the bridge during each test in 111 kN (25 kip) increments. Initial web-shear cracks (observed first) and flexural cracks (observed second) were traced with permanent marker in the loaded girder during each test at the applied loads shown in Table 2. After observation of initial web-shear cracking, the bridge was unloaded to near zero applied load to determine if the cracks were still visible. The primary web-shear crack decreased in width but was still visible in the region of the girder with 61 cm (24 in.) stirrup spacing. After reloading, failure during each test was characterized by web-shear crushing and bridge deck punching, which were observed at the peak applied loads

shown in Table 2. Bridge deck punching occurred even though the applied patch load was centered directly above an interior girder. Bridge deck punching was also the failure mode observed during testing of prestressed concrete girder bridges by Ensink et al. (2018) and Murray et al. (2019) when load was applied directly above a girder. Crack patterns and web-shear deterioration observed in the loaded girder can be seen in Fig. 9 and 10 for testing with and without a partial-depth end diaphragm, respectively. During each test, no web-shear or flexural cracks were observed in any of the adjacent girders. Stirrup strain gages within the damaged portion of the shear span exceeded the predicted yield strain, and the lack of web-shear cracking in adjacent girders was confirmed with stirrup strain gage data that indicated near zero tensile strain in the stirrups throughout each test (Dymond et al., 2016). Typical bridge deck punching observed from below and above the slab are shown in Fig. 11 and 12, respectively. The order in which deck punching and web-shear crushing occurred was not directly observed as the two events happened nearly simultaneously. The ability of an interior girder to shed shear demand to adjacent girders during inelastic loading, particularly after individual girder failure, was of interest. However, deck punching precluded further load redistribution after failure of the interior girder. The patch load of approximately 2 MN (450 kips) that caused bridge deck punching and web-shear crushing was much larger than the patch loads associated with the individual wheel loads of a single vehicle.

Table 2 also shows data from testing of an independent single companion girder; the companion girder was identical to the bridge girders and fabricated at the same time. The companion girder had a 9 in. thick by 4.5 ft wide composite deck placed on top ($f'_c = 58$ MPa or 8,414 psi) compared to the 9 ft girder spacing in the bridge. Load was applied to both the companion girder and the bridge at the same distance from the centerline of the nearest girder support (2.92 m or 9 ft 7 in.). Thus, the shear span-to-composite height ratio (a/h) of 2.5 and the shear span-to-composite depth ratio (a/d_v) of 3.1 were identical in both the companion girder and full bridge test setups. Additional information about the companion girder can be found in Dymond et al. (2016).

Shear Distribution in the Inelastic Range

Shear distribution behavior in the inelastic range was characterized using live load shear forces calculated from measured rosette strain gage data obtained on the girder webs at $0.5d_v$, d_v , and $2d_v$. The effects of dead load shear were not included. The composite girder and deck self-weight created a reaction of approximately 101 kN (22.7 kips) per girder. Additional material and cross-sectional properties used for calculation of elastic shear force in the short shear span are given in Table 3. When the loaded girder developed web-shear cracking in the web, the rosette strain gage instrumentation was no longer used to calculate shear force in the short shear span on the cracked girder. However, the shear force in the short shear span of the damaged girder was calculated by subtracting the sum of calculated shear forces in the short shear span in the remaining undamaged girders from the total shear force in the short shear span. The total shear force in the short shear span was calculated using statics and a beam line analysis with a single applied patch load, where the bridge was idealized as a one-dimensional (1D) structure along its length to determine the shear across a section of the bridge. This technique of “calculating response” to characterize shear distribution was used to near failure, just prior to deck punching. This methodology was deemed reliable because the other girders remained elastic (no web-shear cracking observed and near zero tensile strain in the stirrups).

The total applied load versus the shear force in the short shear span calculated using data from rosette strain gages at d_v is shown in Fig. 13 and 14 for testing with and without an end diaphragm, respectively. In both figures, the calculated interior girder response, highlighted with an arrow labeled “calculated response,” was plotted from the initial loading step to the final loading step (just prior to failure). In the elastic range, two methods were used to compare the shear force in the interior girder: (1) it was determined by subtracting the sum of calculated shear forces in the remaining three girders from the total cross-sectional shear force as previously described above, and (2) it was derived using the strain rosette on the interior girder. Both methods produced similar results. This comparison was used in the elastic range to validate how the shear force was determined in the inelastic range. Changes in the slope of the response for each girder occurred approximately when web-shear cracking was observed in the loaded girder and indicated changes in transverse shear distribution.

At the critical section, d_v , Fig. 13 and 14 show that an average of approximately 89 kN and 98 kN (20 and 22 kips) of shear force in the short shear span (out of approximately 1.29 MN or 290 kips total shear) were redistributed to each girder directly adjacent to focus beams G3 and G2, respectively, as the loaded beam cracked and failed with and without an end diaphragm, respectively. This magnitude of shear force redistribution was approximately the same to each adjacent girder. This behavior is identified with an arrow labeled “Load difference” spanning between the adjacent girder data and a dashed line approximating the linear response. The linear elastic response of the loaded girder was extrapolated beyond cracking using a best fit line of the linear elastic data, from 222 kN to 667 kN (50 to

150 kips) applied load, to highlight the loaded girder loss of stiffness and shear redistribution when the slope of the applied load versus shear force data changed after web-shear cracking. This extrapolated behavior is identified with a dotted line labeled “Extrapolated linear response.” Furthermore, an assumption of bilinear behavior in the loaded girder data (linear elastic until web-shear cracking followed by linear behavior with redistribution of shear as the damaged girder stiffness decreased) was used to calculate the shear force at failure. This bilinear behavior is identified with a dotted line labeled “Bilinear behavior.”

As noted in the previous section, deck punching precluded further load redistribution after failure of the interior girder. Consequently, the live load applied when the bridge deck punched may not have been the load that was required to cause ultimate shear failure in the interior girder alone because the interior girder may have carried more shear demand or the bridge system may have redistributed more shear demand if the applied live load was more representative of distributed wheel loads. Comparison of AASHTO shear distribution factors to laboratory data were not made because AASHTO distribution factors are not intended for distributing the bridge cross-sectional live load shear demand due to a single patch load to an individual girder.

Effects of the Partial-Depth End Diaphragm

Quantitatively, the ratio of live load shear demand with no end diaphragm to live load shear demand with an end diaphragm, shown in Table 2, indicated that the end diaphragm had a minimal effect on the cracking loads and the failure loads in this study when load was applied directly above an interior girder. The live load shear demand at observed web-shear cracking, observed flexural cracking, and failure were slightly lower with the end diaphragm. Data in Fig. 13 and 14 indicated that near failure, load had been redistributed in tests with and without an end diaphragm to approximately the same shear force at the critical section of the loaded girder (hence the similar peak loads at failure). This behavior indicated that the shear force redistributed such that the behavior was similar with and without an end diaphragm near failure.

Qualitatively, Fig. 9 and 10 show that the web-shear cracking pattern was different for the tests with and without an end diaphragm, respectively. Fig. 9 shows that the end diaphragm focused diagonal shear cracking higher in the web and toward the support centerline rather than toward the face of the support. To help visualize this behavior, the location of the first four stirrups are indicated with an arrow and label in Fig. 9 and 10. The web-shear cracking pattern at the end with a partial-depth end diaphragm (Fig. 9) extensively penetrated the top flange and engaged more of the bridge deck above the web along the shear span where three of the four stirrups are highlighted. However, the web-shear cracking pattern at the end without a partial-depth diaphragm (Fig. 10) only penetrated the top flange and engaged the bridge deck above the web near the applied load where two of the four stirrups are highlighted. Furthermore, Fig. 10 shows bottom flange section loss at failure where the concrete spalled off near the bottom layer of prestressing strands at the interior face of the support. The bottom flange section loss did not occur while testing with an end diaphragm (first chronologically); this may have been due to the fact that loading was stopped immediately after failure to preserve the structure for future testing without an end diaphragm. Bottom flange section loss may have occurred while testing without an end diaphragm because load continued to be applied after deck punching.

These cracking patterns corroborated with results presented by Dymond et al. (2016), who used finite element models with eight-node 3D linear continuum elements that were assigned the measured material properties listed in Table 3 to characterize elastic shear load flow through the composite section. Loading in the elastic FEM was also applied directly above an interior girder but at a distance of $4d_v$ from the support. Results indicated that the end diaphragm caused more shear to be carried to the end of the span via the deck rather than the girder web. This behavior, observed with results from an upper bound stiffened diaphragm case (10 times the measured Young’s modulus), indicated that more shear remained in the deck until the very end of the span, near the reaction, and transferred to the support through the end diaphragm or the girder web at the very end of the span.

Visibility of Initial Web-Shear Cracks while Unloaded

Bridge girder web-shear cracks that develop under the presence of an overload may not be visible in the absence of the overload when the structure is subjected to regular traffic loads. Detailed visual inspections of loaded girders during field tests by Dymond et al. (2016) and during bridge inspections by Dereli et al. (2010) revealed that no web-shear cracks were observed when girders in bridges that rate poorly for shear were subjected to routine traffic or loaded dump trucks. Testing of the laboratory bridge provided an opportunity to observe initial web-shear cracks in an unloaded state immediately after crack formation with a stirrup spacing of 61 cm (24 in.). During tests with and without an end diaphragm, the applied load was decreased and held constant at a near zero value after initial web shear

cracking, and initial web-shear cracks remained visible upon unloading. However, observing these cracks would have been very difficult to detect if their locations had not been marked with permanent marker when the load was applied. These results were consistent with web-shear crack behavior observed by Mathys et al. (2014). Mathys et al. found that web-shear cracks in areas with widely spaced stirrups (61 cm or 24 in.) were still visible upon unloading, and web-shear cracks in areas with closely spaced stirrups (20 cm or 8 in.) were not visible upon unloading. The closer stirrup spacing provided additional crack closing force, closing those cracks completely upon unloading.

CONCLUSIONS

A full-scale prestressed concrete girder bridge was constructed with a partial-depth end diaphragm on one end. The bridge was tested into the inelastic range of behavior in the laboratory on diagonally opposite ends to investigate if an interior girder shed shear force to adjacent beams as that girder transitioned from uncracked to cracked to failure. The two tests provided an opportunity to investigate the effect of the partial-depth end diaphragm and no end diaphragm on shear distribution. Furthermore, the visibility of existing web-shear cracks in an unloaded state was studied.

For this laboratory bridge geometry and loading scenario, it was shown that the partial-depth end diaphragm had a small effect on the cracking and failure loads. Differences in the live load shear demand at observed web-shear cracking, observed flexural cracking, and failure were less than 10% during testing with and without a partial-depth end diaphragm. Similar peak applied loads at failure indicated that the shear force redistribution that occurred near failure was similar with and without a partial-depth end diaphragm. Failure during each test was characterized by web-shear crushing and bridge deck punching, which occurred at the peak applied load. Bridge deck punching occurred even though the applied patch load was centered directly above an interior girder. The partial-depth end diaphragm also focused diagonal shear cracking higher in the web and toward the support centerline rather than toward the face of the support. The web-shear cracking pattern between the support centerline and the location of applied load at failure extensively penetrated the top flange and engaged more of the bridge deck above the web when the partial-depth end diaphragm was present whereas the web-shear cracking pattern was mainly in the girder web between the face of the support and the location of applied load when the partial-depth end diaphragm was not present.

The web-shear cracking results from this study and from Mathys et al. (2014) indicated that web-shear cracks that may form due to an overload in the field might not be visible upon inspection if the load that caused the crack is removed. Existing web-shear cracks remained visible when the stirrup spacing was wide (61 cm or 24 in.), but noting the location of the cracks at a low load level was still difficult. More importantly, Mathys et al. (2014) stated that initial web-shear cracks did not remain visible when the stirrup spacing was small (20 cm or 8 in.). Laboratory researchers attempting to locate web-shear cracks have the benefits of high levels of focused lighting, multiple sets of experienced eyes, and limited outside distractions. Experienced bridge inspection engineers trying to locate a web-shear crack in the field complete the task under harsher conditions and have the added duty of inspecting the entire structure with limited resources. To this end, foregoing a poor shear rating because of no visual signs of shear distress as outlined in the AASHTO Manual for Bridge Evaluation (2011) may be unconservative and may lead to an unsafe practice of permitting heavier vehicles to cross the structure.

During inelastic laboratory testing in this project, data collected prior to failure indicated that the live load shear demand in the loaded interior girder redistributed after observation of initial web-shear and flexural cracks. The redistribution of shear continued to increase as the damaged girder stiffness decreased. Therefore, an inherent factor of safety existed in the laboratory bridge for the live load shear demand carried by the interior girder between elastic and inelastic behavior. This illustrates conservatism in the AASHTO LRFD Specifications, which uses linear elastic analysis to determine the shear load distribution at the ultimate limit state. The factor of safety observed in this study for inelastic redistribution may be reduced in bridge geometries with a wide girder spacing where the shear force must distribute over a longer transverse distance.

REFERENCES

- American Association of State and Highway Transportation Officials (AASHTO). (1931). *Standard Specifications for Highway Bridges and Incidental Structures*, 1st Edition, Washington, D.C.
- American Association of State and Highway Transportation Officials (AASHTO). (1989). *Standard Specifications for Highway Bridges*, 14th Edition with 1991 Interim Specifications, Washington, D.C.
- American Association of State and Highway Transportation Officials (AASHTO). (2002). *Standard Specifications for Highway Bridges*, 17th Edition, Washington, D.C.
- American Association of State Highway and Transportation Officials (AASHTO) (2017). *AASHTO LRFD Bridge Design Specifications*, 8th Edition, Washington, DC.
- American Association of State Highway and Transportation Officials (AASHTO) (2011). *The Manual for Bridge Evaluation*, 2nd Edition, Washington, DC.
- Amir, S., van der Veen, C., Walraven, J. C., and de Boer, A. (2016). "Experiments on Punching Shear Behavior of Prestressed Concrete Bridge Decks." *ACI Structural Journal*, 113(3), 627–636.
- Araujo, M. and Cai, C. S. (2006). "Performance of Prestressed Concrete Bridges – Evolution from Elastic to Failure Stages." *Structures Congress: Structural Engineering and Public Safety*, St. Louis, MO.
- Bagge, N., Popescu, C., and Elfgren, L. (2018). "Failure Tests on Concrete Bridges: Have We Learnt the Lessons?" *Structure and Infrastructure Engineering*, (14)3, 292–319.
- Bechtel, A., McConnell, J., and Chajes, M. (2011). "Ultimate Capacity Destructive Testing and Finite-Element Analysis of Steel I-Girder Bridges." *Journal of Bridge Engineering*, 16(2), 197–206.
- Burdette, E. G., and Goodpasture, D. W. (1973). "Tests of Four Highway Bridges to Failure." *Journal of the Structural Division*, 99(3), 335–348.
- Burdette, E. G., and Goodpasture, D. W. (1974). "Test to Failure of a Prestressed Concrete Bridge." *PCI Journal*, 19(3), 92–103.
- Dereli, O., Shield, C. K., and French, C. (2010). "Discrepancies in Shear Strength of Prestressed Beams with Different Specifications." *Report No. MN/RC 2010-03*, Department of Civil Engineering, University of Minnesota, Minneapolis, MN, pp. 242.
- Dymond, B. Z., French, C. W., Shield, C. K. (2016). "Investigation of Shear Distribution Factors in Prestressed Concrete Girder Bridges." *Report No. MN/RC 2016-32*, Department of Civil, Environmental, and Geo-Engineering, University of Minnesota, Minneapolis, MN, pp. 595.
- Dymond, B. Z., French, C. E. W., & Shield, C. K. (2018). Torsional effects on load tests to quantify shear distribution in prestressed concrete girder bridges. ACI SP 323-10: Evaluation of Concrete Bridge Behavior through Load Testing - International Perspectives, 10.1–10.18.
- Ensink, S. W. H., van der Veen, C., Hordijk, D. A., Lantsoght, E. O. L., van der Ham, H., and de Boer, A. (2018). "Full-Size Field Test of Prestressed Concrete T-Beam Bridge." *Proceedings Structural Faults and Repair and European Bridge Conference*, Edinburgh, Scotland.
- Hawkins, N. M., Kuchma, D. A., Mast, R. F., Marsh, M. L., and Reineck, K-H. (2005). "Simplified Shear Design of Structural Concrete Members." National Cooperative Highway Research Program (NCHRP) Report 549, Transportation Research Board, Washington, D.C.
- Jorgenson, J. L., and Lawson, W. (1976). "Field Testing of a Reinforced Concrete Highway Bridge to Collapse." *Transportation Research Record 607*, Transportation Research Board, Washington, D.C., 66–71.
- Miller, R. A., Aktan, A. E., and Shahrooz, B. M. (1994). "Destructive Testing of Decommissioned Concrete Slab Bridge." *Journal of Structural Engineering*, 120(7), 2176–2198.
- Murray, C. D., Arancibia, M. D., Okumus, P., and Floyd, R. W. (2019). "Destructive Testing and Computer Modeling of a Scale Prestressed Concrete I-Girder Bridge." *Engineering Structures*, 183, 195–205.
- Newmark, N. M., Siess, C. P., and Peckham, R. R. (1946). "Studies of Slab and Beam Highway Bridges. Part I: Tests of Simple-Span Right I-Beam Bridges." *Bulletin Series No. 363*, Engineering Experiment Station, University of Illinois, Urbana, IL, pp. 132.
- Precast Prestressed Concrete Institute (PCI) (1997). *PCI Bridge Design Manual*, 1st Edition, Chicago, IL.
- Runzel, B., Shield, C. K., and French, C. W. (2007). "Shear Capacity of Prestressed Concrete Beams." *Report No. MN/RC 2007-47*, Department of Civil Engineering, University of Minnesota, Minneapolis, MN, pp. 237.
- Westergaard, H. M. (1930). "Computations of Stresses in Bridge Slabs Due to Wheel Loads." *Public Roads*, 11(1), 1–23.
- Wood, S. L. (1991). "Evaluation of the Long-Term Properties of Concrete." *Research and Development Bulletin RD102*, Portland Cement Association, Skokie, IL.

