Field Evaluation of Staged Concrete Bridge Deck Pours Adjacent to Live Traffic

Peter J. Weatherer, M.S.1; and Brock D. Hedegaard, Ph.D., M.ASCE2

Abstract: Staged construction is the practice whereby a portion of an existing bridge is left open to traffic while the closed portion is constructed, replaced, or widened. A primary concern that has been raised in using staged construction is how traffic-induced deflections and vibrations may affect the integrity of the longitudinal construction joint between the portions of the bridge deck. The side of the bridge deck open to traffic (referred herein as Stage 1) experiences deflections due to the live and dead loads, while the adjacent side under construction (referred herein as Stage 2) is subjected to only dead loads. These displacements may degrade the structure between the reinforcing steel projecting from the existing slab into the slab under construction or impact the permeability of the longitudinal joint, resulting in additional deck degradation near the joint or leakage through the joint.

Several laboratory studies of bond degradation in bridge decks constructed in stages have been recently conducted (Ng and Kwan 2007; Swenty and Graybeal 2012; Andrews 2013), though field studies of staged construction have been sparse. Past field reports have yielded variable conclusions. Oehler and Cudney (1966) observed abnormal defects in various bridges widened in stages.

Low slump, low-water concrete and increased cover was recommended to alleviate these defects. Arnold (1966) recommended traffic control measures limiting the magnitude of traffic-induced displacements during construction. Furr and Fouad (1981) found that omitting reinforcing steel across the longitudinal joint resulted in joint degradation and recommended extending bars from the Stage 1 deck at least 24 bar diameters into the Stage 2 deck. Manning (1981) noted that the practice was widespread throughout the United States, though most bridges showed no adverse effects from maintaining traffic during staged construction. Deaver (1982) concluded from an investigation of bridges in Georgia that maintaining traffic during staged construction causes no defects. Issa (1999) reported that the most common defect in bridge decks constructed in stages was transverse cracking, which was usually attributed to thermal changes, environmental conditions, curing procedures, and, possibly, traffic-induced vibrations. However, Issa (1999) also reported that the curvature observed in freshly placed bridge decks was less than the curvature required to cause cracking in the Stage 2 deck. Committee 345 of the American Concrete Institute (ACI Committee 345 2013) published a guide summarizing the aforementioned literature and provided practices that reduce the possibility of damage.

Two of these previous studies have monitored the differential displacements during staged construction. Furr and Fouad (1981) instrumented nine bridges during and after concrete placement to quantify the relative deflections between girders adjacent to longitudinal construction joints. Deaver (1982) also conducted field monitoring of two bridges undergoing widenings, where the widened portion was isolated from the existing bridge and subsequently connected using a closure pour between the two. In both studies, deflections were measured by attaching string potentiometers to the bridge girders and measuring the absolute deflections from traffic events. The measurements were taken for random traffic events, and vehicle weights were not available. The maximum measured displacements during construction were small, with a maximum of 1.1 mm (0.043 in.), both during construction and several weeks after.
relative deflections between girders adjacent to longitudinal construction joints are summarized in Table 1. The magnitudes of these deflections were small, with a maximum recorded differential deflection of 3.0 mm (0.12 in.). There appeared to be no correlation between the bridge girder type, span length, or girder spacing and the magnitude of observed relative deflections.

The present research concerns the integrity and performance of longitudinal joints in bridge decks constructed in stages, where deflections induced by traffic on the Stage 1 deck occur during curing of the Stage 2 concrete. Specific topics include assessing the condition of existing staged construction bridge decks and estimating magnitudes of differential deflections in the region adjacent to the longitudinal joint. To quantify the displacements, an instrumentation setup was designed to be placed on bridges under construction immediately after casting the Stage 2 concrete deck. Two highway bridges were field monitored during their construction in the summer of 2016. Three-dimensional finite-element models of three bridge superstructures were created to further investigate the live load deflections in bridges during staged construction.

**Inspection of Existing Staged Construction Bridges**

To understand the types of defects that could arise in concrete bridge decks due to staged construction practices, a total of 41 bridges were visually inspected by the research team, including 23 steel girder bridges, 10 prestressed concrete girder bridges, and 8 concrete hinged-slab bridges.

No information was available regarding traffic conditions or control at the time the staged construction was performed for each of these inspected bridges. However, the intent of constructing these bridges in stages was to maintain at least one lane of traffic at all times, and so these bridges were assumed to carry traffic during and after Stage 2 concrete placement.

Visual inspection included taking still photographs from underneath and on top of the bridges. These inspections focused on the presence of any cracks, spalls, corroded reinforcement, leakage, efflorescence, etc., in the bridge decks. No lane closures were used during the inspections, and traffic-control measures were limited to a parked vehicle with flashing lights. Due to traffic considerations, it was often not possible to get near the longitudinal construction joints, which were often located in a travel lane. Therefore, in addition to traditional still photography methods, a GoPro camera, capable of recording video at 240 frames/sec, was mounted on a vehicle and used to photograph the condition of the top surface of the deck and longitudinal joint while traversing the bridge. The videos were visually examined frame-by-frame in a video editing program, and any defects were noted. Quality video footage was only possible at speeds of approximately 40 km/h (25 mi/h) or less, and therefore useful video was not collected on bridges with higher traffic volumes and speed limits, for example along interstate highways.

