SPECIAL FOCUS: BRIDGE INNOVATIONS

Field Application Case Studies and Long-Term Monitoring of Bridges Utilizing the Simple for Dead—Continuous for Live Bridge System

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ABSTRACT

The performance of three bridges constructed using the simple for dead load and continuous for live load (SDCL) bridge system for steel girders was monitored during and after construction to compare actual performance with predicted performance. The structure types were a box-girder bridge, an I-girder bridge and a box-girder bridge built using accelerated construction details. During construction, strains and deflections were monitored so that the degree of continuity over the pier could be determined. The design concept assumes that a simply supported condition exists during casting of the concrete deck. However, to provide lateral bracing, the concrete diaphragm—or turndown—over the pier is cast and cured prior to casting the deck. As expected, encasement of the girders provides some continuity over the pier during casting of the deck. The degree of continuity over the pier can be reduced by lowering the height of the construction joint and through the use of crack-inducing details. Long-term monitoring of the structures showed the behavior to be consistent over time with no significant deviations from the predicted bridge behavior. During the initial time period of approximately 18 months, a slight overall change in strain values was observed in concrete elements. The rate of change slowed during this period and eventually ceased. Subsequently, the response of the structure has been consistent with only small seasonal fluctuations observed. These fluctuations are expected and are generally attributable to changes in ambient temperature, relative humidity, incident solar radiation and ground freeze/thaw conditions.

Keywords: steel bridges, steel girders, SDCL, simple for dead load–continuous for live load.

INTRODUCTION

This paper presents the field application and long-term monitoring of the simple for dead load and continuous for live load (SDCL) bridge system for steel girders. The SDCL bridge system utilizes a joint detail at the interior supports that does not become continuous until after the dead loads have been applied. Prior to attaining this final continuity, the girders within the individual spans are simply supported.

This paper is one of five that describes the development and implementation of the SDCL system and provides the theoretical foundations of the system. An overview and general information regarding the behavior and design of the SDCL system can be found in Azizinamini (2014). After completion of laboratory testing (Lampe et al., 2014) and numerical modeling (Farimani et al., 2014), the concepts were utilized in three demonstration structures. The performance of these structures was monitored during and after construction to compare the actual performance with the predicted performance. The structure types were a box-girder bridge (N-2 over I-80), an I-girder bridge (Sprague Street over I-680) and a box-girder bridge built using accelerated construction details (262nd Street over I-80). The most important aspect of SDCL bridges is the detail used to connect the girders over the pier. Photos of the connection details used in each of the three case studies are shown in Figure 1.

The design concept of the SDCL system assumes that a simply supported condition exists during casting of the concrete deck. However, to provide lateral bracing, the concrete diaphragm, or turndown, over the pier is cast and cured prior to casting the deck. Encasement of the girders is expected to provide some continuity over the pier during casting of the deck. One goal of the research presented in this paper was to quantify the degree of continuity developed by the construction details. Two additional, and more general, goals were to ensure the actual behavior of the system in the final condition was consistent with the assumed behavior and also to ensure that the behavior did not change over time.

The following section describes general instrumentation details and data collection procedures that are common to all three projects. The remainder of the paper is broken into three sections, one for each case study.

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INSTRUMENTATION AND DATA COLLECTION

The instrumentation and long-term monitoring practices were similar for all three structures. Vibrating-wire type gages were used to monitor strains both on the surface of the steel and within the concrete. Vibrating-wire gages rely on the frequency of vibration of a wire within the gage. The tension in the wire, and therefore the frequency of vibration, varies with the strain of the component to which it is attached. This design results in a stable device, the zero and response characteristics of which do not vary over time. Additional instrumentation monitored displacements, rotations and weather. The long-term data were collected hourly using a data acquisition system. The collection interval was reduced during construction events such as casting of the concrete slabs. Additional details pertaining to the instrumentation and data collection can be found in the research reports for the individual structures (Ala et al., 2009; Yakel and Azizinamini, 2009; Yakel and Azizinamini, 2012).