- Zhang J, Peng H, and Cai CS. (2013). "Destructive Testing of a Decommissioned Reinforced Concrete Bridge." *Journal of Bridge Engineering*, 18(6), 564-569.
- Zokaie, T., Osterkamp, T. A., and Imbsen, R. A. (1991a). "Distribution of Wheel Loads on Highway Bridges." *Transportation Research Record 1290, Volume 1*, Transportation Research Board, Washington, D.C., 119-126.
- Zokaie, T., Osterkamp, T. A., and Imbsen, R. A. (1991b). "Distribution of Wheel Loads on Highway Bridges." National Cooperative Highway Research Program Final Report 12-26/1, Transportation Research Board, Washington, D.C.

Table 1—Bridge girder properties used for design and the resulting shear capacity (V_n)

Property	Units	Value
Girder specified 28-day concrete compressive strength	MPa (ksi)	51.7 (7.5)
Girder specified release concrete compressive strength	MPa (ksi)	31.7 (4.6)
Deck specified 28-day concrete compressive strength	MPa (ksi)	27.6 (4)
Stirrup yield strength	MPa (ksi)	414 (60)
Stirrup modulus of elasticity	GPa (ksi)	200 (29000)
Strand specified tensile strength	MPa (ksi)	1,860 (270)
Strand modulus of elasticity	GPa (ksi)	197 (28500)
Design girder shear capacity (V_n) at half of the composite depth from the support centerline ($h_c/2$)	kN (k)	676 (152)

Table 2—Applied live load and bridge cross-sectional live load shear demand recorded during testing

Behavior ¹	Applied Patch Live Load, kN (k)		Bridge Cross-Sectional Live Load Shear Demand, kN (k)			Companion Girder ² Live Load Shear Demand, kN (k)
	End Dia.	No End Dia.	End Dia.	No End Dia.	No End Dia. / End Dia.	
Observed web-shear cracking	1001 (225)	1032 (232)	689 (155)	710 (160)	1.03	552 (124)
Observed flexural cracking	1223 (275)	1334 (300)	842 (189)	919 (207)	1.09	721 (162)
Web-shear crushing and bridge deck punching	1975 (444)	2011 (452)	1358 (305)	1384 (311)	1.02	1041 (234)

¹ Bridge age at time of test with partial-depth end diaphragm = 529 days

Bridge age at time of test without partial-depth end diaphragm = 538 days

² A single companion girder identical to the bridge girders was tested independently. Load was applied at the same location during both the companion girder test and the bridge tests.**Table 3—Material properties used to calculate elastic shear forces**

Property	Units	Value
Girder Concrete Compressive Strength (measured)	MPa (ksi)	49.6 (7.2)
Girder Modulus of Elasticity (measured)	GPa (ksi)	34.5 (5000)
Girder Shear Modulus (calculated with $\nu = 0.2$)	GPa (ksi)	14.4 (2083)
Deck Concrete Compressive Strength (measured)	MPa (ksi)	45.5 (6.6)
Deck Modulus of Elasticity (measured)	MPa (ksi)	31.2 (4520)
Rebar Modulus of Elasticity	GPa (ksi)	200 (29000)
Strand Modulus of Elasticity	GPa (ksi)	198 (28700)
<i>Interior Girder</i>		
Deck Effective Flange Width	m (ft)	2.74 (9)
Composite Deck Longitudinal Area of Steel (top)	cm ² (in. ²)	7.74 (1.20)
Composite Deck Longitudinal Area of Steel (bottom)	cm ² (in. ²)	24 (3.72)
Transformed Composite Centroid (from bottom)	cm (in.)	85.1 (33.5)
Transformed Moment of Inertia	cm ⁴ (in. ⁴)	10420000 (250280)
First Moment of Area at Rosette Elevation	cm ³ (in. ³)	100250 (6118)
<i>Exterior Girder</i>		
Deck Effective Flange Width	m (ft)	2.44 (8)
Composite Deck Longitudinal Area of Steel (top)	cm ² (in. ²)	6.45 (1.00)
Composite Deck Longitudinal Area of Steel (bottom)	cm ² (in. ²)	20 (3.10)
Transformed Composite Centroid (from bottom)	cm (in.)	83.3 (32.8)
Transformed Moment of Inertia	cm ⁴ (in. ⁴)	10100000 (242524)
First Moment of Area at Rosette Elevation	cm ³ (in. ³)	98250 (5996)

Note: measured material properties were obtained on the day of testing shown in Table 2 and were the average of data from three concrete cylinders.

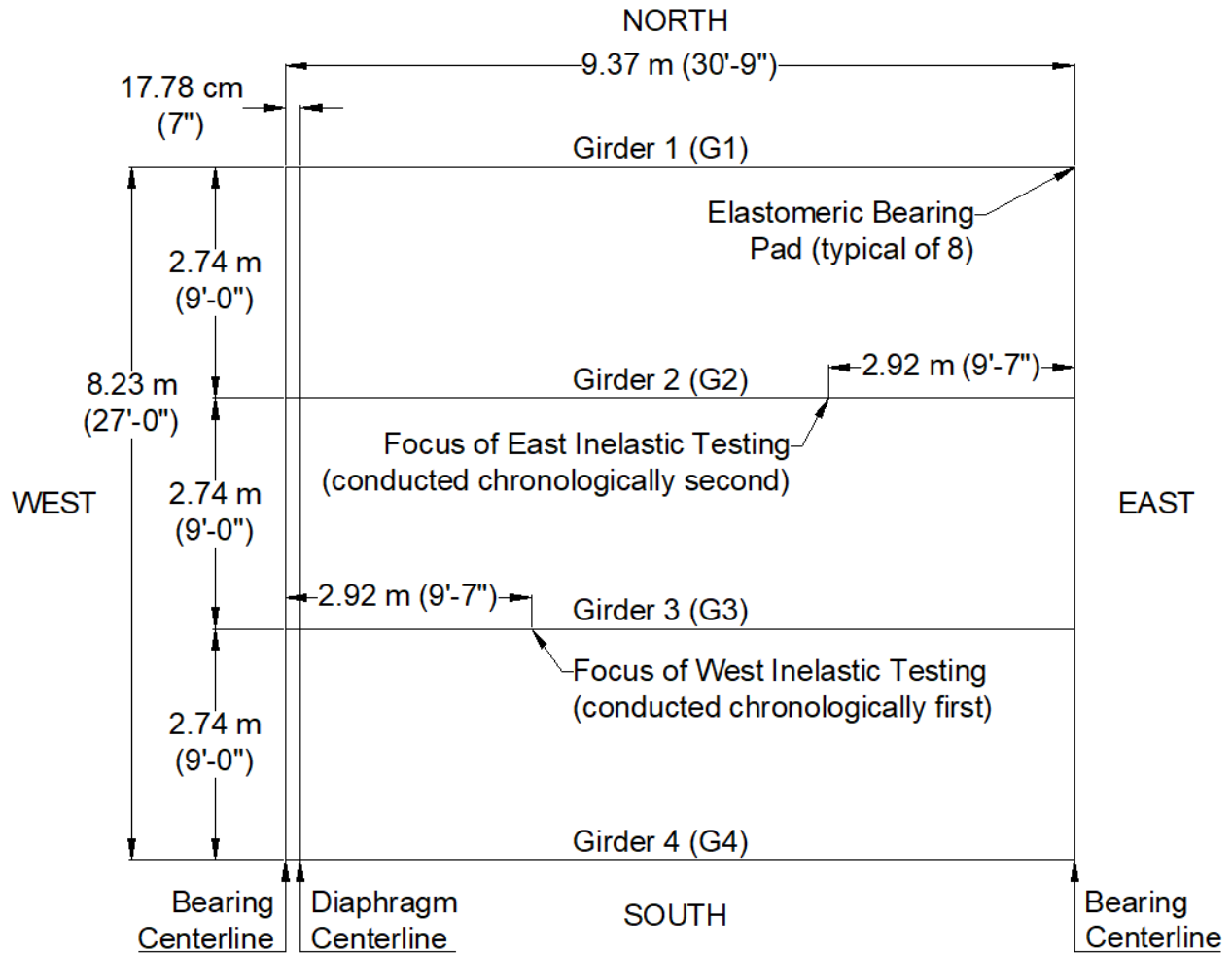


Fig. 1—Laboratory bridge framing plan and locations of inelastic testing

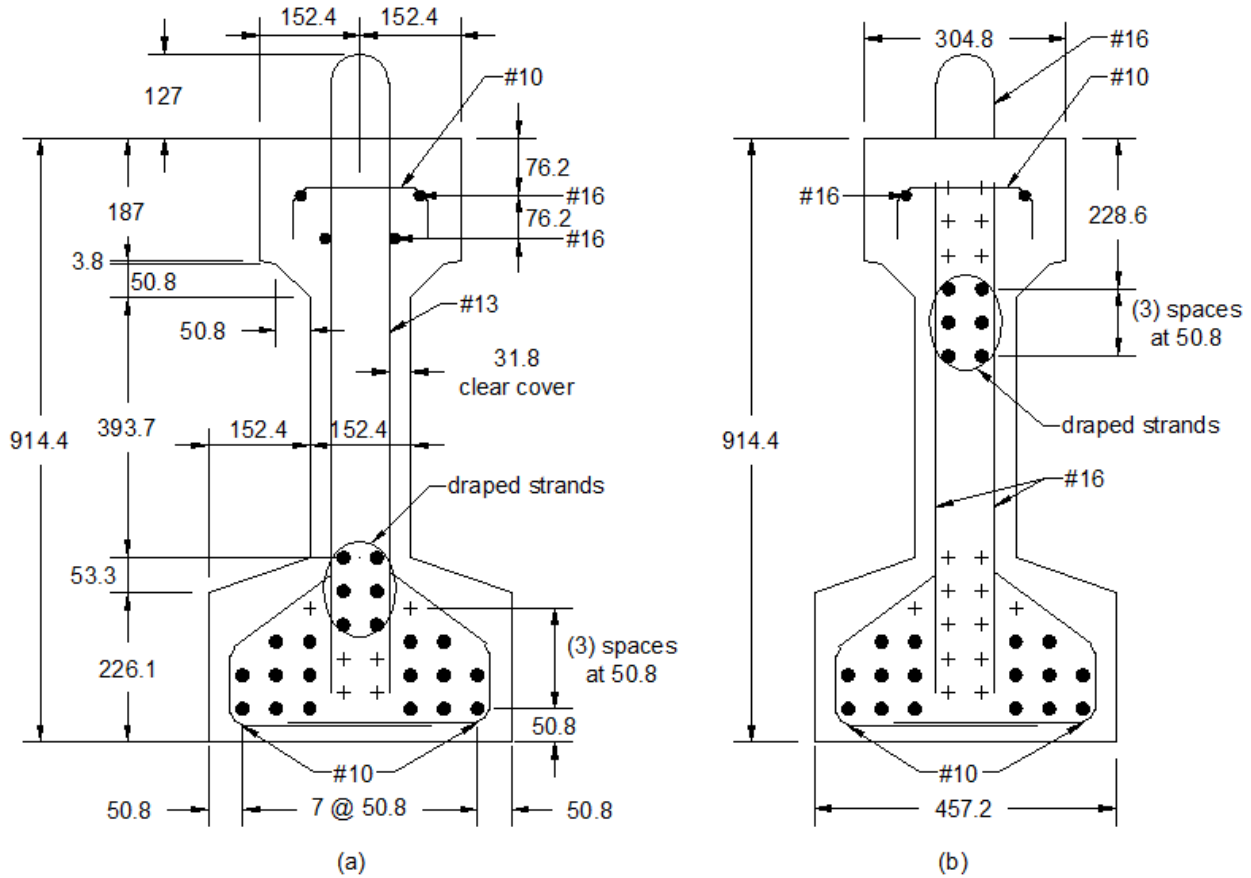


Fig. 2—Laboratory bridge girder (a) midspan and (b) end cross section details (mm dimensions); 1 in. = 25.4 mm

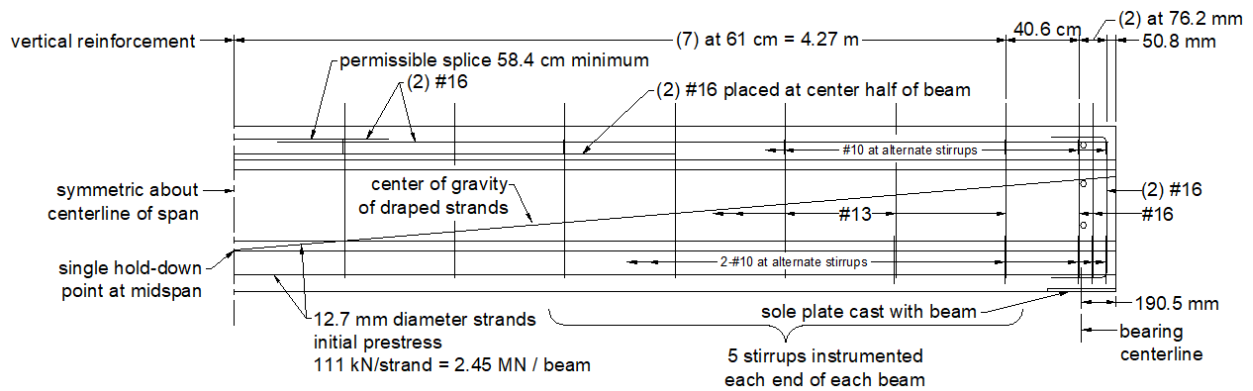


Fig. 3—Half laboratory bridge girder elevation; 1 in. = 25.4 mm and 1 kip = 4.45 kN