**Condition of Inspected Deck-on-Girder Bridges**

Each inspected deck-on-girder bridge had one of three common reinforcement details, presented in Fig. 1, for the longitudinal construction joint, which was always located between normally spaced girders. Detail A uses a simple lap splice of transverse reinforcement, lapped 37 to 51 bar diameters. Detail B uses a dowel bar splicer, where dowel bars are lapped 52 bar diameters with the transverse reinforcement on both sides of the joint and connected by a bar coupler embedded in the Stage 1 deck. Detail C uses one-piece bar couplers, where the transverse steel from both stages are simply connected using a coupler embedded in the Stage 1 deck with no lap splices on either side. Each detail was observed with and without the shear key formed into the existing edge of the deck. Visual inspections did not provide evidence that one detail was preferable over the others in terms of the joint durability and performance. Bridge decks with each detail were free of defects and performed adequately.

One issue often encountered was underconsolidated concrete in the longitudinal joint region. The presence of spliced bars and shear keys in this region results in more congestion, which may make proper consolidation more difficult, particularly with low- or medium-slump concrete commonly used for bridge decks. To prevent this, workers should use extra effort to ensure the concrete in the longitudinal joint region is properly consolidated by internal vibration. Alternatively, using one-piece bar couplers (Fig. 1, detail C) will result in the least amount of congestion and should improve this issue.

Nearly every bridge exhibited transverse cracking with efflorescence to some extent. These cracks were typically widely spaced in the midspan region and were most likely flexural cracks. In some cases, these transverse cracks were concentrated more in one side of the deck (i.e., Stage 1 or Stage 2), but there was no evidence to

<table>
<thead>
<tr>
<th>Reference</th>
<th>Bridge</th>
<th>Girder stage 1</th>
<th>Girder stage 2</th>
<th>Span length m (ft)</th>
<th>Girder spacing m (ft)</th>
<th>Max diff. deflections mm (in.)</th>
<th>Span-to-deflection ratio</th>
<th>Girder spacing-to-deflection ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deaver</td>
<td>Gordon Rd / SR 139</td>
<td>C-Stl</td>
<td>C-Stl</td>
<td>24.4 (80)</td>
<td>2.1 (7.0)</td>
<td>0.25 (0.010)</td>
<td>97.600</td>
<td>8,400</td>
</tr>
<tr>
<td></td>
<td>Old Dixie Rd. / SR 3</td>
<td>S-Stl</td>
<td>S-Stl</td>
<td>21.3 (70)</td>
<td>1.8 (6.0)</td>
<td>0.30 (0.012)</td>
<td>71.000</td>
<td>6,000</td>
</tr>
<tr>
<td>Furr and</td>
<td>I-35 / Ave. D</td>
<td>C-Stl</td>
<td>C-Stl</td>
<td>18.3 (60)</td>
<td>2.5 (8.2)</td>
<td>0.81 (0.032)</td>
<td>22.600</td>
<td>3,090</td>
</tr>
<tr>
<td>Fouad</td>
<td>I-35 / AT &amp; SF RR</td>
<td>C-Stl</td>
<td>C-Stl</td>
<td>21.3 (70)</td>
<td>2.5 (8.1)</td>
<td>1.04 (0.041)</td>
<td>20.500</td>
<td>2,400</td>
</tr>
<tr>
<td></td>
<td>I-45 / FM 517</td>
<td>C-Stl</td>
<td>C-Stl</td>
<td>16.5 (54)</td>
<td>2.4 (8.0)</td>
<td>3.05 (0.120)</td>
<td>5.400</td>
<td>790</td>
</tr>
<tr>
<td></td>
<td>I-10 / Dell Dale Ave.</td>
<td>S-PC</td>
<td>S-PC</td>
<td>26.5 (87)</td>
<td>2.6 (8.4)</td>
<td>1.52 (0.060)</td>
<td>17.400</td>
<td>1,720</td>
</tr>
<tr>
<td></td>
<td>US 75 / White Rock Creek</td>
<td>S-PC</td>
<td>S-PC</td>
<td>15.3 (50)</td>
<td>1.6 (5.4)</td>
<td>0.81 (0.032)</td>
<td>18.900</td>
<td>1,980</td>
</tr>
<tr>
<td></td>
<td>US 75 / White Rock Creek</td>
<td>C-Stl</td>
<td>S-PC</td>
<td>27.4 (90)</td>
<td>1.6 (5.4)</td>
<td>1.47 (0.058)</td>
<td>18.600</td>
<td>1,090</td>
</tr>
<tr>
<td></td>
<td>US 84 / Leon River</td>
<td>O-Stl</td>
<td>O-Stl</td>
<td>20.6 (67.5)</td>
<td>1.9 (6.3)</td>
<td>1.47 (0.058)</td>
<td>14.000</td>
<td>1,290</td>
</tr>
<tr>
<td></td>
<td>Texas 183 / Elm Fork</td>
<td>C-Stl</td>
<td>S-PC</td>
<td>15.3 (50)</td>
<td>2.0 (6.0)</td>
<td>1.02 (0.040)</td>
<td>15.000</td>
<td>1,960</td>
</tr>
</tbody>
</table>

Note: C = continuous; O = overhanging; PC = prestressed concrete; S = simply supported; Stl = steel.
suggest they were related to staged construction practices. If differential movement between concrete and reinforcement were to cause transverse cracking, cracks would be expected to be concentrated in the Stage 2 deck and more closely spaced, at approximately the same spacing as the transverse reinforcement.