SDCL SYSTEM USING BOX-GIRDER AND CONVENTIONAL CONSTRUCTION

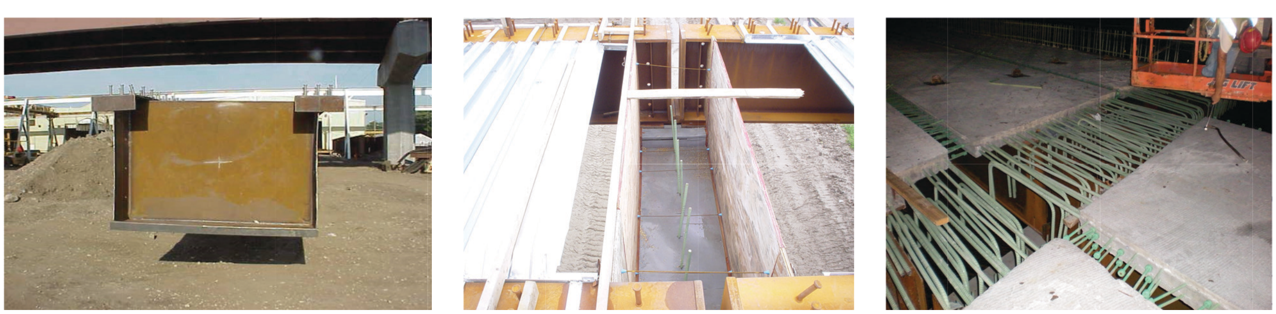
Replacement of the N-2 over I-80 overpass near Grand Island, Nebraska, provided an opportunity to construct a bridge using the SDCL detail. The first bridge to use this detail, it was also the first bridge in the United States to use HPS-100W high-performance weathering steel. The bridge utilized a box-girder configuration. The bridge was instrumented and then monitored during and after its construction. During construction, the bridge behavior was monitored to verify the simple connection behavior and also to determine the amount of possible continuity before the concrete was cast. A live load test using loaded dump trucks was also carried out prior to opening the bridge to traffic. The results of the monitoring and live load testing were used to validate finite element modeling, which allowed further investigation into the behavior of the structure and the connection detail.

Structure Description and Design Details

The construction of the bridge was part of the 2002 FHWA Innovative Bridge Research and Construction (IBRC) initiative. The bridge is a two-span steel box bridge, with each span being 139 ft long. The bridge is a grade crossing over the I-80 highway, so minimizing the interruption to traffic was a major concern for the Nebraska Department of Roads. The bridge incorporated several unique and innovative design features that facilitated both fabrication and construction. Following is a summary of the design, fabrication and construction, including solutions to challenges that faced the designer, fabricator and contractor.

Steel boxes are typically assumed best suited for spans longer than 250 ft. However, preliminary designs indicated that use of high-performance steel (HPS) and steel boxes in conjunction with the new system would result in an economical bridge. A prior study indicated that the best way to use HPS in steel bridges using I-girders is a hybrid arrangement where 50- and 70-ksi steels are used in combination (Horton et al., 2000). Preliminary designs indicated that the same conclusions also apply to the case of steel boxes. Therefore, the design of the box girders was based on the assumption that the girders will use a hybrid arrangement, with bottom flanges of the box sections using 70-ksi HPS and webs and top flanges utilizing conventional 50-ksi steel. Additional parametric studies carried out during design indicated that this hybrid arrangement is the most ideal for the bridge under consideration.

During the design phase of the bridge, 100-ksi HPS was just becoming available in the market, and there was no experience with using 100-ksi HPS in bridge construction. After the design was completed, using 70-ksi HPS in hybrid form, a decision was made to substitute the 100-ksi HPS for all webs and flange materials. This conservative replacement allowed evaluating the fabrication processes using 100-ksi HPS materials without relying on the additional strength provided.



(a) *(b)* *(c)*

Fig. 1. Bridge pier details: (a) N-2 over I-80 (box); (b) Sprague Street (I-girder); (c) 262nd Street (accelerated).

Use of HPS resulted in keeping the maximum weight of each girder under 60,000 lb, the maximum crane capacity of the local fabricators, while increasing the girder lengths to 139 ft from traditional 120-ft values. Use of HPS plates permitted using thinner plates for the bottom flange and a reduction in web depth.

General Structure Information: Location, Spans, Cross-Section, Materials

The N-2 over I-80 overpass is a two-span structure, each span being 139 ft in length. It replaces a four-span, prestressed concrete girder bridge. There are three lines of girders, spaced 16 ft 1 in. on center. The girders are steel box girders fabricated from HPS-100W high-performance weathering steel. The system utilizes the SDCL connection detail at the pier. The cast-in-place concrete deck is 46 ft 4 in. wide and 7.5 in. thick, which includes a 0.5-in. integral wearing surface. Figure 2 shows a cross-section of the structure.

The top flanges are made from ⅞ in. × 1 ft 4 in. plate. The webs are made from ⅜ in. × 4 ft 2 in. plate. The bottom flange is made from ¾ in. × 6 ft plate. These plate thicknesses remain constant throughout the entire bridge length. Cross-frames were located inside of each girder at a spacing of 25 ft.