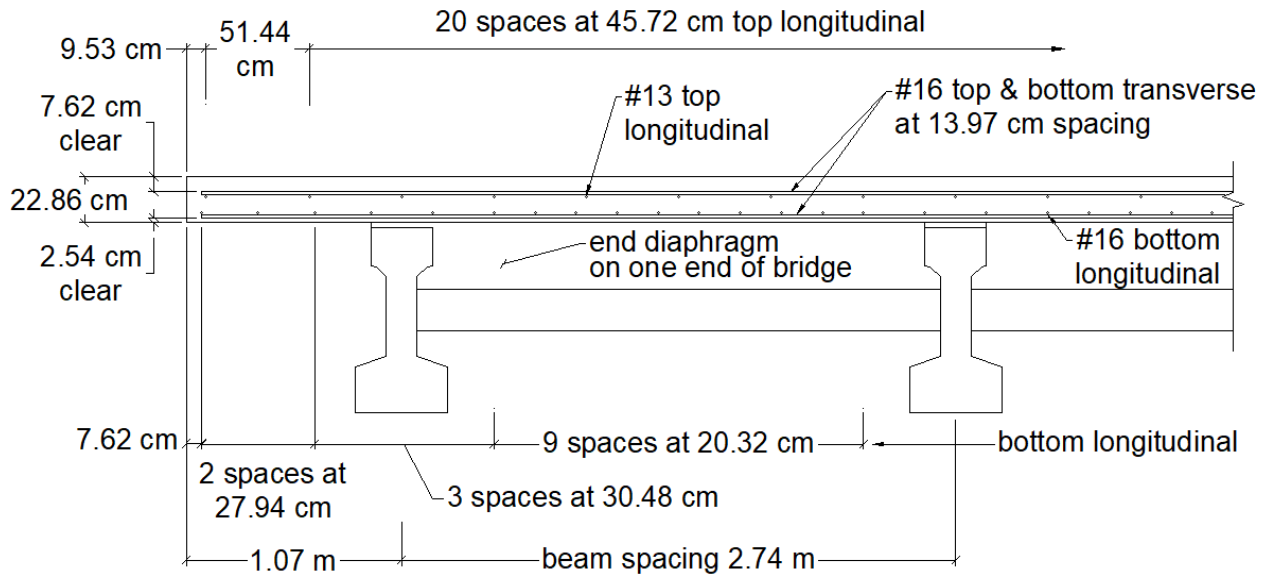


Fig. 4—Half laboratory bridge deck transverse cross section for inelastic testing; 1 in. = 25.4 mm

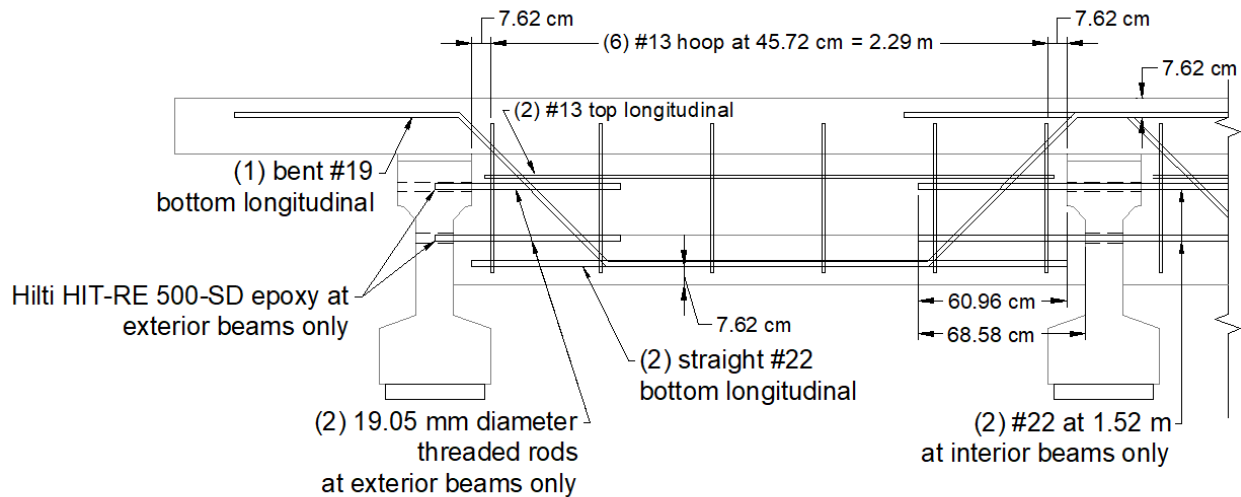


Fig. 5—Partial-depth end diaphragm transverse cross section; 1 in. = 25.4 mm

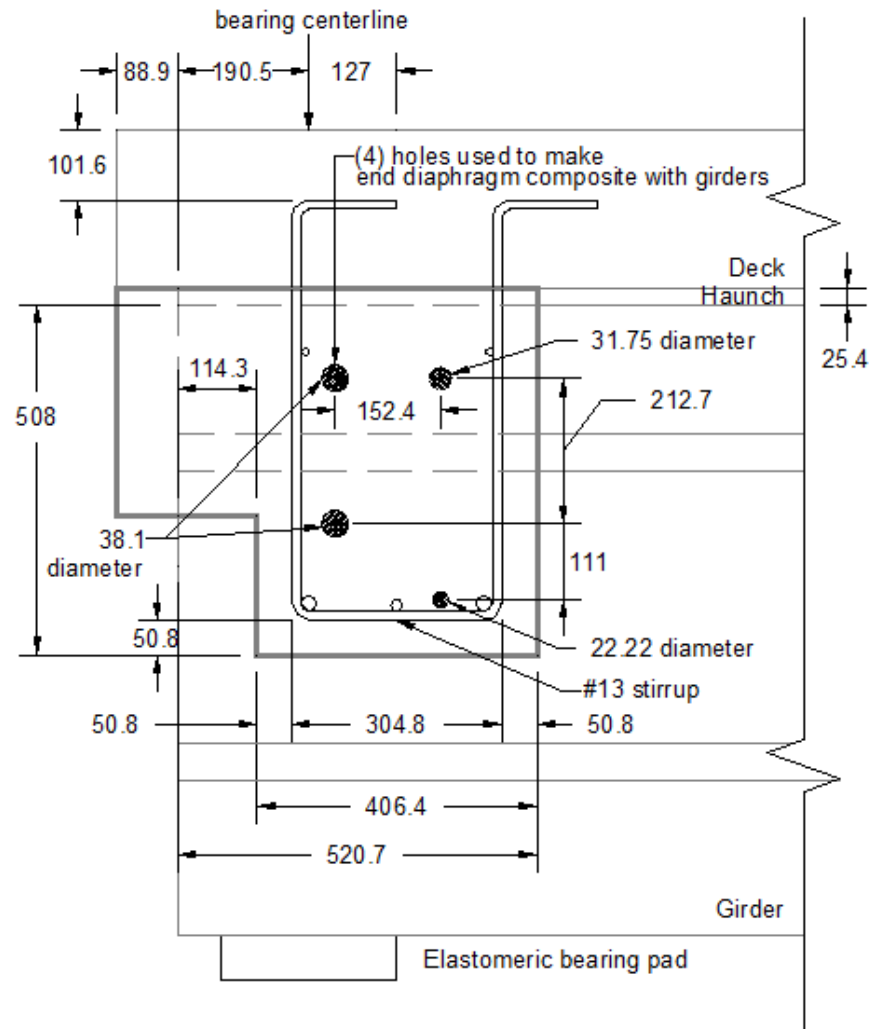


Fig. 6—Cross section of the partial-depth end diaphragm (mm dimensions); 1 in. = 25.4 mm



Fig. 7—Load frame and actuator setup above the deck in the northeast quadrant

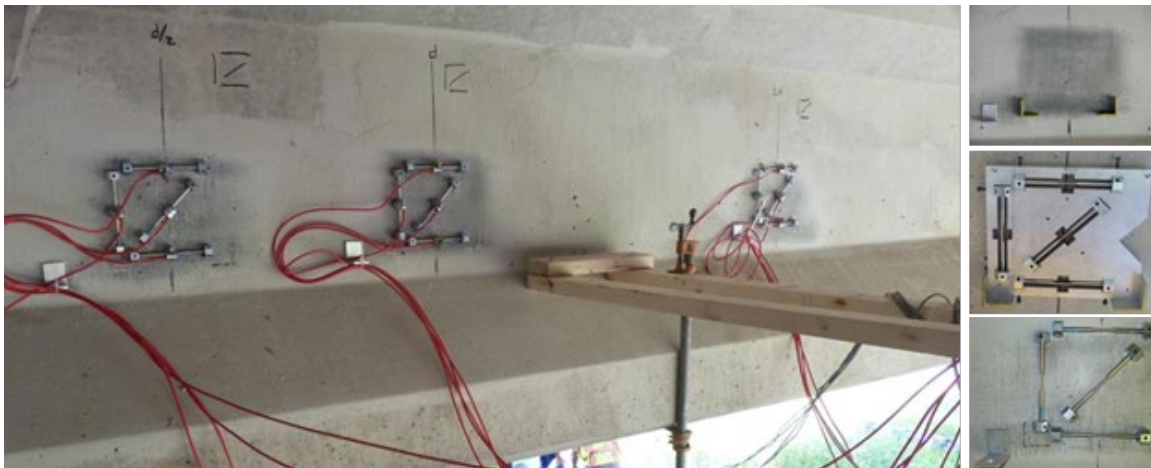


Fig. 8—Photo of box-type rosettes containing four VWSGs at $0.5d_v$, d_v , and $2d_v$ longitudinal locations

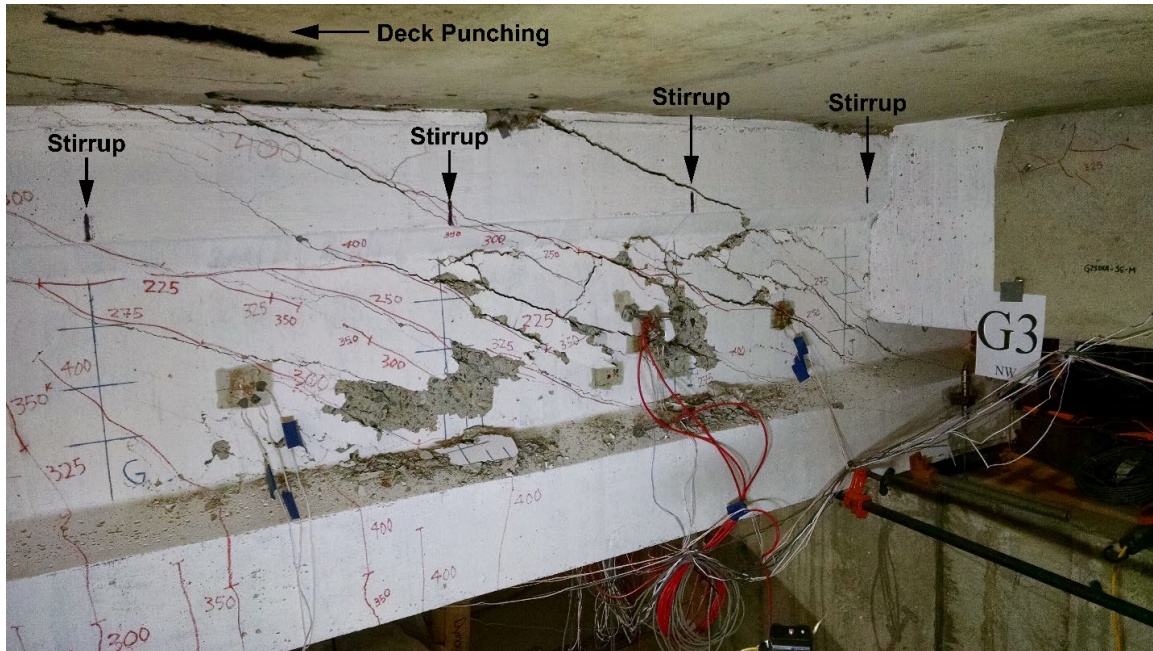


Fig. 9—Web-shear crushing and deck punching in the loaded girder (G3) during testing with a partial-depth end diaphragm

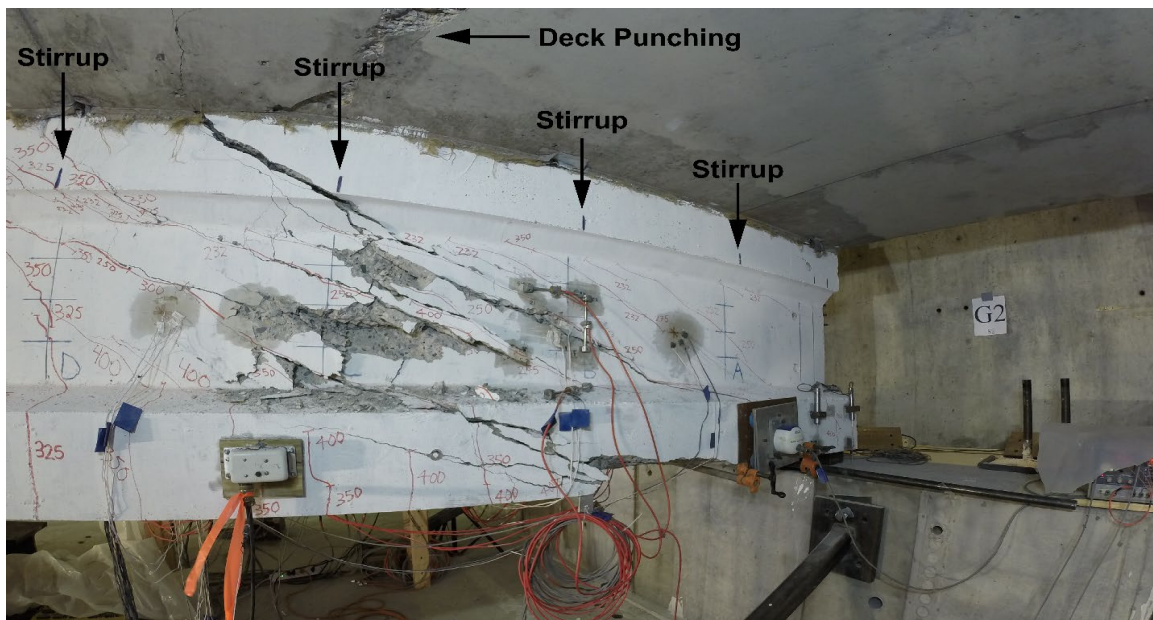


Fig. 10—Web-shear crushing and deck punching in the loaded girder (G2) during testing without a partial-depth end diaphragm

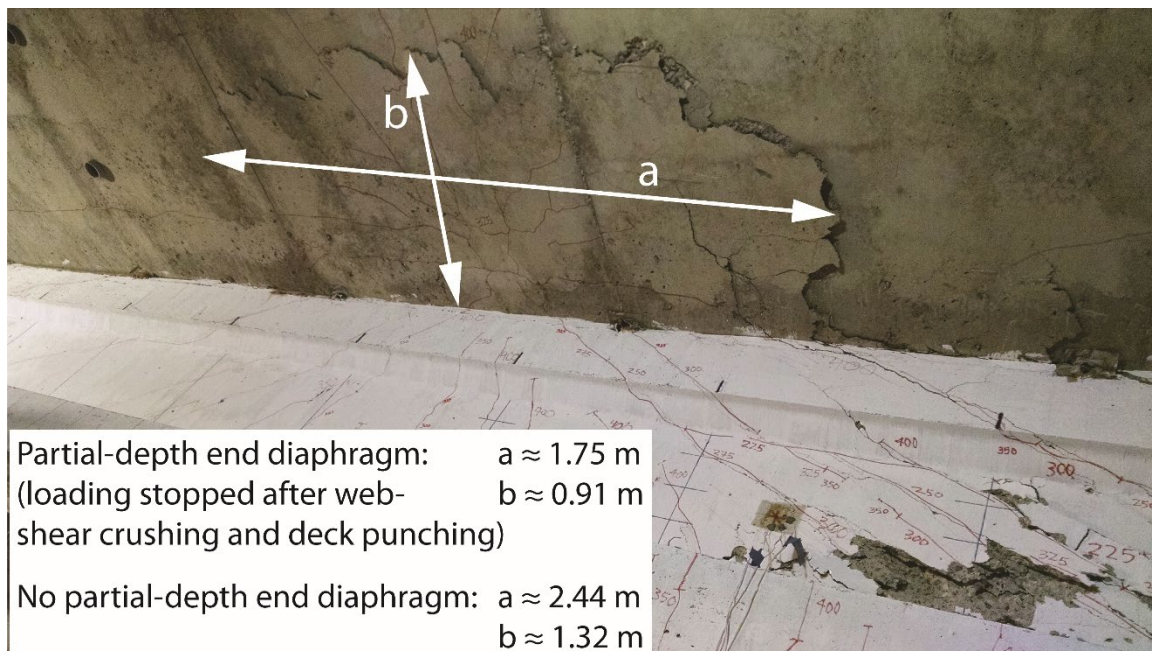


Fig. 11—Typical deck punching surface below the slab; 1 in. = 25.4 mm

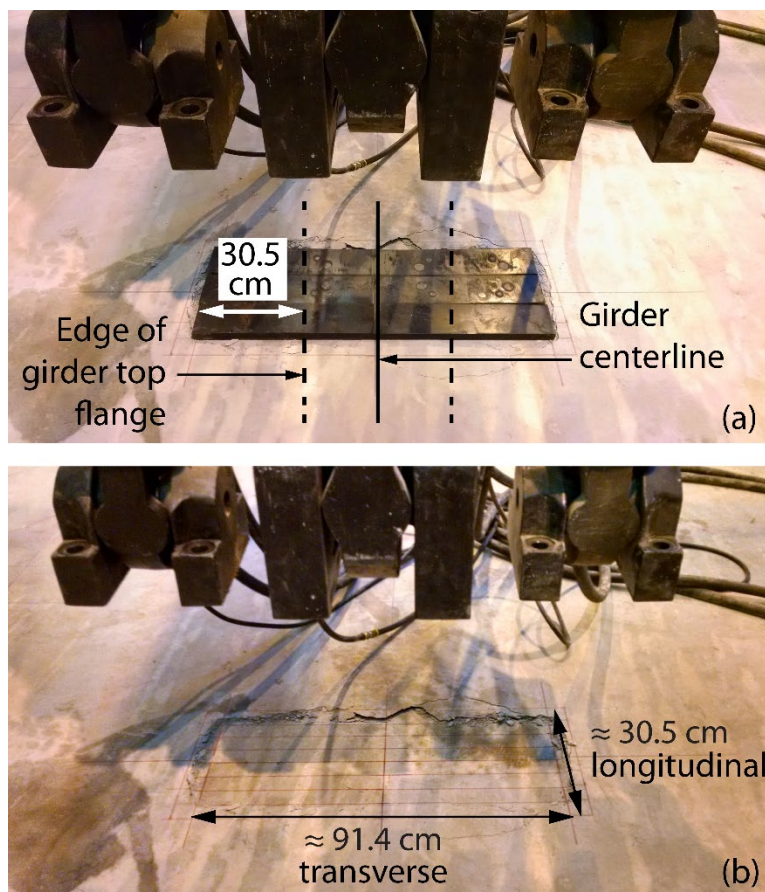


Fig. 12—Typical deck punching surface above the slab (a) with the neoprene bearing surface in place and (b) with the bearing surface removed; 1 in. = 25.4 mm