Differential deflection of girders during construction will result in negative bending over the girder adjacent to the longitudinal construction joint on the side of the new deck and a possible longitudinal crack over that girder. A small number of the inspected bridges exhibited longitudinal cracks over girders adjacent to the staged construction joint. However, these cracks were not necessarily limited to the Stage 2 section of the deck. Therefore, no conclusions were drawn whether these cracks developed because of displacements during curing of the concrete or after the bridge had remained in service for years.

Signs of leakage through the longitudinal construction joint were commonly observed. A typical leaking construction joint is presented in Fig. 2(a). In some cases, spots of corrosion were noticed underneath the deck at the joint location, as presented in Fig. 2(b), indicating that the embedded steel had already begun to corrode. Newer bridges that utilized a 6.4 (0.25) × 9.5 mm (0.375 in.) saw-cut routed above the longitudinal joint and filled with low viscosity crack sealant were observed to have minimal signs of leakage through the joint, suggesting that this detail was adequate when used.

One bridge deck determined to be in poor condition was on B-13–593 in Dane County, Wisconsin. This 98 (32) × 18.0 m (58.9 ft) single-span simply supported steel girder bridge was built in 1939, and had a full deck replacement in stages in 1989. Extensive transverse cracking at approximately the same spacing as the transverse reinforcement was seen in this bridge, extending from both sides of the construction joint. There was also delamination in the longitudinal joint region and two locations with spalled concrete exposing corroded reinforcing bars, presented in Fig. 3. The wearing surface was in serviceable condition, with a few transverse cracks that had been recently sealed. The longitudinal construction joint also appeared to have been sealed recently. The only known design details for this bridge that may have contributed to this

Fig. 1. Types of longitudinal construction joint reinforcement details seen in Wisconsin: (a) detail A using lap splice; (b) detail B using dowel bar splicer and lap splices; and (c) detail C using bar couplers.

Fig. 2. Typical (a) leakage; and (b) corrosion through a longitudinal construction joint.
degradation were that the concrete deck was thin [only 178 mm (7 in.)], and the bottom layer of reinforcement in the slab was not epoxy coated. Although the deterioration in this bridge seemed compelling, it was difficult to definitively classify it as premature and attribute it to staged construction practices. At the time of the inspection, this bridge deck was 27 years old and approaching the end of its service life.

Another bridge with substantial degradation was B-70–176 in Winnebago County, Wisconsin. This continuous steel girder bridge consisted of two 35.2-m (115.5-ft) spans, was initially constructed in 1995, and was widened in 2011 to a width of 18.3 m (60 ft). Patching of the longitudinal construction joint was present along the entire length of the bridge, as presented in Fig. 4. Upon closer inspection, the patches appeared to be concentrated on the Stage 1 side of the construction joint and nearly uniformly spaced. The patching may have been used to remedy damage induced in the Stage 1 deck by attachment and removal of the formwork used to cast the Stage 2 deck. The Stage 2 deck had regularly spaced transverse cracks with light efflorescence throughout the length of the bridge, where the Stage 1 deck only had similar cracks in the mid-span regions. The transverse cracks in the widened portion of the bridge deck were believed to be shrinkage cracks, as the two deck portions were constructed 16 years apart, and therefore the Stage 1 deck may have restrained the Stage 2 deck from freely shrinking. There were neither patches nor extensive transverse cracking in the identical adjacent bridge B-70–177, which received a full deck replacement in 2010 (as opposed to just a widening as was done for bridge B-70–176 in 2011).

**Haunched-Slab Bridges**

Eight nearly identical concrete haunched-slab bridges along a stretch of Interstate Highway 41/94 near Milwaukee, Wisconsin, were also investigated. All had three spans of 10.0, 13.1, and 10.0 m (33, 43, and 33 ft), widths of 18 m (59 ft), and varied in total thickness from 250 to 700 mm (10 to 27.5 in.). They were initially constructed in stages in 1959 and then widened again using staged construction in 1970. All eight bridges received a concrete overlay in either 1980 or 1987, and then a bituminous overlay in 1998 or 2001.

The longitudinal joints in these haunched-slab bridges were in very poor condition. Many of the defects described previously for deck-on-girder bridges were also seen here, except at a more serious level. Many of these bridges had extensive patching of concrete in the longitudinal joint region, as presented in Fig. 5(a), indicating concrete had previously spalled off. However, this patching was not uniformly spaced as observed in Fig. 4. Some bridges had not yet been patched and had large spalled areas exposing corroded rebar, presented in Fig. 5(d). Other common defects in these bridges included large longitudinal cracks in Fig. 5(b) and delamination in Fig. 5(c). These longitudinal cracks visibly opened under traffic loading, which was accompanied by differential vertical movement at the construction joint measured at approximately 6.4 mm (0.25 in.). This repeated opening and closing of large cracks was slowly wearing away the concrete, which was piling up on the embankment underneath. Visible differential movement at the longitudinal construction joint proved that the continuity between deck stages had been severely degraded.