Longitudinal reinforcement consisted of #4 bars at 12 in. on center in the top layer and #5 bars at 12 in. on center in the bottom layer. Transverse reinforcement consisted of #5 bars at 12 in. on center in both the top and bottom layers. Additional reinforcement was placed in the deck over the pier to provide the tensile continuity. Two #7 bars were placed between each #4 bar in the top layer, and a #7 bar was placed between each #5 bar in the bottom layer.

SDCL Details: Bearing Plate, Double Composite, Reinforcement

The pier connection detail is shown in Figure 3; extraneous details have been omitted for clarity. Each girder was set on two 12-in.-long steel tubes that were filled with epoxy grout. To prevent interruption to the diaphragm face steel, #4 bars were placed through the holes in the girder webs and tied with #4 stirrups. The bulkheads served as formwork for when the diaphragm was poured.

Steel bearing blocks welded to the bottom flange of the

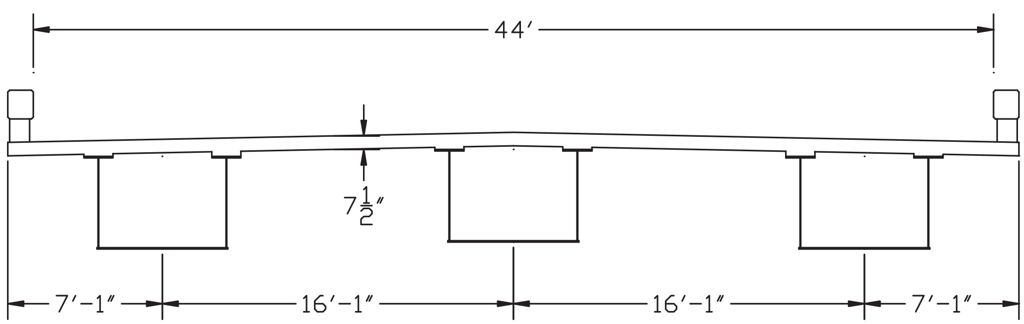


Fig. 2. Bridge cross-section.

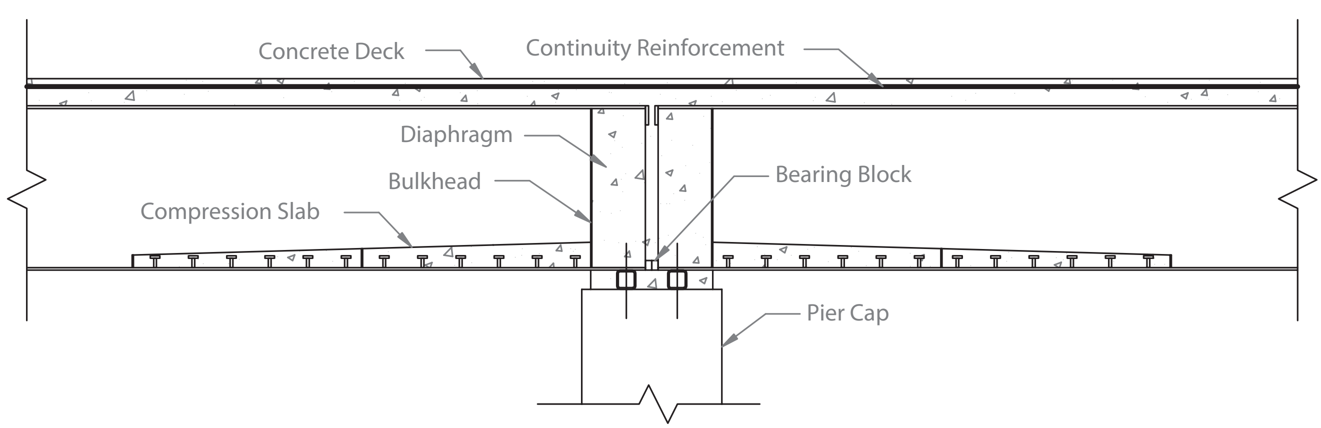


Fig. 3. Pier connection detail.

boxes transfer the compressive force from one girder to the adjacent girder directly, eliminating possibility of crushing of the concrete in the concrete diaphragm. This plate can be seen in Figure 4. Also seen in Figure 4 are small plates welded to the top flanges of the box girder. These welded plates are intended to prevent pull-out of the tension flanges from the concrete diaphragm.