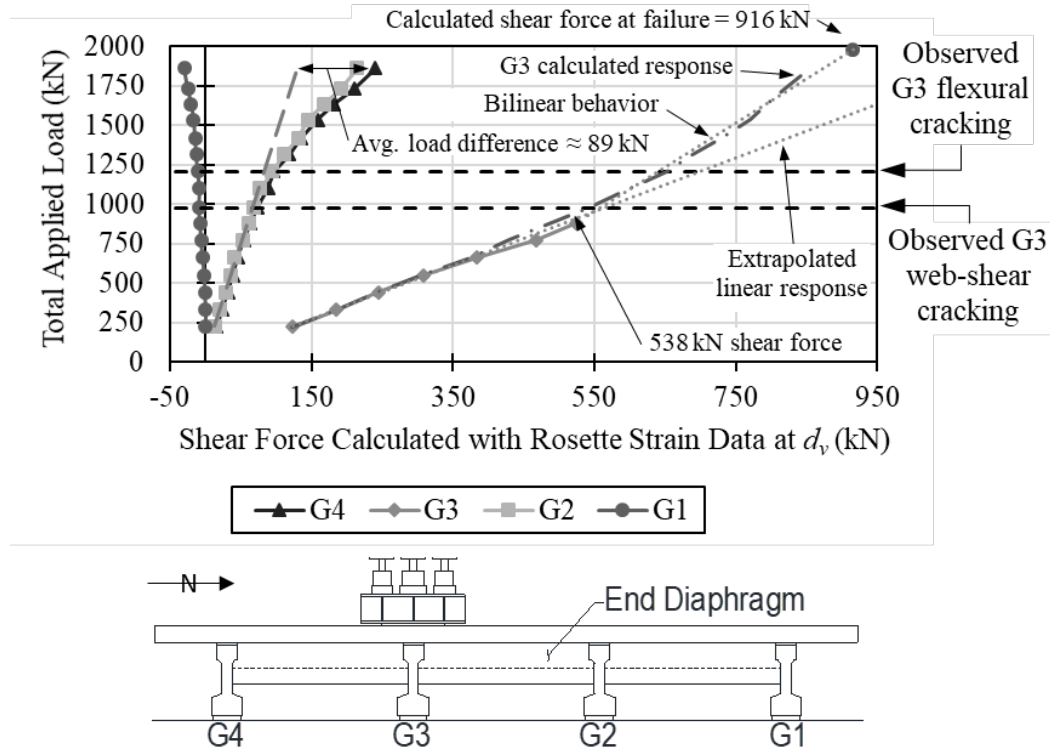


Fig. 13—Shear redistribution at d_v during testing with a partial-depth end diaphragm with load applied above girder G3; 1 kip = 4.45 kN

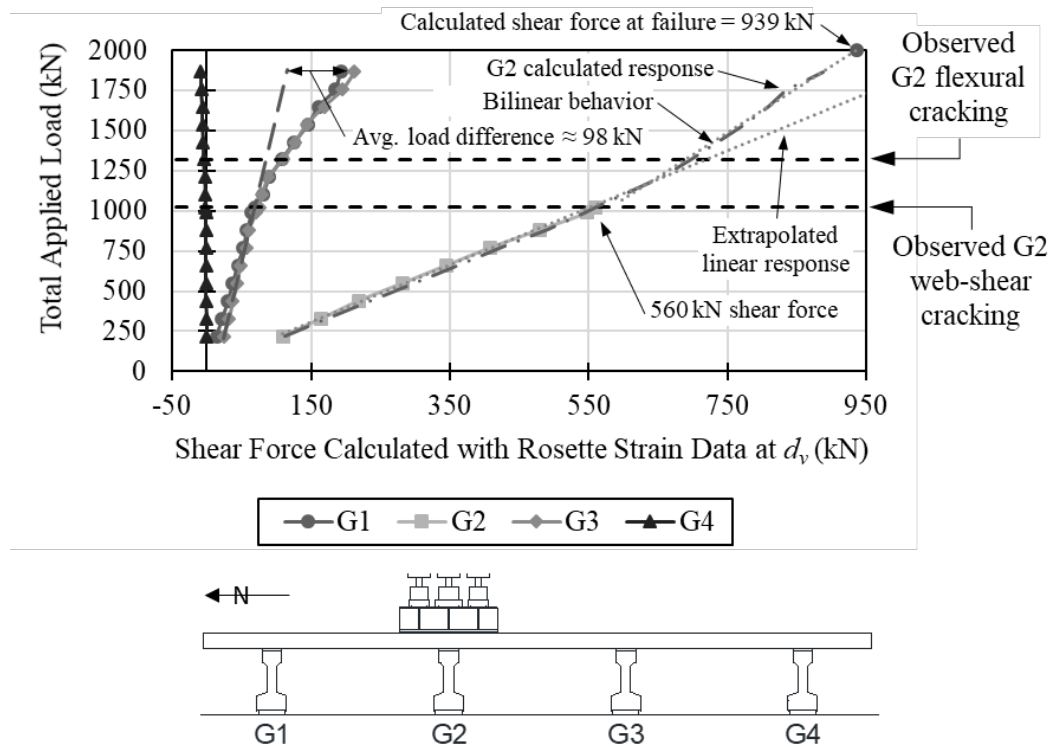


Fig. 14—Shear redistribution at d_v during testing without a partial-depth end diaphragm with load applied above girder G2; 1 kip = 4.45 kN

Ben Dymond is an Assistant Professor in the Department of Civil Engineering at the University of Minnesota Duluth. He is the secretary of ACI Committees 342 (Bridge Evaluation) and S802 (Teaching Methods and Materials), and he is a member of Committees 343 (Bridge Design), 423 (Prestressed), and 123 (Research). His research interests include design and analysis of concrete structural systems and their components, experimental investigations, and long-term structural monitoring.

Catherine French is a CSE Distinguished Professor in the Department of Civil, Environmental, and Geo- Engineering at the University of Minnesota Twin Cities. She is a member of several ACI technical committees including 318 (Building Code) and 352 (Joints). Her primary research areas involve investigation of the behavior of reinforced and prestressed concrete structural systems.

Carol Shield is a CSE Distinguished Professor in the Department of Civil, Environmental, and Geo- Engineering at the University of Minnesota Twin Cities. She is past chair of ACI committee 440 (FRP Reinforcement) and a member of 408 (Bond and Development of Steel Reinforcement) and 423 (Prestressed). Her primary research interests are in the use of composite materials in the infrastructure, prestressed concrete structures, and methods of seismic simulation.

Monitoring and Assessment of a Prestressed Concrete Segmental Box Girder Bridge

Marc Savard and Jean-François Laflamme

Synopsis: Several of the first prestressed concrete segmental bridges in North America were built in Quebec, Canada. The Rivière-aux-Mulets bridge was one of them. Built in the early 1960s, this bridge experienced several disorders due to inadequate design criteria enforced at that time. Despite a structural strengthening in the late 1980s, a bridge behavior follow-up has been required to ensure reliability. The structural health monitoring program implemented to track structural disorders, along with results from modal analysis and diagnostic load tests, is presented with a focus on the instrumentation and the data analysis. A three-dimensional finite element model was developed and calibrated using the frequencies and mode shapes detected under ambient traffic conditions. Data analyses showed that the expansion bearings were frozen, causing bending of the associated piers, which generated axial forces in the deck and decompression of concrete in the area surrounding active cracks. This process enables premature failure of prestressing tendons in the vicinity of these cracks, especially those located in the top flange, which is a corrosion-friendly environment. Development of cracks and associated prestress loss caused a reduction in the bridge load-carrying capacity. Analyses of health monitoring data led to acute assessment of the overall bridge structural performance.

Keywords: data analysis, finite element analysis, load tests, modal analysis, prestressed concrete bridge, structural damage, structural health monitoring

INTRODUCTION

Located about 70 km (43.5 mi) north of Montreal, the Rivière-aux-Mulets bridge is part of a northbound section of Highway 15 and crosses the Rivière-aux-Mulets river in the municipality of Sainte-Adèle, Québec. This highway is the main access to the northwestern part of the province. The ambient temperature in Sainte-Adèle varies greatly; the average maximum temperature reaches +29 °C (+84 °F) while the average minimum is as low as -35 °C (-31 °F). The concrete structure was built in 1964 and consists of a three-span continuous cast-in-place post-tensioned segmental box-girder of variable inertia. The central span is 80.77 m (265 ft) long, while the two end spans are 40.54 m (133 ft) long. A fourth simply-supported span 17.60 m (57.75 ft) in length completes the structure at its north end. At the time of construction, there were two twin bridges; both structures were the first in North America to be built according to a new European method (successive cast-in-place concrete segments) without scaffolding. A more detailed description of the bridge can be found in Ouellet and Gaumond (1990).

At both ends of the three continuous spans (abutment 1 and pier 4), bearings allow rotation along with horizontal movement. Vertically, these special bearings provide a bidirectional reaction (restraining uplift). At pier 2, simple supports allow horizontal movement of the box-girder above the pier. Fixed bearings are placed at the top of pier 3. Figure 1 shows an elevation view of the structure.

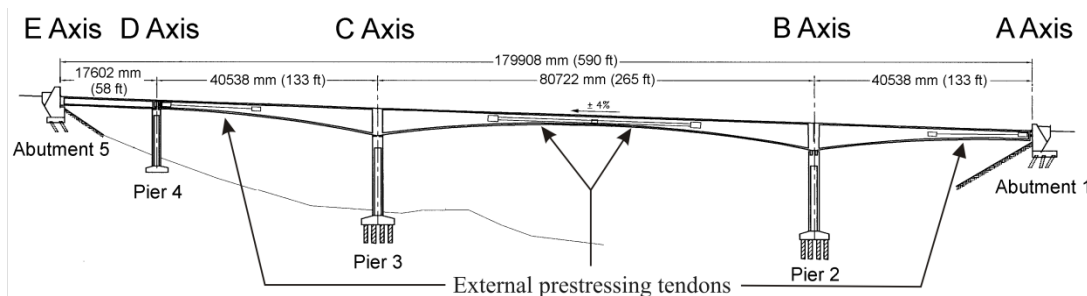


Fig. 1—Elevation view of the Rivière-aux-Mulets bridge

The depth of the box-girder section of the bridge follows a parabolic profile, from 5.0 m (16.4 ft) at the piers to 1.8 m (5.90 ft) at mid-span. Figure 2 shows a generic cross section of this single-cell girder.

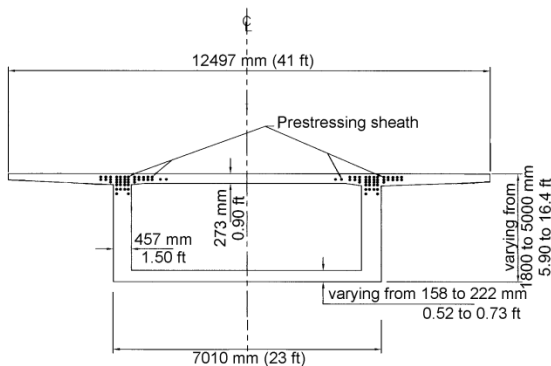


Fig. 2—Cross section of the Rivière-aux-Mulets bridge

OBSERVED BEHAVIOR AND REINFORCEMENT

Most prestressed concrete box-girder bridges built prior to 1985 have developed similar structural problems throughout their service life. Excessive vertical deflection at mid-span and cracking observed in some of these structures have led to intensive visual inspections (mainly documenting crack growth) and surveys aiming to accurately track the behavior of these structures over time. In Québec, results from electronic monitoring have already led to the premature replacement of the southbound twin structure that crosses the Rivière-aux-Mulets river in 2006.

Only a few years after opening to traffic, both Rivière-aux-Mulets bridges began to sag much more than anticipated. In view of today's recommendations, this comes as no surprise since the original design did not meet the recommended span-to-depth ratio: between 16 and 18 at the piers (16.2 for this bridge) and between 30 and 35 at the key-segment (44.8 for this bridge). Moreover, other structural disorders have been noticed over the years, such as vertical cracks in the webs of each end segment and diagonal cracks in webs near the key segment. Also, longitudinal cracks have been found inside the box-girder at the junction of the top flange and the webs.

As early as 1980, 16 years after commissioning, the bridge was monitored and a first phase of rehabilitation work took place in 1985 to repair the top flange and inject a few web cracks. During this repair work, the observed damaged condition of the bridge led to a thorough study of its structural condition. Calculations identified large tension zones in the lower flange of the end and central spans. In addition, plan review also showed that the end segments were not properly linked to the deck by prestressing tendons. Finally, review of initial calculations revealed that the design prestressing force was insufficient to withstand bending forces. The original design underestimated live load effects, prestress losses, temperature effects and effects of delayed concrete deformations (shrinkage and creep).

To overcome these deficiencies, major reinforcement work was carried out in the summer of 1988 without traffic interruption. On this occasion, external prestressing tendons were added to portions of the end spans and central span (see Figure 1), on both sides of both webs of the box section. To increase longitudinal shear strength, vertical prestressing tendons were introduced into concrete columns firmly connected to the inner face of both webs of each segment.

In subsequent years, surveys were conducted to monitor mid-span deflection. Unfortunately, given the significant influence of temperature variations on the deck profile, it appeared difficult to compare longitudinal profiles from one year to another and to detect a specific trend of the mid-span vertical deflection. Moreover, the barrier walls of the structure were completely rebuilt in 2002, and the pavement was redone many times over the years. These operations also had a significant impact on the deck profile.

In 2004, a structural health monitoring (SHM) program was initiated for the northbound bridge by the Ministère des Transports du Québec (MTQ) bridge department. This decision was based on the success of the health monitoring program that had been in operation for the southbound structure since 2002.

During replacement of the southbound structure, from February 2006 to November 2006, traffic had to be diverted to a third traffic lane added on the northbound bridge. Consequently, the northbound bridge experienced larger traffic loads. Consequences arising from this overloaded period and reconstruction work very close to the studied structure will be discussed later.

INSTRUMENTATION FOR BRIDGE STRUCTURAL HEALTH MONITORING

The objectives of the SHM program, initiated in 2004, were to follow the progression of mid-span vertical deflection and to collect information relevant to the management of the structure in order to ensure user safety and bridge sustainability. During the monitoring process, the instrumentation layout has undergone several modifications. These have been dictated by the necessity to validate and cross-check collected data, better understand bridge behavior and detect unexpected progressive failure modes. Only slowly progressive failure modes can be properly monitored.