The different staged construction procedures for deck-on-girder bridges versus haunched-slab bridges may have resulted in the increased degradation of the joints in the slab bridges. For deck-on-girder bridges, the formwork is often supported on the girders. The stiffness contributed by the formwork, diaphragms, and cross braces all help distribute forces transversely and equalize deflections. In concrete slab bridges, the formwork is often supported by shoring on the ground below, meaning the Stage 2 concrete does not deflect at all. If there is no shoring below the Stage 1 deck, all differential live load deflections will be concentrated in the region just outside the shoring and possibly transferred to the reinforcement embedded in the Stage 2 slab. Minimal details were available regarding the shoring for these eight haunched-slab bridges, so it was impossible to determine if this indeed was the cause of the observed deterioration. These bridges also had a short lap splice (only 24 bar diameters) provided between construction stages, which may also have contributed to the observed degradation. Though this short lap splice was consistent with recommendations by Furr and Fouad (1981), AASHTO LRFD (2012) requires uncoated bars to be spliced 26 to 34 bar diameters, depending on the class of lap splice.

**Monitoring of Differential Displacements**

Two Wisconsin highway bridges constructed in stages during the summer of 2016 were chosen for field monitoring of the differential...
displacements across the longitudinal joint caused by live loading during staged construction.

**Instrumentation and Displacement Measurement**

To measure differential deflections during staged construction, an “instrumentation arm” setup, pictured in Fig. 6, was designed to be placed on the hardened concrete deck of Stage 1, where it would extend over the freshly placed concrete for measuring distances to points on the deck surface. The arm was designed to be long enough to extend between the two girders adjacent to the longitudinal construction joint, and thereby measure the differential displacement across the joint. Ideally, the instrumentation arm base would be placed directly over the girder nearest the construction joint on the hardened side of the deck to allow for direct measurement of deflections between the adjacent girders. The instrumentation consisted of string potentiometers with range of 250 mm (10 in.), linear variable differential transducers (LVDTs) with range of 25 mm (1.0 in.), DC accelerometers of range ±2 g, and tiltmeters with range of ±3 degrees. A sample rate of 100 Hz was used for all field monitoring.

Traffic events were determined using the excitation measured by the accelerometer at the base of the instrumentation arm. The

Fig. 5. Degradation of haunched-slab bridges: (a) patching along construction joint; (b) longitudinal crack near joint; (c) delamination in Stage 2 near joint; and (d) spalling along joint.

Fig. 6. Field monitoring instrumentation arm setup after casting Stage 2 deck (bridge B-16–136).
beginning of an event was triggered when a threshold acceleration of 0.002 g was reached, and the event ended once the acceleration dropped below the threshold value for a duration of four seconds. Each traffic event did not necessarily correspond to a single vehicle and could have been caused by several vehicles one after another. It was not possible to isolate the bridge’s response to each vehicle passing, because sometimes multiple vehicles were on the bridge simultaneously.

Differential deflections were measured using three methods: the Corrected LVDT method, the Corrected SP method, and the Deck Accelerometer method. The Corrected LVDT and Corrected SP methods were identical, whereby the change in displacement of the tip of the arm from the concrete deck was measured by LVDT or SP, respectively, and rigid body rotation of the arm was considered. Comparing the rotation of the tilmeters mounted at the base and tip of the instrumentation arm showed that the arm remained almost perfectly rigid when the base of the arm was vibrated. However, it was later determined that the tilmeters could not respond quickly enough to accurately measure the magnitude of the rotation of the arm during a typical dynamic event during the field monitoring; the period of the tilmeter response was approximately 0.4 s, which was too slow for traffic events occurring over a few seconds at most. Rigid body rotation of the arm was instead computed using the accelerometers, as explained below. Inertial effects of the displacement sensors were not considered, as dynamic properties of these sensors were not known. The resonant frequencies of the displacement sensors were assumed to be much greater than the bandwidth of frequencies in the measured response, which was approximately from 0 to 30 Hz.

The Deck Accelerometer method applied double integration of the recorded accelerations to compute absolute displacements at the base of the arm, the tip of the arm, and points on the bridge deck above the adjacent girders. By minimizing the duration of the integrated signal and using a combination of high- and low-pass filtering, compounding integration errors were reduced, resulting in an acceptable way to calculate displacements for individual traffic events.

After selecting the signal per the method previously described, accelerations with frequencies above 30 Hz and below 0.1 Hz were initially filtered out before performing the first integration. The resulting velocities with frequencies above 30 Hz and below 0.25 Hz were filtered out before performing the second integration, producing displacement of the accelerometer. Vehicles were on the monitored bridges for approximately 1–2 s, and therefore the filtering was not believed to remove the pseudostatic response of the bridge under the moving vehicle load.

Validation of the filtering and double integration was performed in the laboratory by deflecting the tip of the arm and then releasing it, thereby creating a signal with a pseudostatic response followed by a dynamic response. The double-integrated acceleration response at the tip of the arm matched well with the deflection response from an LVDT attached at the same location, as presented in Fig. 7.

The differential displacements per the Deck Accelerometer method were the differences between the absolute displacements computed from accelerometers placed on the bridge deck above the girders. Rigid body rotation of the arm was calculated by taking the difference of displacement between the tip and the base of the instrumentation arm and dividing by the length of the arm, thus allowing the Corrected LVDT/SP method to be compared to the Deck Accelerometer method. Fig. 8(a) shows the accelerometer
records from a typical traffic event during monitoring of bridge B-16–136, and Fig. 8(b) compares the time history of deflections from the same event computed by the two methods. Though the time-history responses were not identical between the two methods, the peak displacements were similar, thereby validating the effectiveness of the instrumentation and methodology.