Another unique feature of the bridge is the use of a new concept that allowed keeping the thickness of the bottom flange constant over the entire span length without the use of longitudinal stiffeners near the pier. Buckling capacity of the compression flange near the pier demanded either welding longitudinal stiffener to the bottom flange or increasing the thickness of the bottom flange plate in the vicinity of the pier. Besides the obvious cost issues, this option would have added to the total weight of the girder to be shipped to the job site, which was undesirable. The solution was to keep the thickness of the bottom flange constant over the entire girder length and increase the compressive capacity of the bottom flange by placing concrete inside the box, on top of the bottom flange, near the pier. This detail can be seen in Figure 3. This concrete was intended to prevent the buckling of the compression flange and allow it to reach the full yield strength of the plate. This concrete was added after the girders were placed on the support. Figure 5 shows the inside of the box, near the pier, after casting this concrete. The effect of this concrete in enhancing the strength of the cross-section near the pier was conservatively ignored.

Although the strength contribution was ignored, the stiffening effect of this additional concrete was considered in the analysis as the additional stiffness will result in increased stresses. This was accomplished by carrying out two different analyses—one with the additional concrete and one without—and using the maximum moment obtained from both analyses for design. This conservative approach was undertaken due to lack of precise knowledge about the level of the composite action that could develop between the bottom flange of the box near the pier and the concrete placed over it. This concrete was attached to bottom flange of the box using shear studs placed at the relatively large spacing of 24 in.

Bridge Construction and Monitoring

Construction of the N-2 over I-80 overpass began in May 2003. Girder fabrication began in June 2003. The construction sequence and the girder fabrication sequence are outlined in the following sections.

Girder Fabrication

As can be seen in Figure 6, the webs of the girder were perpendicular to the bottom flange. Traditionally, the webs of box girders are sloped. Making the webs perpendicular to the bottom flange significantly reduced the fabrication time and cost because this allowed the fabricator to use equipment and procedures commonly used for fabricating I-shapes.

A semi-integral abutment detail, with limited rotational stiffness, was used at each end of the two spans. Three sets of four piles were connected at the top with a steel channel, which the girders would bear on.

Girder Placement

Use of the SDCL steel bridge system significantly reduced the time required to place the girders over the supports compared to a typical arrangement that would have required a field splice within the span. In the SDCL system, each field section spans between the abutment and pier and, therefore, can be set in its final position and immediately released. Temporary shoring is not required. The bridge is a grade crossing over I-80, which was closed for only 90 minutes to place the three girders for each span. The SDCL system allowed accelerating the process of setting the girders and

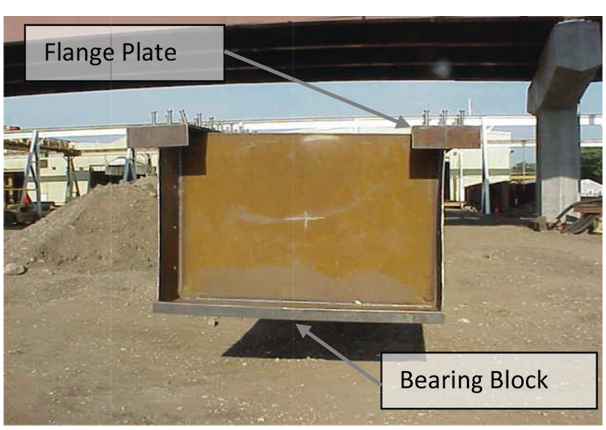


Fig. 4. Girder end to be embedded in concrete diaphragm.

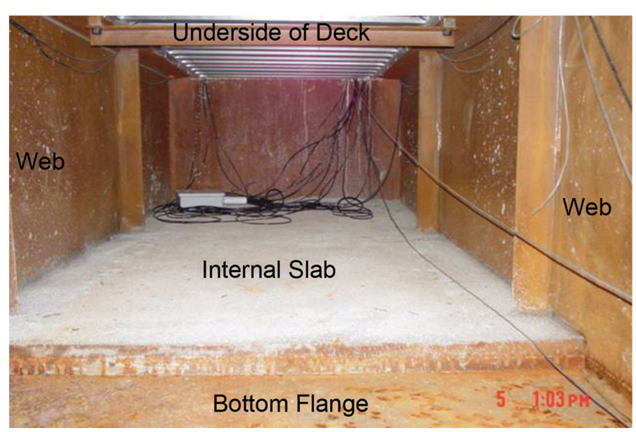


Fig. 5. Inside the box girder near the pier.

reduced the interruption to traffic. Figure 6 shows one of the 129-ft box girders being maneuvered for placement on the supports.

Installation of a box girder with the traditional construction method could exceed 4 days considering the time needed for erecting the temporary shoring and fastening of the field splices.