In the fall of 2012, Ministry authorities mandated the bridge department to determine if the bridge replacement (planned at that time in 2013) could be delayed by at least five years. Careful analysis of the data gathered after eight years of SHM revealed that the bridge replacement could be delayed provided the addition of some sensors to better capture anticipated failure modes, if applicable. Data were recorded at a rate of one record every 10 minutes and the mean of six measurements gathered was stored at each hour. Figure 3 shows the 2012 bridge instrumentation layout. In the present paper, a measuring section is designated by its longitudinal coordinate. For instance, section 42, 79 and 118 have been localised in Figure 3.

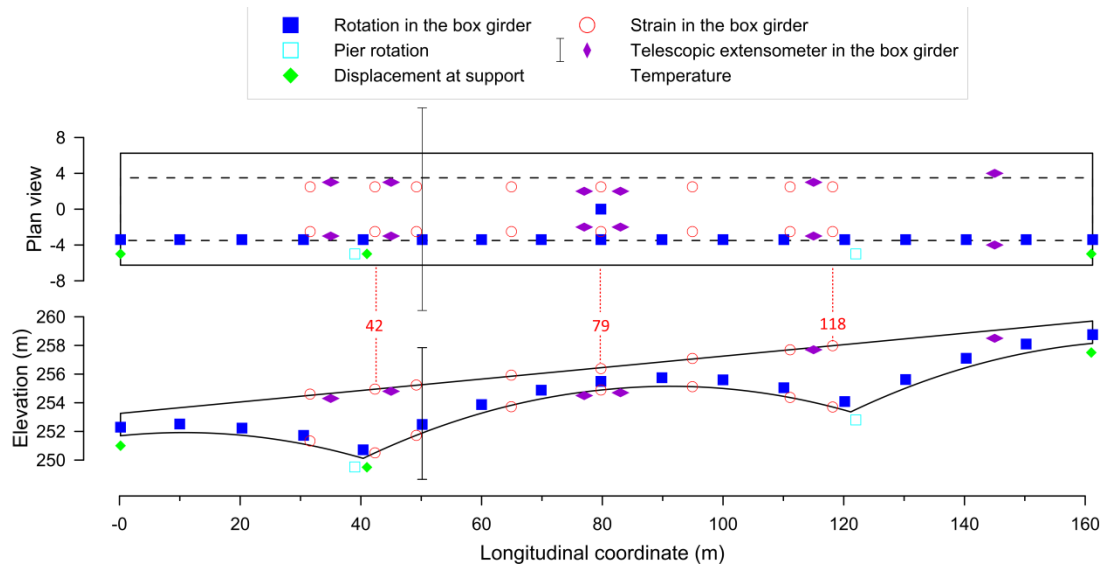


Fig. 3—2012 bridge instrumentation layout

Estimation of mid-span deflection using tiltmeters

In order to estimate vertical deflection, a total of 17 tiltmeters (sensors measuring rotation in the vertical plane) were used. This rotation represents the rate of change of the vertical deflection. A positive rotation was designated clockwise. These sensors were spaced 10 m (32.8 ft) apart along the three continuous spans. They were installed on the east side web rather than in the center of the lower flange because the fall of concrete debris, caused by significant delamination in the upper flange, could damage the sensors. A numerical integration procedure enables the estimation of the vertical deflection at each rotation measurement point.

Temperature

Since the vertical deflection is strongly influenced by the vertical thermal gradient, this parameter has to be recorded. Sixteen resistance temperature detectors (RTDs) were installed at a section located in the central span near the south pier to measure internal temperature variations. Temperature measurement sensors distributed over a section of the bridge allow the calculation of the average internal temperature as well as the thermal gradient (variation of internal temperature over the height of the section).

Strains

Thirty-two vibrating wire strain sensors located at the top and bottom of both webs allow the measurement of longitudinal strains at eight sections. Strain measurements are proportional to axial force, bending moments and distortion. Measurement sections are located mainly in the central span, above the piers, near the inflection points of the central span, and close to mid-span. Given the small size of these sensors (125 mm (5 in.) in length), measurements are sensitive to cracks developing nearby.

Rotation and displacement at abutments and top of piers

Potentiometers were installed to measure the relative horizontal movement between the box-girder and abutment 1, pier 2 and pier 4. Tiltmeters were also added at the top of piers 2, 3 and 4 (piers 2 and 4 are topped by expansion bearings). These sensors aimed to assess support mobility. In fact, horizontal restraint at these supports would cause additional internal forces in the box girder (axial force) and in the piers (mainly bending).

Box girder torsion

A tiltmeter was installed in the middle of the lower flange at the center of the bridge to measure torsional deformation about the longitudinal axis of the bridge (girder transverse rotation associated with torsion).

Telescopic extensometer

Twelve telescopic extensometers were installed at different locations on the bridge. These sensors measure total longitudinal displacement and axial strains over their length. In the case of the Rivière-aux-Mulets bridge, the sensors were 6.5 m (21 ft) long. For instance, if cracks perpendicular to sensors' axis appear between the anchor points of the sensors, crack breathings will be summed and the total movement will be captured by the sensor. The concrete thermal expansion is also measured by these sensors (no thermal compensation).

The extensometers were located at the top of the box-girder on each side of piers 2 and 3, as tension may develop in those areas prone to concrete cracks. Four extensometers were also installed outside at the bottom of the box-girder near the centre of the central span, since cracks may also appear in that area. Two other sensors were installed outside the girder, in the upper part of the web in the third span, 145 m (476 ft) from abutment 1, since damaged portions of concrete have been observed in that area.

FINITE ELEMENT MODEL

Concurrently with the SHM of the bridge, a numerical finite element model of the structure has been developed. Prestressing tendons were not explicitly included in the model. However, prestressing allows for the assumption of linear and elastic concrete behavior, both in tension and compression. This assumption implies that the largest compression stress in the concrete shall never exceed 45% of the concrete strength.

Moreover, prestressing forces are indirectly taken into account in the calibration process of the overall model. Calibration is done by adjusting the modulus of elasticity of different strategic structural components and the rigidity of the bearing devices. The objective of the calibration process is to have predicted vibration frequencies as close as possible to frequencies measured experimentally, both in bending and torsion. For example, specific properties of concrete have been established for certain portions of the webs, upper flanges and piers. Linear spring elements have been introduced to simulate actual support conditions.

The model has been developed in well-established commercial software environment with a total of 9982 linear plate elements for a total of 59754 degrees of freedom. The three piers have also been modeled (they are embedded at their base) since their flexibility significantly influences the overall structural behavior of the bridge, acting like elastic supports.

MODAL ANALYSIS

Modal analysis provides valuable information about the dynamic structural characteristics of a bridge. Recorded signals from accelerometers are analysed and vibration frequencies and corresponding mode shapes are identified. A mode shape is a property of a given structure that depends mainly on mass distribution and stiffness along the structure. Assuming mass remains constant over time, a change in the mode shapes or frequencies would indicate a change in rigidity resulting from cracks, concrete damage or modification of support conditions.

A total of six modal analyses have been carried out at the Rivière aux Mulets bridge in the falls of 2004, 2006, 2009, 2011, 2014 and 2016. Since temperature conditions affect mode shapes, it is important to compare results under similar thermal conditions in order to detect damage or structural deficiencies. As reported by Cremona (2005), measured frequencies may vary up to 5% for a 15 °C (27 °F) variation of the ambient temperature. The fall season has been chosen for *in-situ* modal analysis because thermal gradients are minimal during this period, thus limiting the effects of temperature on the behavior of the structure. Vibrations have been measured under ambient traffic conditions.

Twelve uniaxial accelerometers, individually connected to a data acquisition system, were distributed over the whole bridge in 2004. In 2011, the use of a wireless system with 12 uniaxial and six triaxial accelerometers simplified the data acquisition procedure. The location of the measuring points is the same.

LOAD TESTS

Load tests are intended to acquire data when the bridge is solely loaded by truck of known weight. The loads consisted of semi-trailer trucks of about 40 tons (88 kips) each. The number of trucks and their relative position on the deck

varied according to predefined load cases. Measured data are used to estimate force distribution in the bridge and to validate whether the bridge returns to its initial profile upon unloading. The end of the fall season has low thermal variations, which is suitable for such load tests, and measurements are less tainted by thermal effects. Therefore, the tests mainly represent the behavior of the bridge under known applied loads.

Three static load tests were conducted in the fall of 2004, 2013 and 2016. In 2004, the tests objectives were:

- to validate the proper functioning of all sensors,
- to measure bridge response under different predetermined load cases, and
- to provide measurements that can be used to calibrate the bridge numerical model.

The tests carried out in 2004 took place during the night of November 2nd. Three longitudinal paths were considered: 600 mm (24 in.) off west side parapet (path A), 600 mm (24 in.) off east side parapet (path C) and a path centered transversely on the deck (path B). A total of 18 stop lines were defined on the deck. Load cases included one or two semi-trailer trucks. Measurements from all of the sensors were recorded for each truck position.

The 2013 load tests took place during the night of October 28th, while the 2016 tests occurred during the night of October 24th. For the 2013 tests, up to three semi-trailer trucks were used for various load cases, while only two were used in 2016. Both tests had the same objectives, namely:

- to evaluate the change in structural behavior by comparing measurements under identical load conditions,
- to assess bridge behavior under maximum live load using three semi-trailer trucks, which represented approximately 90% of the bridge service load.

For these tests (2013 and 2016), the load cases included:

- two trucks along path A (same as 2004),
- two trucks along path C (same as 2004),
- two trucks centered on the bridge, one aligned on path A and the other following path C. This load case represented the most unfavorable configuration that was tested in 2004, and
- three trucks (2013) centered on the bridge, each occupying one of the three paths (A, B or C).

During all of these tests, data from all SHM sensors were recorded for each load case. A precision survey was also performed for specific truck positions.

DATA ANALYSIS

Data from modal analyses

Table 1 presents the mode shapes and associated vibration frequencies that were identified. In this table, comparisons can be made with the predicted mode shapes (eigenvalues extracted from the finite element model). The numerical model was calibrated with 2004 mode shape data.

For dominant mode shapes, very small frequency variations are found, except in 2006 where lower values are measured for all modes. The raise of average internal temperature may explain this result. However, even with similar internal temperature, significant discrepancies were obtained in 2006 and in 2016, suggesting that support conditions or structural damage (loss of rigidity) may have occurred over this 10 year period, as discussed later. Moreover, it is worth noting that over the years, high order modes show the largest frequency attenuations.

Table 1—Identified mode shapes and associated frequencies — Rivière-aux-Mulets bridge

Mode Shapes	Predicted Frequencies (FEM)	Experimental Frequencies					
		2004 Nov. 8-11.	2006 Nov. 14-16	2009 Dec. 3-4	2011 Nov. 21-22	2014 Nov. 25-26	2016 Nov. 16
Average temperature (°C)		-0.7	6.1	1.8	0.3	2.6	5.4
Mode shape	(Hz)	(Hz)	(Hz)	(Hz)	(Hz)	(Hz)	(Hz)
Bending #1	1.55	1.53	1.51	1.55	1.56	1.55	1.54
Bending-torsion #1	1.97	1.98	1.95	1.99	2.00	1.99	2.00
Bending #2	3.30	3.29	3.23	3.46	-	3.39	3.36
Bending #3	4.53	4.56	4.48	4.57	4.58	4.57	4.51
Sideways bending	5.64	-*	-*	-*	5.06	5.05	5.02
Bending #4	5.56	5.57	5.49	5.60	5.62	5.58	5.52
Torsion #1	6.79	-*	6.16	-*	6.10	6.15	6.02
Bending #5	8.04	8.08	7.97	8.16	8.17	8.14	8.02
Torsion #2	8.39	-*	8.59	-*	-*	-*	-*

* Corresponding modes were not significantly excited under ambient traffic conditions

Data from load tests

Longitudinal rotation comparison—Figure 4 compares rotations measured in 2004, 2013 and 2016 along the deck when two trucks were placed one behind the other, centered on the bridge, according to path C. The values are almost identical for all tests. The largest discrepancies occur in the central portion of the central span. The spacing between sensors was different in 2004 and therefore discrepancies appear to be more important. Furthermore, only half of the bridge was monitored in 2004.

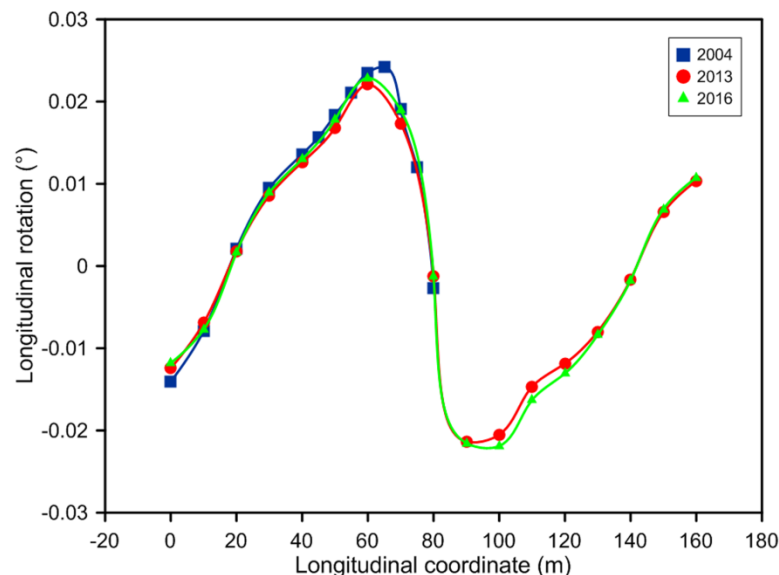


Fig. 4—Rotations along the deck when two trucks were placed one behind the other at the center of the central span (path C)

Longitudinal rotation comparison under different 2013 and 2016 load cases—Figure 5 compares rotations along the deck when two trucks were placed one behind the other, both following path A or C. The largest discrepancies occur in the central portion of the central span. End span data (0-40 m (0-131 ft) and 120-160 m (394-525 ft) from the south end of the bridge) show similar results. In the central span, rotations corresponding to path A (blue marks) are lower than the ones measured when two trucks follow path C (red marks). This can be explained by the fact that the east side web appears to be weaker than west side web. Moreover, rotations corresponding to path A have increased significantly from 2013 to 2016. Path C data have not changed significantly over the three year period.

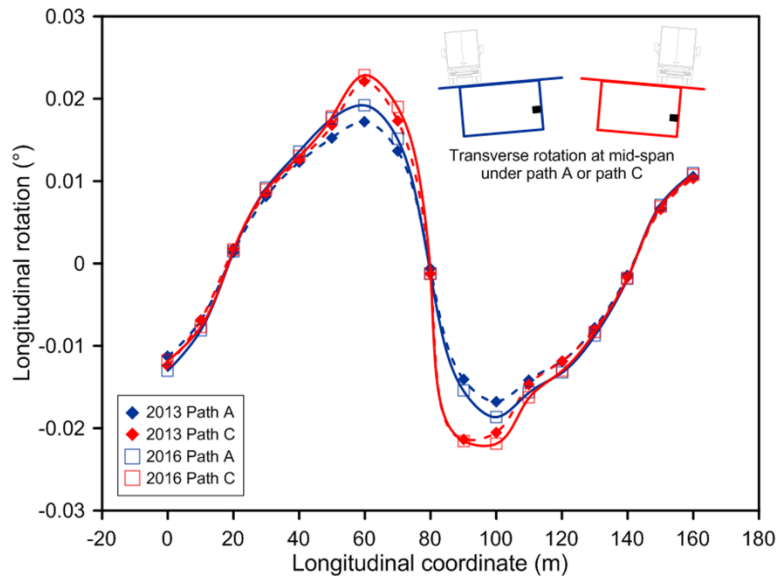


Fig. 5—Rotations along the deck when two trucks were placed one behind the other at the center of the central span (path A or C)

Calculated deflection for different load cases—On Figure 6, calculated deflection at different points along the deck can be compared with the predictions of the bridge numerical model. This deck profile was generated by two trucks at mid-span, one along path A and the other along path C. It can be seen that a roughly symmetrical behavior is predicted by the model. The calculated deflection from rotational data show some asymmetry, deflection being higher in the second half (northern half) of the central span. The maximum predicted deflection was slightly higher than results deduced from experimental values. Moreover, deflection increased in the mid-span portion from 2013 to 2016.