**Monitoring of Bridge B-16–136**

Bridge B-16–136 was a simply supported prestressed concrete girder bridge in Dairyland, Wisconsin. The bridge had a single 18.3-m (60-ft) span, and the five 910-mm (36-in.) deep prestressed concrete girders had a constant spacing of 2.44 m (8 ft). The total bridge width was 11 m (36 ft). The cast-in-place concrete deck was 200-mm (8-in.) thick. At the longitudinal construction joint, the construction plans called for #5 transverse bars spaced at 165 mm (6.5 in.) to be lapped 790 mm (31 in.), equivalent to 50 bar diameters. Actual field measurements of the splice length varied between 55 and 58 bar diameters. There was no shear key formed into the edge of the Stage 1 deck. The bridge had an average daily traffic count of 1,025 (recorded in 2016) with less than 1% truck traffic.

This structure replaced an existing bridge and was constructed in stages. Stage 1 involved construction of the northbound lane and extended 300 mm (12 in.) past the centerline of the bridge. The concrete deck for the first stage was poured 43 days before the second stage was poured. The remaining portion of the deck, including the southbound lane, was constructed during the second stage. The bridge cross section and staging are presented in Fig. 9. The Stage 2 portion of the deck was covered in moist burlap and plastic to begin moist curing immediately after casting. The instrumentation arm was then moved into place at midspan of the center girder, where the largest differential deflections were expected. Because the instrumentation could not be moved into place until after the deck had been poured and finished, the data collection began approximately 4 h after the concrete pour started, and about 2 h after all the concrete had been placed. Data were collected for approximately 4.25 h.

Locations of the instruments used in monitoring bridge B-16–136 are given in Fig. 9. String potentiometer SP-2 was connected to the top layer of the embedded reinforcement, with the goal of measuring any relative displacement between reinforcement in the longitudinal joint region and the surrounding concrete. This did not prove to be successful, so it was eliminated from following tests.

During the data collection period, 228 traffic events were measured by the instrumentation setup, which corresponded to approximately one traffic event every 67 s. The total differential deflection for each event was taken to be the total range of deflection (i.e., difference between maximum and minimum deflections) during the traffic event.

To determine the probability of experiencing a certain differential deflection, the entire collection period was divided into five-minute windows, and the maximum differential deflection in each five-minute period was recorded. These maximum differential deflections were then plotted using a histogram and fit to a Type I extreme value distribution, also called a Gumbel distribution. For Fig. 9. Cross section of bridge B-16–136 with instrumentation setup during Stage 2 construction.
this research, only the largest demands were of interest. The Gumbel distribution was chosen because it only considers the maximum event in a given period of time; the frequent, smaller, and less critical demand events are not included and therefore do not skew the distribution. The Gumbel distributions for the Corrected LVDT and Deck Accelerometer methods of calculating differential deflections are given in Fig. 10.

The maximum differential deflections measured by the Corrected LVDT and Deck Accelerometer methods during this curing period were 0.96 mm (0.0379 in.) and 0.86 mm (0.0338 in.), respectively. From the fitted Gumbel distributions, for a given five-minute window, there was a 95% probability that the maximum differential deflection experienced would be less than 0.67 mm (0.0263 in.) using the Corrected LVDT method, and less than 0.64 mm (0.0253 in.) using the Deck Accelerometer method.

As the concrete hardened for the 4.25 h during which the data were collected, a reduction in the magnitude of differential deflections was expected. However, this trend was not apparent from the monitoring data. The small deflection magnitudes and randomness of traffic made undetectable any change of differential deflection with time.

**Monitoring of Bridge B-64–123**

Bridge B-64–123 was a three-span prestressed concrete girder bridge in Darien, Wisconsin. The three span lengths were 13.6, 19.5, and 10.2 m (44.5, 64.0, and 33.5 ft). The four 1.14-m (45-in.) deep prestressed concrete girders had a constant spacing of 3.75 m (12.29 ft), and the total bridge width was 12.2 m (40 ft). The cast-in-place concrete deck was 250-mm (10-in.) thick, with transverse reinforcement of #6 bars top and bottom at 180-mm (7.0-in.) spacing and longitudinal reinforcement of #4 bars top and bottom at 200-mm (8.0-in.) spacing. At the longitudinal construction joint, transverse reinforcing bars were lapped 0.94 m (37 in.), equivalent to 49 bar diameters. A shear key was provided in the edge of the Stage 1 deck. The bridge had an average daily traffic count of 10,250 (recorded in 2018) with 17% truck traffic.

This project consisted of a bridge deck replacement constructed in stages. Stage 1 involved replacement of the south lane and extended 1.26 m (4.15 ft) past the centerline of Girder 3 as presented in Fig. 11. The remaining portion of the deck, including the north lane, was constructed during Stage 2. The Stage 1 portion of the deck was cast 64 days before the Stage 2 portion. Immediately after casting the Stage 2 portion of the deck, it was covered in burlap and soaker hoses to begin wet curing. The instrumentation arm setup was then placed at midspan of the main span, where the largest differential deflections were expected. Because of the construction-staging geometry and traffic control, the base of the instrumentation arm could not be placed directly over Girder 3. The base was instead placed on the hardened Stage 1 deck between the traffic barrier and the longitudinal construction joint. It was assumed that the deck overhang would experience minimal deflection relative to Girder 3, so differential deflections could still be measured with the instrumentation arm in this location. Data collection began approximately 4.25 h after the concrete pour started and about 2 h after all the concrete had been placed. Data were collected for approximately 4.5 h.