Global Stability Issues Related to the N-2 over I-80 Bridge

As shown in Figure 2, there were no cross-frames between the box girders. This provided an aesthetically pleasing bridge configuration while reducing the cost. However, this necessitated the detailed investigation of the global stability issue.

Designers need to be very cautious about global stability issues during construction. The AASHTO *LRFD Bridge Design Specification* (1998) does not provide any check or guideline for checking the global stability of I- or box girders during construction. The AASHTO specification only provides design provisions for checking the stability issues related to local flange, web or lateral torsional buckling of single girder. It does not, however, provide any checks for global stability. Global stability is where one box as a whole or the all girders in the cross-section move in a lateral direction, causing catastrophic collapses during construction. The global stability analysis was carried out using detailed nonlinear finite element analyses. The target factor of safety was 2.7 against collapse due to construction loads. Further, the analyses checked for adequate ductility to provide warning prior to collapse.

Evaluation of Continuity for Dead Load

In the SDCL system, the connection of the girders over the pier is assumed to not transfer moment from one span to the other during the construction phase (simple span behavior). However, due to the partially filled concrete diaphragm and reinforcement, particularly the dowels passing through the web, some amount of continuity was anticipated. Data obtained during casting of the deck from strain gages located near the pier and displacement gages placed within each span can be used to evaluate the degree of continuity over the pier. Note that the degree of continuity may vary during the course of casting the concrete in response to changes within the system, such as cracking in the concrete diaphragm.

Fig. 6. One of the girders during placement over the support.

Figure 7 shows the strain at the mid-span of both spans at the top flange, bottom flange, and web. In this chart, the first letter (S or N) indicates the span (south or north) and the second letter (B, W or T) indicates bottom flange, web and top flange, respectively. Key events are listed here, and their effect can be readily seen in the resulting strains.

1. Concrete casting began at 8:00 a.m. moving from north to south.
2. Casting entered the south span at 9:30 a.m.
3. Casting was completed at 11:45 a.m.

Moment

Figure 8 shows a comparison of the moment diagrams for a two-span bridge with a simple connection assumption, full continuity over pier assumption and experimental observation in the practice. The experimental value was obtained by examining the strain through the depth of the section at several locations and calculating the resulting moment. The resulting values at these locations allowed a curve, whose shape is consistent with the loading pattern, to be obtained. The details of this procedure can be found in the full project report (Ala et al., 2009).

As was expected, the behavior of the connection indicates partial continuity. If the transition from a simple connection to a fully continuous connection is assumed to be linear, then the percentage of continuity is given by Equation 1.

$$\text{Percent continuity} = \frac{\text{Observed} - \text{Simple}}{\text{Continuous} - \text{Simple}} \times 100 \qquad (1)$$

where
Observed = value of response variable from monitoring
Simple = value of response assuming simply supported conditions
Continuous = value of response assuming full fixity

For the response variable of positive moment at the midspan of the first span, Equation 1 indicates that 44% continuity was observed.

Table 1 shows the summary of the continuity percentage for positive and negative moments in both spans. It should be mentioned that for this bridge, no measures were taken to reduce the level of continuity over the pier during deck casting. A number of details could be used to reduce this level of continuity (Azizinamini, 2014).