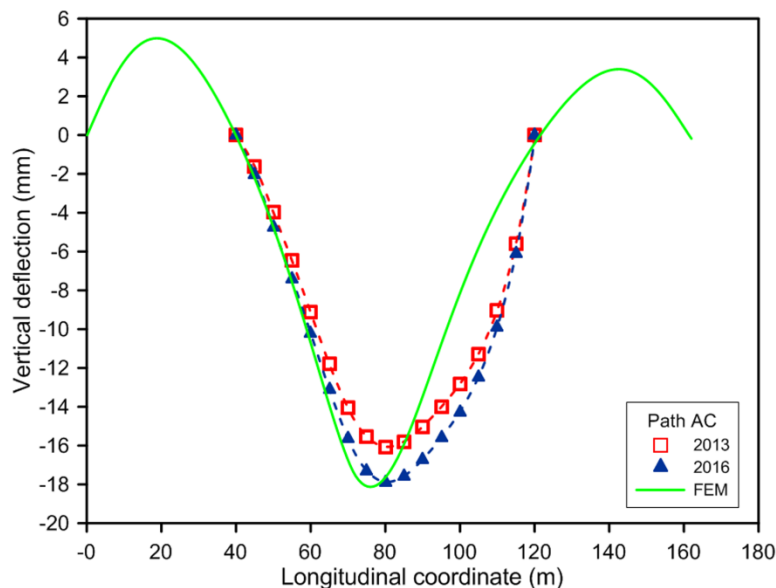


Fig. 6—Calculated deflection along the deck when two trucks were placed at the center of the central span (Path A and C)

Stress estimates for different load cases—Stress measurements from load tests carried out in 2004, 2013 and 2016 illustrate longitudinal and lateral force distribution in the structure. Behavior symmetry may also be assessed. A symmetric load case is created by loading paths A and C simultaneously, and the structure is expected to behave accordingly. Two trucks in a single lane (path A or path C) induce torsional forces and corresponding shear strains

that are not detectable because of strong torsional rigidity of this type of deck (closed section). Meanwhile, box distortion acts with bending moments to developed longitudinal strains. In figures 7, 8 and 9, measured stresses corresponding to different load cases are plotted (labeled “trucks on deck”) along with measurements recorded upon unloading (labeled “Final record”). Unloaded bridge records give indications whether the deck recovers its original state of stress after the loads have been removed.

Maximum stresses at section 42—As expected, for different load paths and with all trucks close to mid-span, bottom web fibers are in compression while top web fibers are in tension. As shown in Figure 7, top fiber stress ranges are slightly lower than 0.5 MPa (0.08 ksi), which is less than a quarter of the concrete tensile strength. In order to check whether the concrete tensile strength is exceeded (in other words, if live loads can cause cracking), dead load stresses would have to be estimated. In addition, particular attention would have to be paid to thermal stresses (mainly thermal gradients). These stresses can reach values similar to live load stresses, which would increase the total stress accordingly. Data in figure 7 also indicate that 2016 tests were systematically larger than those from previous tests, while unloaded deck records were larger in 2013 than 2016. Finally, comparison of the east and west side responses indicates significant discrepancies, regardless of the path considered or symmetry of the loading.

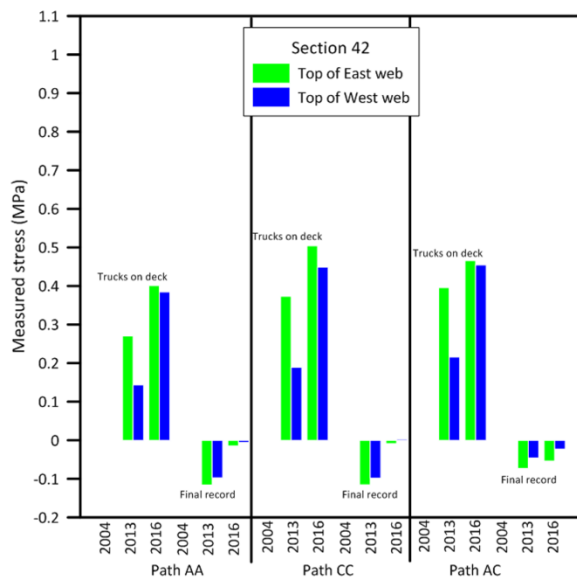


Fig. 7—Maximum stresses at section 42

Maximum stresses at section 79—As shown in Figure 8, a stress increment lower than about 1 MPa (0.15 ksi) at the bottom fiber of section 79 is generated by trucks placed at the center of the central span, in any pathways. Also, data from 2016 tests are always larger than those from previous tests. As previously noted, a symmetrical load case does not necessarily correspond with symmetrical behavior, except for load path AC in 2013, where symmetry is almost observed.

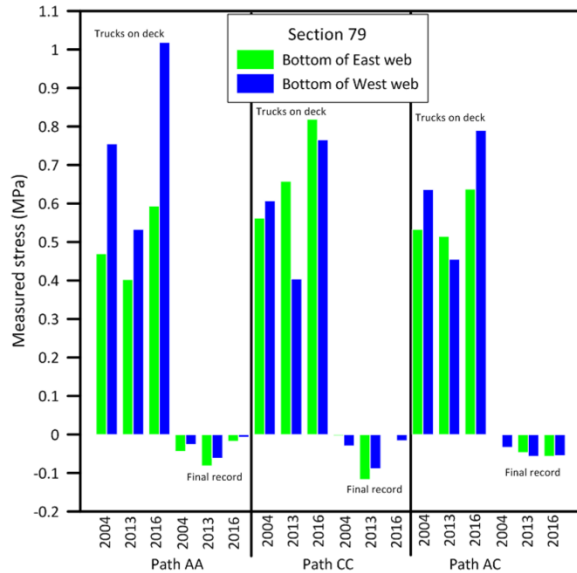


Fig. 8—Maximum stresses at section 79

Maximum stresses at section 118—Figure 9 shows a stress amplitude at section 118 that is significantly lower than the amplitude at section 42, a little less than 0.4 MPa (0.06 ksi) overall. Considering structure and load symmetry, this result suggests that the northern half of the bridge attracts less force than the southern half. Data gathered at other measurement sections show similar trends to those presented in Figures 7 to 9. Data recorded in the southern part of the deck (sections 42, 49 and 65) were systematically larger than those in the northern part (sections 95, 111 and 118). Structural damage may have occurred in the northern half portion of the bridge. This result is consistent with the asymmetric deck profile, deflection being higher in the second half (northern half) of the central span, as mentioned earlier.

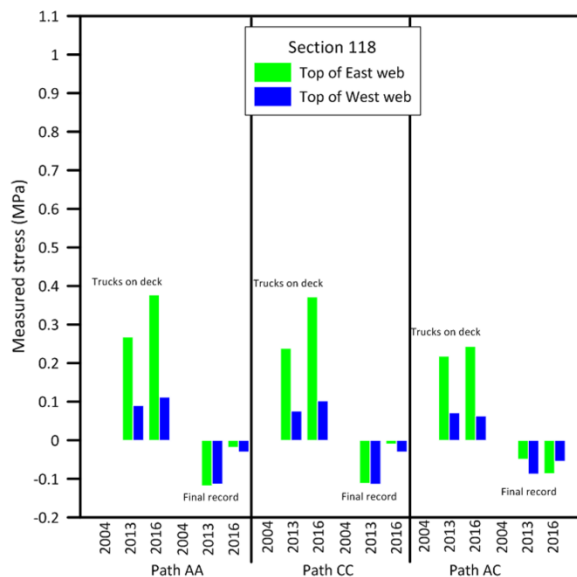


Fig. 9—Maximum stresses at section 118

Structural health monitoring data

The SHM of Rivière-aux-Mulets bridge started in 2004 with 61 measured parameters. In 2018, a total of 130 parameters were recorded several times a day. Considering all of the analyses, the following discussion will address only the main results, in order to present a global portrait of the bridge behavior findings.

Tracking longitudinal movements—Figure 10 presents relative movement of the box-girder with respect to supports and average internal girder temperature since 2006. The fixed bearing of the structure is located at the top of pier 3 (C axis). Therefore, no sensor has been installed at that location and no relative movement is expected. A bearing is considered fixed when no relative displacement occurs at that location. However, the bearing and the connected girder may move all together.

For the two expansion bearings at the ends of the box-girder (at A and D axes), relative displacements vary according to average internal temperature. However, at B axis, where a simple support is present, a very small relative movement has been recorded between the box-girder and the top of the pier, having a range (difference between the maximum and minimum values recorded since 2006) of about 4 mm (0.16 in.). This result is not in accordance with theoretical predictions and indicates a problem with the bearing device at the B axis.

At the D axis, although the selected sensor was adequate to grasp the anticipated span length variations, a movement larger than the sensor's operating range happened during the winter of 2014. This potentiometer was therefore replaced. The difference between anticipated and measured movements is undoubtedly the consequence of the malfunction of the B axis support. Meanwhile, based on the recorded data at the D axis support as shown in figure 10, it may be assumed that the deck longitudinal movement is not restrained at this support.

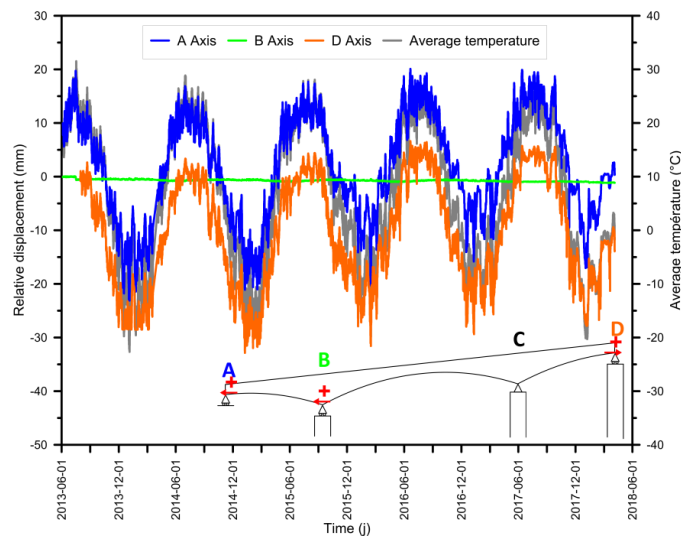


Fig. 10—Relative displacements at supports since June 2013

A linear regression evaluates the degree of proportionality between two parameters. The correlation coefficient (R^2) characterizes the linear regression. When the coefficient approaches unity, a linear relationship can be established between the two parameters. When the correlation coefficient approaches zero, the parameters are said to be uncorrelated (or completely independent). The correlation coefficient ($R^2 = 0.95$) indicated that the mean temperature variation is the factor that mostly influences the amplitude of the longitudinal displacements measured at the end supports of the main box-girder (A and D axes).

Figure 11 shows the rotation measured at the top of the piers as a function of time. An adequate bearing behavior should allow the main girder to freely move longitudinally over its supports, with the piers keeping their verticality. Instead, the data indicated significant rotation at the top of the piers in phase with average temperature variation. In the summer, the top of the piers at B and C axes move away from each other, in accordance with deck dilatation and restraints in the bearings. A reverse behavior is observed in the winter. The rotation range at the top of the piers is similar for both piers, estimated at a little more than 0.12° .

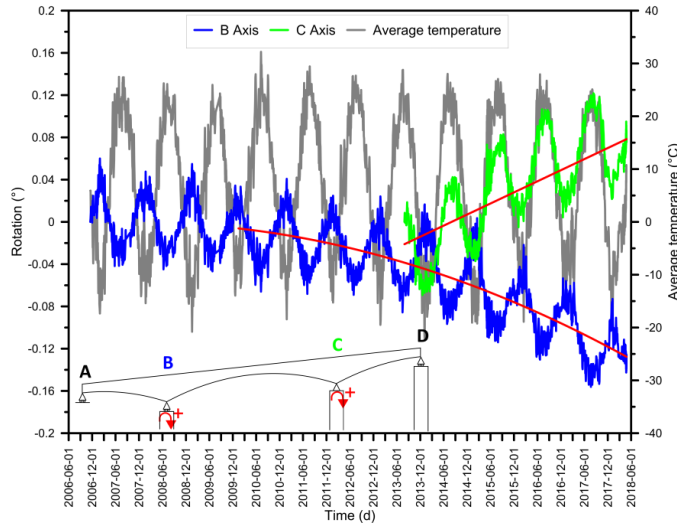


Fig. 11—Top of pier rotation as a function of time

Pier bending comes with an axial force applied through the frozen bearings into the concrete box-girder. In winter, as internal temperature lowers, the bridge deck shortens and a tension force is developed axially in the deck. This mechanism is not without consequences, as will be discussed later. At the top of C axis pier, measured rotation indicated that this point is not immobile, and the displacement amplitude at D axis is therefore amplified, explaining why this sensor had exceeded its operating range.

Figure 11 also shows the progressive bending of pier 2 (B axis) towards the south abutment. An average rotation of $6 \times 10^{-3} \text{ }^\circ$ per year was detected over the past eight years. Meanwhile, pier 3 (C axis) bends gradually toward the north abutment. These pier movements are in accordance with progressive sag at mid-span.

One can estimate the relative displacement δ_D (positive to the right) at D axis with the following equation:

$$\delta_D = \frac{2}{3} h_{PC} \theta_{PC} + \alpha (L - L_{TS}) \Delta T \quad (1)$$

where h_{PC} is the free height (m) of C axis pier, θ_{PC} is the rotation (rad) measured at the top of C axis pier (see Figure 11 for elected positive direction), α is the coefficient of thermal expansion of reinforced concrete ($1.1 \times 10^{-5} / \text{ }^\circ\text{C}$), L is the distance (m) between D and C axes (fixed support), L_{TS} is the length (m) of the simple span supported at D axis (18.25 m (59 ft)) and ΔT is the studied temperature variation ($^\circ\text{C}$). In equation (1), it is assumed that the length variation of the simply supported span induces a horizontal movement at the top of D axis pier, since the expansion bearing of this span is similar to that at B axis, which is blocked (no relative displacement measured). Also, C axis pier base is assumed to be immobile.

Similarly, for the theoretical estimate of the relative displacement δ_A at A axis support, the following expression can be used (positive relative displacement to the left):

$$\delta_A = \alpha L \Delta T - \frac{2}{3} h_{PB} \theta_{PB} \quad (2)$$

where h_{PB} is the free height (m) of B axis pier, θ_{PB} is the rotation (rad) measured at the top of B axis pier (see Figure 11 for elected positive direction) and L is the distance (m) separating A and B axes.

Table 2 compares measured relative displacements (recorded in 2014 and calculated at temperature extrema) with corresponding theoretical estimates calculated using equations (1) and (2). Results confirm the inadequate behavior of the two expansion bearing (B axis and expansion bearing of the simply supported span), because they do not allow the free movement of the deck relative to the top of the concerned piers. If this restraint is taken into account in the numerical model of the bridge, Figure 12(a) shows an overview of the bridge deformed shape when the mean internal

temperature is low. Figure 12(b) illustrates the anticipated deformed shape when the average internal temperature is high.

The theoretical approach leading to these results (Table 2) does not take into account the variability of the internal temperature along the bridge deck. In addition, it assumes a bending behavior of piers in order to estimate the horizontal displacement at the pier top. Moreover, it does not take into account the influence of the box-girder bending on the expected horizontal movements at the supports.