The instrumentation was subsequently reinstalled in the same position 20 days after casting the Stage 2 deck. The deck had fully hardened and the formwork had been removed, but the bridge was not yet fully opened to traffic. Data collection for the post-construction monitoring lasted for approximately 1 h.

The instrumentation used for the field monitoring of this bridge is given in Fig. 11. During the staged construction monitoring, LVDT-2 was malfunctioning, so instead, the string potentiometer (SP-4) in this location was used to calculate the differential deflections for the Corrected LVDT/SP method. LVDT-2 was functioning for the post-construction monitoring and was therefore used for the calculation of differential deflections during this test.

During the staged construction monitoring, 1,039 traffic events were detected and measured by the instrumentation setup, which corresponded to approximately one traffic event every 15 s. Differential deflections were calculated using both methods for each of the recorded traffic events. As with bridge B-16–136, the collection period was divided into five-minutes windows, and the maximum differential deflection in each five-minute period was computed. The maximum differential deflection in each period was then plotted using a histogram and fit to a Gumbel distribution, given in Fig. 12.

The maximum differential deflections measured by the Corrected SP and Deck Accelerometer methods during the curing period were 1.01 mm (0.0399 in.) and 0.93 mm (0.0365 in.), respectively. From the fitted Gumbel distributions, for a given five-minute window, there was a 95% probability that the maximum differential deflection experienced would be less than 0.67 mm (0.0264 in.) per the Corrected SP method, and less than 0.71 mm (0.0278 in.) per the Deck Accelerometer method.

![Image](https://example.com/image.png)

**Fig. 10.** Distribution of maximum differential deflections of bridge B-16–136 calculated using (a) Corrected LVDT method; and (b) Deck Accelerometer method. PDF scaled by a factor of 3.
During the post-construction monitoring, 319 traffic events were recorded, which was equivalent to about one event every 13 s. The maximum differential deflection in each five-minute period was plotted using a histogram and fit to a Gumbel distribution, given in Fig. 13.

A reduction in differential deflection from the staged construction monitoring to the post-construction monitoring was not apparent. Using the Gumbel distributions from the Corrected LVDT/SP method, 95% of the time the expected differential deflection was less than 0.67 mm (0.0264 in.) during curing and 0.72 mm (0.284 in.) after the deck had hardened. Using the Deck Accelerometer method, 95% of the time the expected differential deflection was less than 0.71 mm (0.0278 in.) during curing and 0.61 mm (0.241 in.) after the deck had hardened. These variations were likely not significant and may have been due to the randomness of the traffic loading.

Discussion of Monitoring Results

The results from the field monitoring of bridges B-16–136 and B-64–123 showed that the differential deflections experienced during staged bridge construction, at least in these two cases, were very small. In the nearly ten hours of data collection, the largest measured differential deflection was 1.1 mm (0.0432 in.). The magnitudes of these differential deflections agreed with those recorded in previous studies (Table 1). Results from both field-monitored bridges proved to be very similar. These bridges had similar span lengths but different girder spacings and deck thicknesses, suggesting that these details may have a less significant effect on the magnitude of differential deflections during staged construction.

Comparing results from the staged construction monitoring and post-construction monitoring of bridge B-64–123, very little difference in the magnitude and distribution of differential deflections was observed. This may have been due to a transfer of loads through the formwork or diaphragms spanning between Stage 1 and Stage 2 during construction.

Numerical Analysis of Differential Displacements

To further investigate differential deflections during staged construction, finite-element models were created using ABAQUS 6.12 for three existing bridges in Wisconsin that were constructed in stages. Bridges B-16–136 and B-64–123 were modeled for comparison with the field monitoring results. The third bridge, B-70–177, was also chosen as part of the visual inspections discussed previously. Whereas the two field-monitored bridges were both prestressed concrete girder bridges with similar main span lengths, bridge B-70–177 was a steel plate-girder bridge with span lengths almost twice as long and twice as many traffic lanes that remained open during staged construction.

Modeling Assumptions

Models were composed of girders, hardened and curing concrete decks, diaphragms, cross bracing, and parapets. All materials and
components in the models were assumed to exhibit linear-elastic behavior, as the structures were expected to be within the service range during staged construction. All steel components, such as girders, cross braces, diaphragms, and reinforcing bars were given a modulus of elasticity of 200 GPa (29,000 Ksi) and a Poisson’s ratio of 0.30. Concrete components, such as prestressed girders, decks, and parapets were given a modulus of elasticity $E_c$ suggested by AASHTO (2012)

$$E_c = 4830 \sqrt{f_c} \text{ (MPa)} \quad \text{or} \quad 1820 \sqrt{f_c} \text{ (Ksi)}$$

(1)

where $f_c$ = concrete strength with the same units as $E_c$. Values for the concrete compressive strength were assumed to be equal to the design compressive strength specified on the bridge plans: 27.6 MPa (4,000 psi) for all hardened concrete decks, 24.1 MPa (3,500 psi) for parapets, and either 41.4 MPa (6,000 psi) or 55.2 MPa (8,000 psi) for the prestressed concrete girders. All concrete materials used a Poisson’s ratio of 0.20.