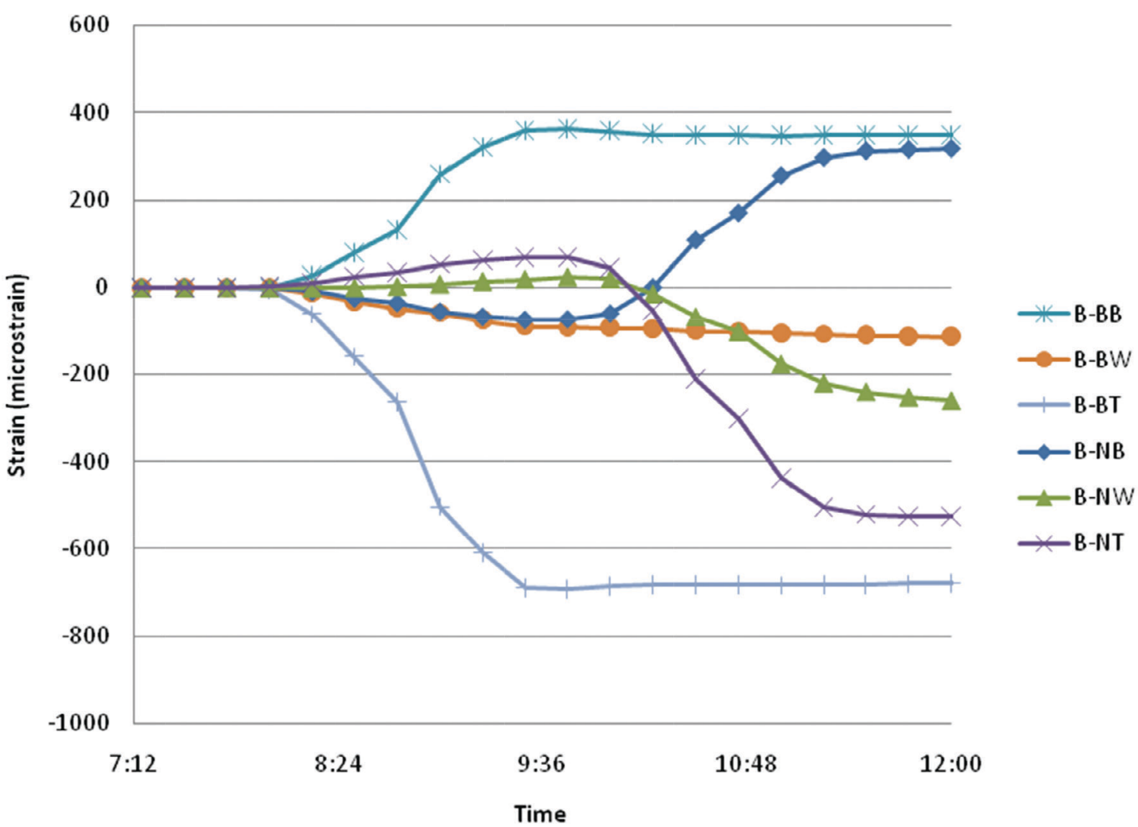


Fig. 7. Strain changes at the mid-span on both internal girders during pouring of the north and south spans.

Deflection

The amount of continuity over the pier also affects deflection. Determination of the deflection during construction is important because it will be used to specify the amount of camber during fabrication.

Figure 9 shows the observed deflection during concrete casting on the deck. Observe that the north began deflecting downward until the casting reached the pier, at which point the deflection remained relatively constant. The south span did not begin to deflect until the casting operation entered the south span. In fact, the south span initially deflected upward as casting was occurring in the north span, as would be expected of a continuous girder. This small upward deflection is a reflection of the fact that some level of continuity existed as a result of casting the concrete diaphragm first. Solutions to minimize this continuity are provided in Azizinamini (2014).

Table 2 reports the observed bridge deflections in addition to the predicted deflections obtained assuming both a simple connection and fully continuous connection.

Based on the comparisons carried out for deflections from the field measurements and the analytically calculated deflections for a simple connection and a fully continuous connection, there was 29.5% and 19% continuity for the constructed connection for deflection in the north and south spans, respectively. Note that the level of continuity based on deflection uses a single point of deflection observation while the moment-based approach uses a curve fit obtained from strain data throughout the length. These two methods are not expected to yield the same results because deflections can occur that do not cause strain in the girder. The deflection-based method is specifically used to help determine camber. The moment-based result is used for evaluating the structural behavior and validity of the assumptions. However, the strains are a result of continuity provided by the young concrete of the pier diaphragm, the effects of which will rapidly diminish. Further, once the deck has hardened, the presence of these strains will be of little consequence with regard to the strength and behavior of the system.

The degree of continuity over the pier can be reduced by lowering the height of the construction joint and through the use of crack-inducing details. One such option is shown in Figure 10, a solution used on a similar bridge system and constructed in Montana, which uses a strip of sheet metal to induce a crack to form between the ends of the two girders.

Live Load Testing

A live load test was performed to investigate several aspects of the bridge behavior. Three trucks filled with gravel were used to load the bridge. The behavior under the live load was recorded using both the long-term monitoring instrumentation and some additional sensors placed throughout