Table 2—Comparison of theoretical estimates of maximum longitudinal displacements with the corresponding measured values

	South Abutment (A Axis)	North Abutment (D Axis)
Maximum measured range [mm]	40.0	31.9
Maximum calculated range (Equations [1] and [2]) [mm]	47.2	35.9

Note: $\alpha = 1.1 \times 10^{-5} / ^\circ\text{C}$ and $\Delta T = 51.5 ^\circ\text{C}$

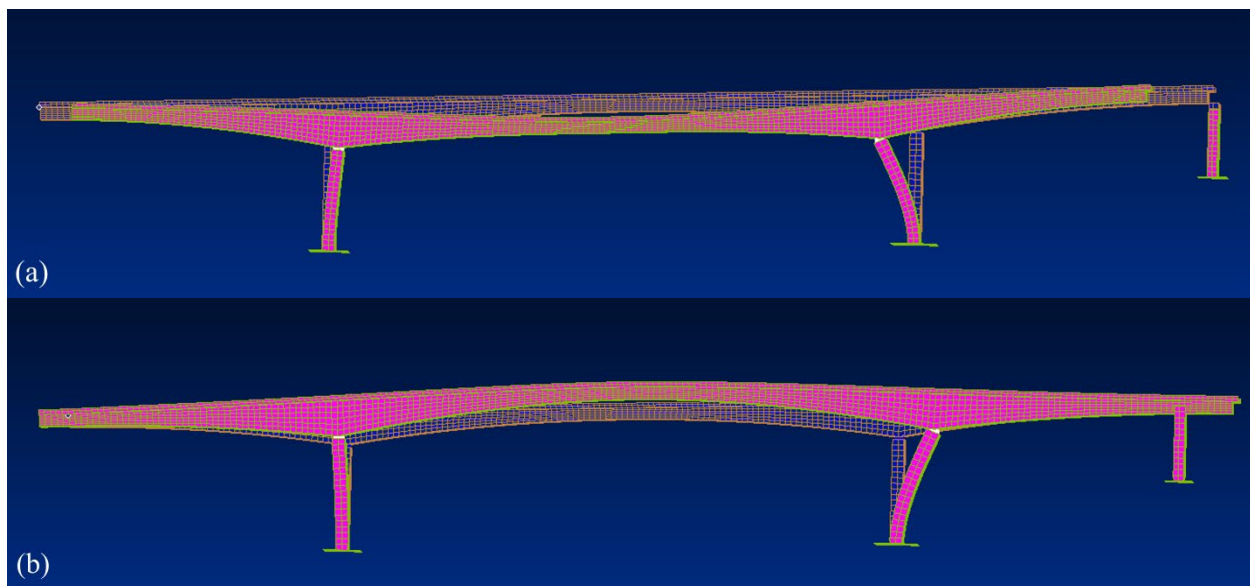


Fig. 12—Predicted bridge deformed shapes with (a) low or (b) high internal temperature

Tracking breathing of groups of cracks with telescopic extensometers—As mentioned earlier, 12 telescopic extensometers have been installed at six different measuring sections located 35, 45, 77, 83, 115 and 145 m from the south end of the structure. A deck subjected to internal temperature fluctuations is expected to have theoretical movement of approximately 3.7 mm (0.15 in.), which is measured by these sensors.

Figure 13 shows five years of data recorded from extensometers 35, 45 and 115 located near the internal supports. It can be seen that the sensor data from the west side of the bridge vary in a similar way, regardless of their longitudinal position along the bridge. Since these sensors are centered over the supports, and given the symmetry of the structure, this result aligns with expectations.

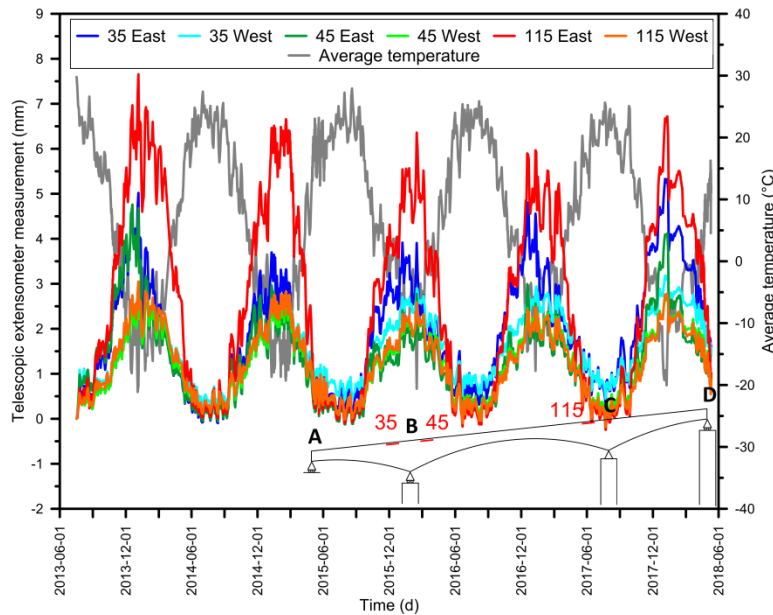


Fig. 13—Recorded telescopic extensometer measurements at sections 35, 45 and 115

Sensors located on the east side (right under the top flange) monitor greater crack breathing in cold weather as compared to the summer, particularly those at section 115. In the summer, as the deck lengthens (see Figure 12b), the pier bending generates compressive and bending forces in the deck, causing cracks to close, thus creating additional compression in the deck. Once the cracks are closed, additional compression in concrete induces minute variation in extensometers response. In the winter, the deck shortens (see Figure 12a), compression in concrete decreases up to a point where the cracks re-open. The opening (breathing) of these cracks is captured by the extensometers and the measurements are amplified accordingly, as shown in Figure 13. This behavior also appears on the east side of sections 35 and 45, to a lesser extent. It should be emphasized that “freezing” of bearings at B and C axes is mandatory for such crack breathing.

When cracks widen and concrete decompresses locally, the mechanical properties of the box-girder at surrounding cracks are locally modified and the axial rigidity (EA) is significantly reduced. This may invalidate the estimate of local stress from strain measurements in the vicinity of cracks, as addressed later in this paper. Results from telescopic extensometers at sections 77, 83 and 145 are similar. Measured ranges align with the theoretical prediction (3.7 mm (0.15 in.)).

Tracking of mid-span girder transverse rotation—A tiltmeter was installed at the bridge center in order to capture rotation associated with torsional deformation of the box-girder (rotation about deck longitudinal axis or girder transverse rotation). Figure 14 shows this sensor’s recordings. A positive rotation is measured when the east side web moves downwards with respect to the west side web that may rise. In other words, a clockwise rotation around deck longitudinal axis pointing north is assumed positive.

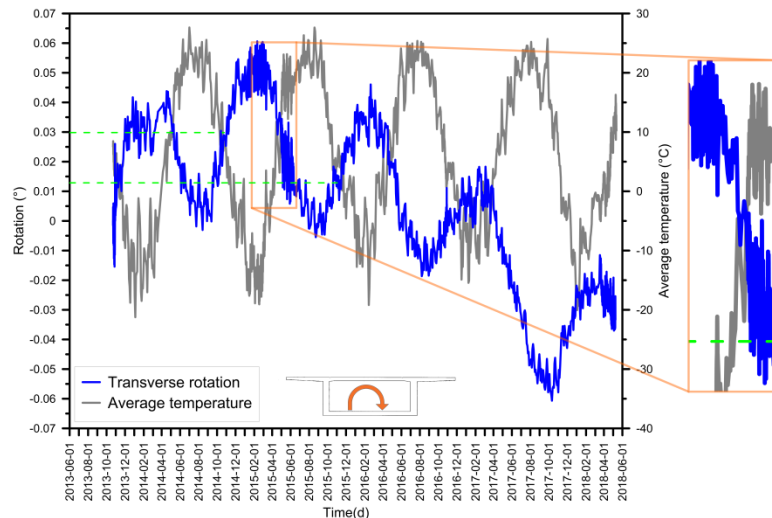


Fig. 14—Girder transverse rotation at mid-span

Data in Figure 14 indicate that girder transverse rotations are correlated with internal temperature variations. Also, an “event” may be noted around the end of April 2015 and is characterized by a rapid decrease (from 0.045° to 0.035°) of the girder transverse rotation (orange enlarged rectangle on the figure). This coincides with a sudden rise in temperature (from 5 to 20°C (41 to 68°F)), which is associated with a rapid and significant increase of thermal loads in the bridge. Comparison of the November 2014 data with the ones obtained one year later (green dashed lines on the figure) reveals a permanent rotation of about -0.01° (counterclockwise). With permanent loads unchanged, this result suggests that structural damage took place, and the so-called event that occurred at the end of April 2015 may be the starting point of this apparent damaging process, as will be discussed later.

Tracking of mid-span deflection—Figure 15 shows mid-span deflection as a function of time from September 1, 2004 to May 2018. Deflection is estimated from tiltmeters fixed on the east side web of the box-girder. Results shown in Figure 15 came from two sensor layouts, and two configurations of the data acquisition system have been taken into account. The tiltmeter located at mid-span was defective between December 21, 2014 and May 1, 2015, which affected the accuracy of the deflection estimates.

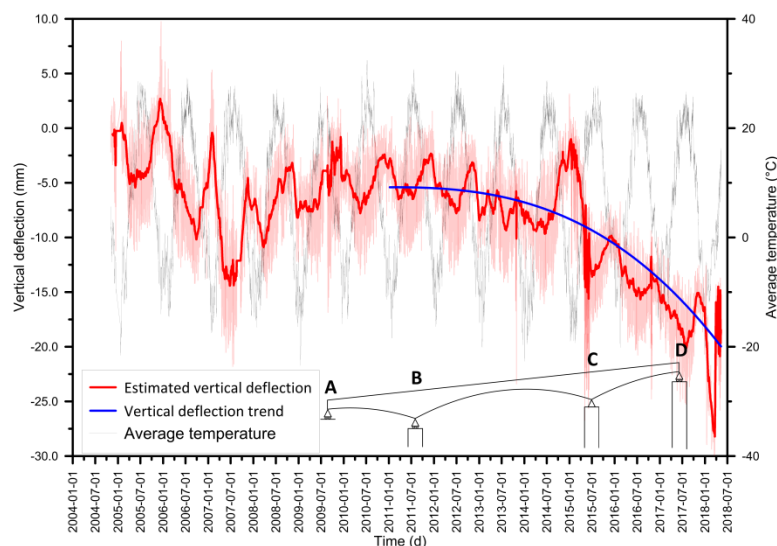


Fig. 15—Mid-span deflection from September 1, 2004 to May 2018

A moving average calculation highlights the deflection trend over time. Figure 15 shows that the mid-span deflection was fairly stable from June 2004 until the beginning of 2015, with sag increasing progressively by about 5 mm (0.2

in.) over this period. The largest sag was observed during the replacement of the southbound bridge in the summer of 2006.

Figure 15 also shows that a permanent deflection has been building up since the end of 2012. The permanent nature of this sag is confirmed with the comparison of the elevation of the deck measured at the same date, one year apart. Between November 1, 2014 and November 1, 2015, the deck center sagged about 10 mm (0.4 in.), and the sag increased by about 5 mm (0.2 in.) every year thereafter. The “event” detected at the end of April 2015 may have contributed to some extent to this significant progression of deflection. Increase in deflection had not been observed since the replacement of the southbound bridge in 2006. These observations occur simultaneously with a rapid variation in girder transverse rotation, as discussed previously.

Tracking of concrete longitudinal stresses—A total of 32 strain gauges were installed at the bottom and top fibers of different measurement sections along the bridge. Assuming a realistic value of the concrete modulus of elasticity, longitudinal stresses can be estimated. It should be emphasized that measured strains (and resulting calculated stresses) do not represent the total strain (including permanent state), but rather the strain increase since the sensors were installed. Furthermore, the average elastic modulus of prestressed reinforced concrete may depend on local conditions such as cracks, delamination, corrosion, and other disorders.

Stresses at section 42—Figure 16 shows stresses at section 42, located in the central span near pier 2 (B axis). In November 2006, negative stresses (compression) appeared in the upper parts of the webs, mainly on the west side. This result may be explained by a progressive settlement under piers 2 and 3 (B and C axes) caused by the dismantlement/reconstruction of the southbound twin structure (February to November 2006). The latter is located a few meters on the west side of the studied bridge, where a large sand mattress had to be erected in April 2006 to facilitate its dismantlement. Also, the erection of new piers required driving piles close to the existing bridge. Progressive soil reworking close to the pier foundation may have contributed to assumed settlement. The presence of a tiltmeter tracking the pier in-plane rotation (in the vertical east-west plan) could have corroborated this assumption.

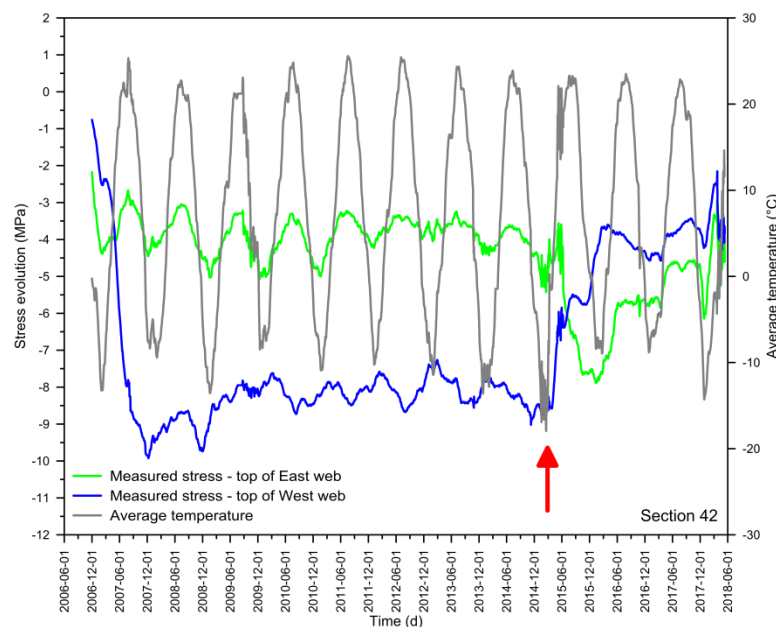


Fig. 16—Stress variation on both webs of section 42

Subsequently, seasonal stress fluctuations of approximately 1.5 MPa (0.22 ksi) were observed but overall, mean stresses remained stable. Extreme stress estimates on the east side were in phase with temperature extremes until the end of April 2015 (red arrow in Figure 16), while there was a shift with the west side stress estimates. Also, stress estimates are opposite, meaning that a stress increase in the upper part of the west side web comes with a decrease in the upper part of the east side web. This result is in agreement with girder transverse rotation shown in Figure 14 (warping stress).

After April 2015, an increase in stresses at the top of west side web occurred simultaneously with a decrease in stresses at the top of the east side web. The amplitude of these stress variations are similar. Such variations may be the result of prestress loss and/or concrete cracking at the top of the east side web or in the east side portion of the top flange above B axis pier. The east side web would then be unloaded at the expense of the west side web, which would have to absorb a larger negative bending moment. Loss of rigidity above the pier induces an increase of the mid-span bending moment, as will be shown later. Telescopic extensometers and girder transverse rotations also suggest that rigidity loss may have occurred in the top east part of the web or in the top flange, resulting in a counterclockwise rotation at mid-span.

Stresses at section 79—Figure 17 shows stress variation at the lower fiber of the west and east side webs, 79 m (259 ft) from the south end of the bridge (less than 1 m (3.3 ft) from the center of the central span). Because of a sensor failure, recorded data in the west side web only became available in June 2007. Recorded data vary according to and are in phase with internal temperature. The stress range is close to 1 MPa (0.15 ksi) for the east side web and close to 1.5 MPa (0.22 ksi) for the west side web. Unlike at section 42, extremes stress estimates are in phase with extreme internal temperature.

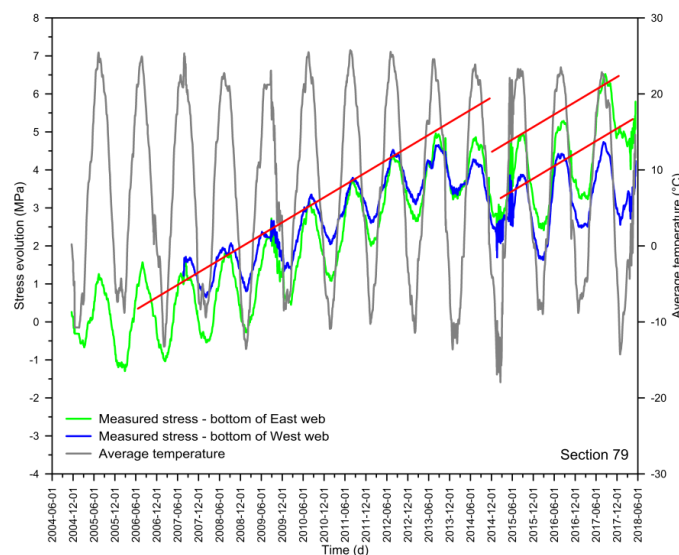


Fig. 17—Stress variation at bottom fiber of section 79

In Figure 17, it can also be seen that stresses measured at the lower fiber have increased steadily by about 4.5 MPa (0.65 ksi) over seven years (red straight lines in the figure). This stress variation is observed in both webs. Such a progressive stress variation at a given section generally indicates a force redistribution initiated by a damaging process occurring somewhere in the structure. Forces are generally redistributed towards more rigid areas. Figure 17 suggests an increase in bending moment at section 79 as a consequence of prestress losses and/or concrete cracking over intermediate piers.

At the lower fiber of section 79, the stress intensity seems to reach a limiting value since it started to decrease during the summer of 2013, before it starts to increase again later on. From then on, internal forces (bending moments) seem to be successively relocated, from the central section to the piers and then back again to the central section. However, with the progression of the damage zones, serviceability criteria are eventually violated to some extent before collapse would become imminent.