Results were computed for several values of the modulus of elasticity of the fresh Stage 2 concrete deck to determine how the differential deflections change as the concrete gains stiffness. No model updating or model calibration using the measured data was conducted to estimate the stiffness of the Stage 2 concrete deck. Instead, the compressive strength of the Stage 2 deck concrete $f_c(t)$ at age $t$, specified in days, was estimated using Eq. (2), suggested by ACI 209 for Type I cement (ACI Committee 209 1992)

$$f_c(t) = f_{28} \left( \frac{t}{4 + 0.85t} \right)$$

(2)

where $f_{28}$ was the design 28-day concrete strength assumed to be 27.6 MPa (4,000 psi). The investigated values for the modulus of elasticity were then calculated per Eq. (1) corresponding to those of a concrete that had been curing for 0, 0.5, 1, 3, 7, and 28 days.

The decks were modeled using shell elements with embedded layers of reinforcement. Four total reinforcement layers were included, one each for the top and bottom longitudinal bars and top and bottom transverse bars. When the Stage 2 concrete deck was modeled as plastic (fresh) concrete at an age of zero days, the steel reinforcement was not included because, before hardening, the steel
reinforcement does not work compositely with the concrete deck and provides very little stiffness to the section, if any. The steel reinforcement was introduced to the Stage 2 deck starting at 0.5 days.

The geometry and weight for the loading truck were chosen to provide an upper-bound estimate of differential deflections expected during staged bridge construction. The truck was similar to the AASHTO (2012) design truck, with three axles at a constant 4.27-m (14-ft) spacing and a 1.83-m (6-ft) width between wheels. The total vehicle weight was equal to 254 kN (57.1 kips), selected to be the maximum three-axle, 8.53-m (28-ft) truck allowed by WisDOT on Class A highways (WisDOT 2007). The load was distributed to the three axles in the same proportion as the AASHTO (2012) design truck. A 610-mm (24-in.) square contact patch was used for each wheel, and the load was applied as a uniform pressure over this area. Only static analysis was performed, and a 33% dynamic load allowance was included to account for dynamic effects. Dead loads were not included in this numerical analysis, as only the differential deflections caused by live loads were of interest.

**Bridge B-16–136 Model**

The geometry of bridge B-16–136 is described above in the section entitled “Monitoring of Bridge B-16-136” and Fig. 9, and the model is presented in Fig. 14. Prestressed concrete girders were modeled with homogeneous solid elements. Prestressing strands were not modeled because the strands do not contribute significantly to the stiffness of the section. Steel diaphragms were modeled using shell elements. All diaphragms were hot-rolled channel shapes with their webs bolted to the webs of the concrete girders. The parapet on the existing Stage 1 side of the deck was modeled using solid elements and was tied in place to the hardened deck. The parapet on the other side of the bridge was not present during pouring of the Stage 2 deck, and was therefore not modeled.

One lane of the bridge remained open during curing of the concrete deck, with a signal alternating the direction of traffic. In the model, the truck was moved along the bridge pseudostatically in both directions to determine what location would produce the maximum differential deflection between girders on either side of the longitudinal construction joint. The differential deflection was calculated at midspan at the location of the instrumentation arm during field monitoring.

**Bridge B-64–123 Model**

Bridge B-63–123, described above in the section entitled “Monitoring of Bridge B-64-123” and Fig. 11, was modeled in a similar fashion as bridge B-16–136, but special consideration was given to how the three spans were made continuous. At the piers, the ends of the girders were encased in concrete diaphragms, which were modeled by restraining the outside surfaces at the girder ends against translation in the vertical and horizontal directions. The deck was continuous along all three spans. The model for bridge B-64–123 is presented in Fig. 15.

One lane of the bridge remained open during curing of the concrete deck, with traffic traveling in one direction. In the model, the truck was moved along the bridge pseudostatically in this direction to determine what location would produce the maximum differential...
de\text{fl}ection between girders on either side of the longitudinal construction joint. The modeled differential deflection was taken at midspan at the location of the instrumentation arm during field monitoring.

**Bridge B-70–177 Model**

Bridge B-70–177 was a two-span steel plate-girder bridge in Oshkosh, Wisconsin. The two continuous spans had equal lengths of 35.2 m (115.5 ft). In 2010, this bridge underwent a full deck replacement and widening in stages. The widened bridge had six identical 1.37-m (54-in.) steel plate girders; four were from the original structure and were spaced at 3.66 m (12 ft), and two were added during the widening and were spaced at 3.20 m (10.5 ft). The cast-in-place concrete deck was 280-mm (11-in.) thick. At the longitudinal construction joint, #6 transverse bars spaced at 180 mm (7.0 in.) were lapped 940 mm (37 in.), equivalent to 49 bar diameters. No shear key was provided at the construction joint. The bridge had an average daily traffic count of 25,700 (recorded in 2017) with 9% truck traffic.

The first construction stage involved demolishing the inside 4.65 m (15.25 ft) of the existing deck and parapet, then placing the two new girders to support the widening. While traffic continued to use the remaining portion of the existing bridge, the widening was constructed. Traffic was then switched over to the newly constructed widening while the remaining portion of the original deck was removed and replaced. The plans showed that two traffic lanes remained open during Stage 2 construction.