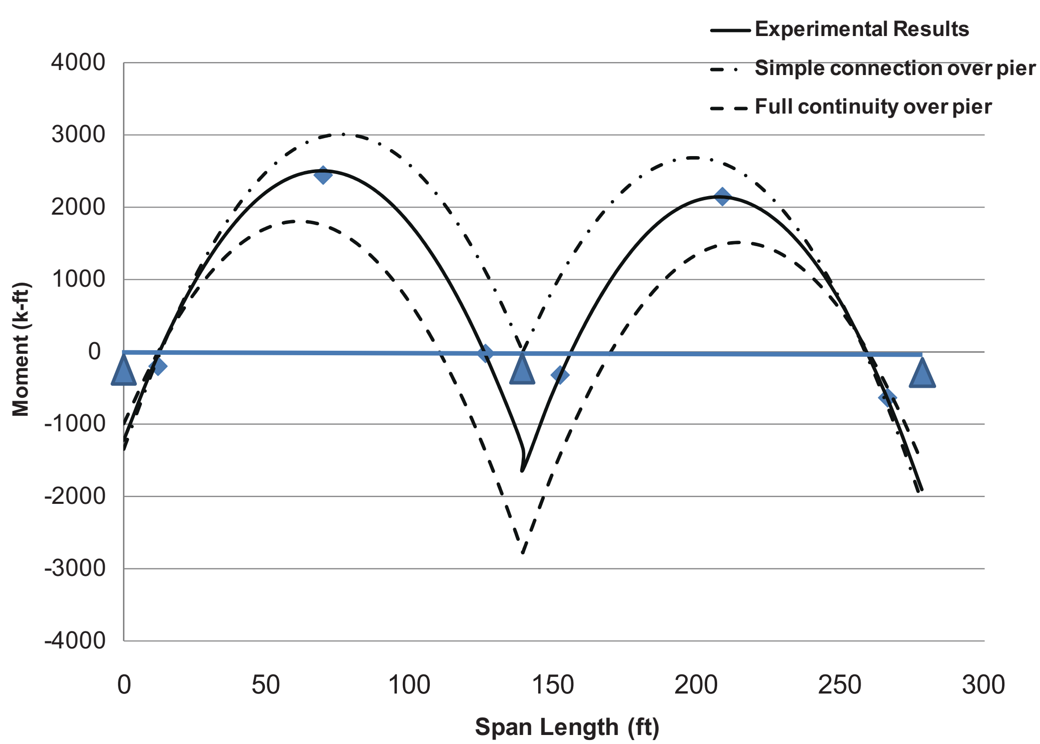


Fig. 8. Comparison among simple connection, fully continuous and experimental observations.

Table 1. Continuity Percentage for Positive and Negative Moment at Both Spans		
	North Span (%)	South Span (%)
Positive moment at midspan	44	42
Negative moment over pier	46	55

the structure. The location of the instrumentation and configuration of test vehicles were based on the objectives of the live load test, which are listed here. The observed distribution factors and load rating analysis are discussed in the following sections. More details of the live load test results are provided elsewhere (Ala et al., 2009). The objectives are as follows:

1. Compare distribution factors to AASHTO values.
2. Determine load rating according to AASHTO standards.
3. Verify the validity of assuming superposition in analysis.
4. Verify the accuracy of finite element modeling technique.
5. Investigate bottom flange slab behavior near pier.
6. Determine neutral axis locations.
7. Investigate continuity over the pier.

Experimental Determination of Distribution Factors

One of the objectives of the live load test was to determine the live load distribution factors for test data for comparison with code values. The strains used to calculate the distribution factors were taken from the bottom flange gages at the midspan section. The bottom flange tensile strains have the largest magnitude, making the resulting distribution factor less sensitive to error. Equation 2 gives the formula to calculate distribution factors from strain data (Stallings and Yoo, 1993):