Stresses at section 118—Figure 18 illustrates the behavior at the top of the east and west side webs at section 118. From the beginning in 2006, a reduction of the stress intensity at the top of the web is observed on both sides, probably initiated by ground settlement (as discussed previously). This phenomenon is observed until December 2007. Subsequently, seasonal cycling stress ranges of 2.5 MPa (0.36 ksi) were observed on both sides, but overall stresses remained relatively stable until the end of April 2015. From that moment, longitudinal stress estimates at the top of the east side web seemed to increase progressively by 5 MPa (0.73 ksi) in about three years, while no clear trend is seen in the west side data. This result indicates a significant increase in forces associated with gravity loads (negative

bending moment in the box-girder over C axis pier). Furthermore, this stress increase occurs simultaneously with the stress increase in the opposite end span and has the same amplitude (top of the west side web at section 42).

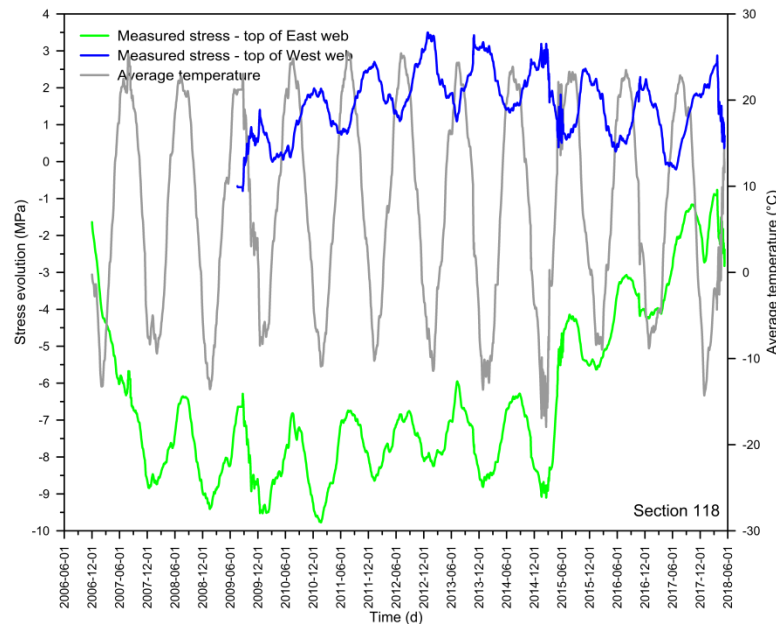


Fig. 18—Stress variation at the top fiber of section 118

CONCLUSIONS

The Rivière-aux-Mulets bridge has been monitored for many years until its dismantlement in 2018. Three load tests have been conducted with trucks of known axle weight, and recorded data have been used to calibrate a representative finite element model. The model, in conjunction with long-term monitoring data, allowed for the identification of the following structural disorders:

- Most significant reductions in frequencies of vibration have been observed for higher order modes (Sideway bending, Bending #4 and up). This observation is typical of bridges with an active damaging process. From load tests results, at section 42 (over pier 2) and at section 79 (close to mid-span), the estimated stresses recorded in 2016 were all greater than the ones recorded in 2013. Meanwhile, the 2016 results from section 118 (over pier 3) were all lower than the 2013 corresponding stresses.
- In addition to a slow and progressive permanent mid-span deflection, there was an unexpected and spontaneous sag increase at the end of April 2015. Simultaneously, permanent girder transverse rotation was detected. Along with longitudinal stresses estimated at different sections along the deck, these observations unveil structural damage to the southern half of the east side web of the central span. Structural damage may take different forms, such as concrete cracking (accompanied by tension stress relaxation) or prestress loss (breaking of a tendon, loss of anchorage, corrosion), the latter being accompanied by compression stress relaxation.
- Free relative longitudinal movement is expected between the box-girder and the top of B axis pier support. Sensors showed that no such movement is allowed (bearings are “frozen”) and that the pier bends according to internal temperature changes. C axis pier is topped with fixed bearings. Consequently, an axial force developed into the deck between B and C axes – compression in summer when the deck lengthens and tension in winter when the deck shortens. Incidentally, a permanent and progressive tilting has been detected in these piers.
- Telescopic extensometers have been used to track breathing of groups of cracks. Recorded data indicated that concrete in the vicinity of some cracks is decompressed when the temperature was low. In the winter, the deck shortens and the compression in the concrete reduces up to a point where cracks widen. Crack openings were measured by extensometers and the detected amplitude is larger than expected. Frozen bearings are mandatory for this behavior to occur, and residual prestressing forces are not sufficient to prevent crack breathing. As a consequence, prestress loss causes a reduction of the load carrying capacity of the

structure. Moreover, breathing cracks and concrete decompression causes larger stress cycles in prestressed tendons. This phenomenon promotes premature failure of tendons, especially those located in the top flange, which is a corrosion-friendly environment (de-icing salts are spread on the driving surface). This diagnosis was achieved only with the help of long-term monitoring data and without information on the total applied stress.

In summary, the analysis of the SHM data over the past years indicated that the bridge was subjected to a variety of structural damaging processes. A damaged structure loses its ability to perform properly. Since it was not possible to anticipate the rate of progression of structural damage, the recommendation to replace this structure in the short-term had to be maintained. Nevertheless, the structural health monitoring program implemented for this bridge allowed the requested five-year delay for bridge replacement to be granted.

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REFERENCES

- Cremona, C. 2005. Qu'est-ce qu'une évaluation dynamique ? Principes et méthodes. *Revue européenne de génie civil*, 9 (1-2), 11-42.
- Ouellet, C. Gaumont, Y. 1990. Strengthening of Two Prestressed Segmental Box-Girder Bridges. *Developments in Short and Medium Span Bridges '90*, Third International Conference on Short and Medium Span Bridges, Vol. 2, Toronto, Ontario.

Marc Savard earned his doctorate in civil engineering from Laval University in 1993. Subsequently, he joined the bridge department at Ministère des Transports du Québec where he is the principal investigator for bridge data interpretation from load tests and structural health monitoring programs. His expertise includes finite element modeling of bridges and retrofit of fatigue cracks in steel bridges. He also teaches graduate and undergraduate courses at Université du Québec à Chicoutimi.

Jean-François Laflamme is a graduate engineer from Laval University where he obtained his bachelor's degree and a master's degree in civil engineering. He worked at Laval University as a researcher before joining the mobile laboratory team at the Ministère des Transports du Québec. He has participated in numerous load testing projects, modal analysis campaigns, and structural health monitoring of several major structures in Québec. He is now responsible for instrumentation and finite element modeling.

Field Testing to Failure of a Skewed Solid Concrete Slab Bridge

Fabien Lagier, Bruno Massicotte, David Conciatori, Jean-François Laflamme

Synopsis: In 2006 in Quebec, a skewed cantilever solid concrete slab bridge without shear reinforcement collapsed due to a shear failure, which highlighted the need to improve the assessment of this type of structure. A large experimental program was carried out to test three decommissioned solid slab bridges to failure. In parallel, an extensive nonlinear finite element analysis study was performed with the aim of better understanding the failure mechanisms, the degree of load redistribution, and to gain insight into the ultimate shear capacity of these structures. A beam shear failure mode was expected for the first two bridge tests, but a flexural failure mode was observed. This paper focusses mainly on the last field test of a simply supported solid slab bridge having a 40 degree skew. The load position and the loading protocol were established with the objective of causing a shear failure at the obtuse corner of the slab where high shear forces develop. The main test motivation was to illustrate that simply supported solid slab bridges would normally not be prone to shear failure due to an intrinsic redundancy. The paper presents experimental techniques that could help bridge owners in assessing the performance of their bridges. The test results also provide valuable information for calibrating nonlinear element models that can be used for assessing the carrying capacity of existing concrete bridges.

Although the actual bridge conditions were worse than anticipated, a global shear failure mode occurred near the obtuse corner at a maximum load of 1400 kN, which significantly exceeded the factored shear force due to the maximum traffic load. The failure was followed by a gradual load redistribution toward undamaged portions of the slab. This field test confirmed the assumption of non-fragility for this type of bridge, where support conditions enable development of an intrinsic redundancy. Despite these observations, nonlinear analyses carried out in parallel to the testing program indicated that this beneficial effect diminishes with an increase of slab thickness.

Keywords: solid slab bridges, field testing, load rating, shear failure, nonlinear finite element analysis

INTRODUCTION

Concorde Bridge Collapse

Many existing reinforced concrete bridges require repair, strengthening, or even replacement before reaching their expected service life. The action plan required for each structure is largely related to the performance and relevance of the bridge inspection and assessment programs. On September 30, 2006, the Concorde Bridge, located in Laval near Montreal, collapsed due to the sudden shear failure of a 1.2 m (4 ft) deep solid reinforced concrete cantilever slab supporting a precast concrete drop-in span. This event highlighted the need to improve the understanding of solid slab bridge behavior and enable more robust assessments of these structures.

Typically, solid reinforced concrete slab bridges have span lengths less than 20 m. In Quebec, they represent 23% of the Province bridge network, excluding culverts. These structures are mainly used for secondary road overpass, or for crossing small obstacles such as streams. The average age of this type of bridge is 58 years. Their design is relatively simple and they required conventional construction methods. This type of bridge is perceived as reliable and not prone to sudden brittle failure.

North-American bridge design codes [1, 2] do not require the use of minimum shear reinforcement in solid slabs. However, the Inquiry Commission [3] that was held following the bridge collapse raised concerns about the behavior of solid slab bridges without shear reinforcement. In the days following the Concorde Bridge collapse, the Quebec Ministère des Transports du Québec (MTQ) initiated an extensive research program aimed first at determining the causes of the Concorde Bridge collapse but also, with a broader perspective, on the reliability of solid slab bridges, especially for failures associated to shear. Except for identifying what caused the bridge collapse, the research program was limited to simply supported and continuous solid slab bridges acting as one-way beams; the few existing bridges with a cantilever system similar to the Concorde Bridge were demolished following the collapse. The program included extensive nonlinear finite element analyses (NLFEA) and testing to failure of three existing bridges. This paper deals with the experimental aspects whereas the analysis aspects were presented by Massicotte et al. in 2015 [4].

Solid slab bridge analysis

The analysis and design of solid reinforced concrete slabs in North-American Bridge Codes [1, 2] are done according empirical methods. The slabs are considered as parallel beams of unit width for computing the longitudinal bending moments and shear forces caused by live loads. Self-weight effects are assumed equally distributed across the bridge width. This simplified approach does not take into account factors affecting the load carrying mechanisms such as in-plane geometry (span/width ratio, skew) and actual support conditions. In fact, very little attention has been paid to the analysis of slab bridges, probably because they are generally perceived as one-way systems in which the main longitudinal reinforcement carries the longitudinal moments while secondary transverse reinforcement is designed using empirical rules or a minimum reinforcement ratio. Moreover, shear strength has never been a concern for solid concrete slab bridges until recently with the collapse of Concorde Bridge. Recent research in Europe also looked at the shear strength of solid slab bridges without shear reinforcement for beam shear failure modes at supports [5] or for cantilever slabs [6].

Finite element analyses of simply supported solid slab bridges [7] showed that shear stress concentrations develop at the slab corners and along the longitudinal free edges of the slab due to the transverse bending moments associated with Poisson's effect. For straight bridge geometries, these stresses are typically equal to two times those assuming a simple beam action whereas they can be significantly higher for bridges with skewed geometries. Many existing solid slab bridges present cracks on their longitudinal free edges that can be attributed to this effect [4] and recommendations for better reinforcement details have been proposed [7]. Although shear stresses are generally small in slab bridges supported on continuous supports compared to the concrete shear strength, more attention should be paid to the localized shear forces that develop at obtuse corners of skewed bridges; specifically, the presence of sidewalks and heavy barriers or the use of discrete supports should be of interest. These conditions were present on the Concorde Bridge, which had a skew angle of 20.5 degrees with heavy cantilever sidewalks.

MTQ field testing program on solid slab bridges

Three bridges were selected by the MTQ for carrying out the field-testing portion of the research program on solid slab behavior. All bridges were simply supported structures with a span length of around 10 m. The bridges contained typical mid-20th Century details: specified 21 MPa (3000 psi) concrete and plain 230 MPa (33 ksi) reinforcing steel,

AASHTO H15 or H20 design load, and no bearings, with the slab resting directly on the abutment top. The main difference between each bridge was the skew angle: 0 and 10 degrees for the first two structures and 40 degrees for the bridge described in this paper. A steel loading frame was designed specifically for the application of various load patterns associated with typical design truck configurations.

A first decommissioned concrete slab bridge was selected by the MTQ to perform field tests to failure [8]. The two-span bridge consisted of simply supported non-skewed reinforced concrete 0.46 m thick solid slab with a 9.0 m clear span. Two tests to failure were carried out with the same loading arrangement, which aimed to induce a beam shear failure mode. In both cases, the failure load exceeded 4000 kN, more than five times the maximum current code traffic design load [1]. A ductile flexural mode of failure was observed for each span, despite the occurrence of a local shear failure in a corner of one slab, at about half the maximum load level at a location where some internal defects had been observed from coring prior to testing. A second decommissioned concrete slab bridge was also tested to failure [9]. The bridge was built in 1937 and had a clear span of 9.0 m with a slab thickness of 0.53 m and a skew angle of 9.5 degrees. Similar to the first bridge, the solid slab exhibited a ductile failure mode dominated by bending. Yielding of reinforcing bars occurred prior to failure. In addition to the bending mode of failure, the presence of a shear crack was noticed at one obtuse corner. The applied load reached more than three times the maximum current code traffic design load.

The three tests carried out on the first two bridges indicated that old concrete slab bridges without shear reinforcement have large strength reserves that are not accounted for using current bridge code load rating procedures [1, 10]. Additional nonlinear finite element analyses highlighted the beneficial mechanisms that contributed to improve shear capacity, especially the role played by the axial compression forces generated by restricting horizontal movement at supports [4]. The tests demonstrated the ability of slab bridges to redistribute the load when a local failure occurs due to the intrinsic redundancy of this type of bridge construction. The nonlinear analyses confirmed that conclusion and also showed that, even without the support longitudinal restraints, solid slab bridges with small skew angles were not likely to experience local shear failures [4].

Other experimental evidence on solid slab bridges

Limited information is available on the one-way shear strength of skewed slabs and on the shear force redistribution after cracking due to bending or even shear. Morrison and Weich [11] carried out two large-scale tests on skewed slabs in which they measured high support reactions at the obtuse corner, as predicted by analysis. Although failure was governed by bending, they observed shear stress redistribution after shear cracking, which suggested that redistribution can take place in concrete slabs under certain conditions even without shear reinforcement. Jorgenson and Larson [12] performed a test on a 305 mm (12 in.) thick slab with three continuous spans to find the ultimate bending capacity. Azizinamini et al. [13] conducted two tests to failure on a five-span slab type bridge with three continuous central spans. The experimental results of these two studies indicated that the old slab bridges showed an important reserve of capacity not predicted by the typical assessment procedures.

Miller et al. [14] studied the capacity of a continuous 438 mm thick concrete solid slab over three spans with a 30-degree skew. The bridge had severely deteriorated shoulder regions, where the top main reinforcing bars were exposed. The loading system was designed to simulate the tandem axle footprint of the HS20-44 truck placed at 3.75 m from the abutment at the obtuse corner. An asymmetrical loading position was chosen to intensify torsion effects. The test results showed that the bridge had a large reserve of capacity, the equivalent of 22 HS20-44 trucks. It was shown that the boundary conditions greatly affected the bridge response. The failure began in the deteriorated region and cracking propagated through the slab. The final failure surface was identified as an unexpected shear failure. The existing deterioration on the flexural capacity was not noticeably affected, but these damages greatly reduced the shear capacity. The bridge abruptly failed in shear, but the dowel action of the reinforcement prevented collapse. Pressley et al. [15] reported the details of two destructive bending and two destructive punching shear tests on a 305 mm thick continuous-span slab bridge. During one of flexural loading tests, a brittle shear failure occurred around the loaded region, thus controlling the mode of failure despite the initiation of bending failure associated with rebar yielding. The fact that unexpected shear failure modes occurred in the investigations of Miller et al. and Pressley et al. demonstrates the difficulty of understanding and integrating the effects of local deterioration on the structural response. More in-depth studies are needed to understand the phenomena associated with the shear failure mode preceding or accompanying bending failure of a reinforced concrete slab.