The steel plate girders were modeled using linear-elastic, homogeneous shell elements. Steel cross braces were modeled using truss elements and were connected to the girders using pinned connections ensuring that they only experienced axial forces. The parapet was modeled using solid elements and was tied in place to the hardened deck. The model for bridge B-70–177 is presented in Fig. 16.

Both lanes of the bridge were assumed to remain open during curing of the concrete deck, with traffic traveling in the northbound direction. In the model, the truck was moved pseudostatically across the bridge in both lanes independently to determine the location that would produce maximum differential deflections. The lane closest to the longitudinal joint was called the “near lane,” while the lane farther from the joint was called the “far lane.” The location of maximum differential deflection was not specified a priori, unlike for the models representing the field monitored bridges, but was determined by observation of the model results. The location of maximum differential deflection was the same for the near and far lanes, so simultaneous loading in both lanes gave results in accordance with linear superposition.

**Modeling Results**

Table 2 summarizes the changing magnitude of predicted maximum differential deflections from each bridge model as the Stage 2 deck hardened. As expected, the differential deflection decreased as the Stage 2 deck hardened and could transfer more load between girders. However, the decrease in differential displacement from 0 to 0.5 days was far larger than the decrease in differential displacement from 0.5 days to 28 days. These results indicated that once the deck gained some nominal stiffness and changed from plastic to hardened, the differential displacements were nearly independent of the stiffness of the deck. Furthermore, the formwork was not modeled, which would add some nominal stiffness to the deck even when the concrete was plastic, and therefore the decrease in displacements was smaller than expected.

![Fig. 15. Finite-element model of bridge B-64–123: (a) top view; and (b) deck hidden to show girders and diaphragms.](image-url)
from 0 to 0.5 days may be less in the field than observed in the model.

The results from the numerical analysis of bridges B-16–136 and B-64–123 were consistent with the findings from the field monitoring. Magnitudes of differential deflections predicted by the finite-element models were similar for both bridges. The modeled results reflected key findings from the field monitoring: first, the two bridges experienced similar differential deflections; second, the
observed maximum differential displacements were approximately 1.0 mm (0.04 in.), similar to the modeled deflections after the Stage 2 deck had gained nominal stiffness; and third, the differential displacements remained nearly unchanged as the deck stiffness increased.

The numerical analysis of bridge B-70–177 showed that considerably larger deflections than those observed in the other two bridges are possible. Even when only the far lane was loaded, the predicted differential deflections were double of those predicted and observed for the two monitored bridges. This was likely caused by the longer spans and steel plate (versus prestressed concrete) girders of bridge B-70–177. Limiting traffic to the lane farthest from the construction joint would significantly decrease the differential displacement when the constructed deck is still plastic but would provide only modest benefits after the deck has gained some nominal stiffness.

Summary and Conclusions

This research has sought to establish to what degree concrete bridge decks are compromised when a staged construction process is used and curing occurs in the presence of live traffic loads. A review of previous research, field inspections of bridge decks constructed in stages, field monitoring of deflections during staged construction, and finite-element modeling of existing structures have provided the basis for several conclusions and recommendations.

Field inspections of staged-constructed bridge decks showed that most bridges show no adverse effects from being constructed in stages. Deck-on-girder bridges were, for the most part, in very good condition. The major defects that were seen in a few of these bridges could not be directly attributable to the staged construction process and were most likely influenced by the age of the bridge deck. The one defect that was commonly found in staged construction deck-on-girder bridges was underconsolidation of the concrete in the longitudinal construction joint region. Where lap splices or shear keys exist, extra effort should be made to ensure that concrete is properly consolidated. Alternatively, using one-piece mechanical splicers will reduce congestion in these areas and make proper consolidation of the concrete easier.

The inspected haunched-slab bridges exhibited longitudinal construction joints in very poor condition. While it was impossible to determine the exact cause for their deterioration, the use of staged construction and the resulting presence of longitudinal construction joints were believed to be major factors. If shoring is not provided under the portion of the bridge that is open to traffic, all the vertical displacement will translate into differential deflections in the spliced reinforcement region in the curing concrete. It is therefore recommended to provide shoring under the entire bridge throughout construction of slab bridges.

Differential deflections were measured in two prestressed concrete girder bridges during staged construction. Results from this field monitoring showed that the magnitudes of differential deflections in bridges of this type are very small and similar to those observed in previous studies. Finite-element models of the same two bridges confirmed these results for short-to-medium-span prestressed concrete girder bridges. These deflections were considered unlikely to adversely affect the integrity of the deck.

Finite-element modeling of a longer, steel plate-girder bridge showed that differential deflections can become considerably larger as the span length increases. This is especially true if more than one traffic lane is maintained during staged construction. If this is the case, it is recommended to close the lane(s) closest to the curing deck for at least 24 h. At the very least, truck traffic should be restricted to the lane farthest from the curing deck for the same amount of time. Further analyses of long-span bridges should be performed to determine how much span length and girder configuration influences the magnitude of differential deflections.

Acknowledgments

The writers would like to acknowledge the support of the Wisconsin Department of Transportation under project number 0092-16-04. The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views of the sponsors at the time of publication.

References

ACI Committee 345. 2013. ACI 345.2R-13 guide for widening highway bridges. Farmington Hills, MI: ACI.