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Advanced Analysis and Testing Methods for Concrete Bridge Evaluation and Design

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

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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.

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

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.

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