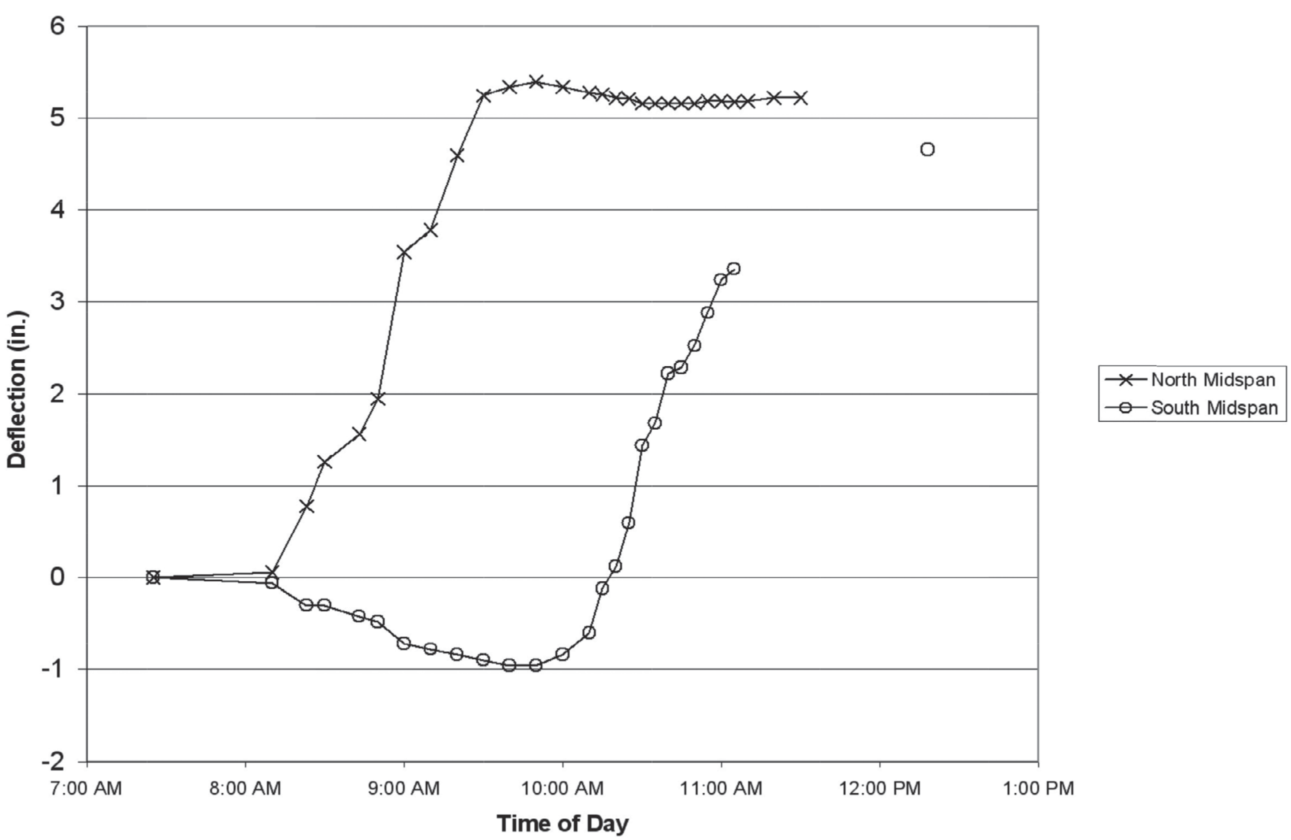


Fig. 9. Observed girder deflections during the deck pour.

Table 2. Deflection at Three Conditions for the Bridge		
	Deflection at Mid-North Span (in.)	Deflection at Mid-South Span (in.)
Simple connection over pier	6.15	5.20
Full continuity over pier	3.00	2.30
Field measurement	5.22	4.66

$$DF_i = \frac{mn\varepsilon_i}{\sum_{j=1}^{k} \varepsilon_j w_j} \tag{2}$$

where

DF_i = distribution factor for the ith girder
m = multiple presence factor
n = number of lanes loaded
k = number of girders
ε_j = bottom flange strain of jth girder
w_j = ratio of moment of inertia of jth girder to an interior girder

The weighting factor, w_j, was taken equal to 1, which means all girders are assumed to have equal stiffness. The multiple presence factor is included so the calculated values can be compared with the AASHTO values, which are given by Equation 3 (AASHTO Table 4.6.2.2.2b-1) for a concrete deck on multiple steel-box-section beams (AASHTO, 1998).

$$DF = 0.05 + 0.85\frac{N_L}{N_b} + \frac{0.425}{N_L}$$

$$\text{for } 0.5 \le \frac{N_L}{N_b} \le 1.5 \tag{3}$$

where

DF_i = distribution factor
N_L = number of lanes loaded
N_b = number of box girders

This equation was developed for simple spans but is also considered applicable to continuous-span bridges by the LRFD specifications. The distribution factor is used for both interior and exterior girders as well as for moment and shear.

Table 3 shows the calculated AASHTO distribution factors compared with the measured values for 1, 2 and 3 lanes loaded. The controlling experimental distribution factor was taken as the maximum observed value along the span for the moving load.

The difference between measured values and AASHTO values is within 15% for the three cases of number of lanes loaded. The AASHTO value compared with the measured value is slightly conservative for one lane loaded and slightly unconservative for two lanes loaded. The AASHTO distribution factor for three lanes loaded is conservative compared with the measured value. The observed result for the case of three lanes loaded is considered attributable to the conservative nature of the AASHTO equation and the particular geometry of the structure—and not a consequence of using the SDCL system.

Load Rating

Before opening a bridge to traffic, many state Departments of Transportations carry out rating calculations as part of the permanent documentation for the bridge. Consequently, rating calculations were carried out for the structure.

The bridge was load rated according to the procedure specified in the *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges* (MCE) published by AASHTO (2003). This procedure is consistent with the LRFD bridge design philosophy. The purpose of this rating is to recognize any need for the posting of loads or the strengthening of the bridge. It is also a basis for determining the safe loading capacity. The basic load-rating equation is as follows:

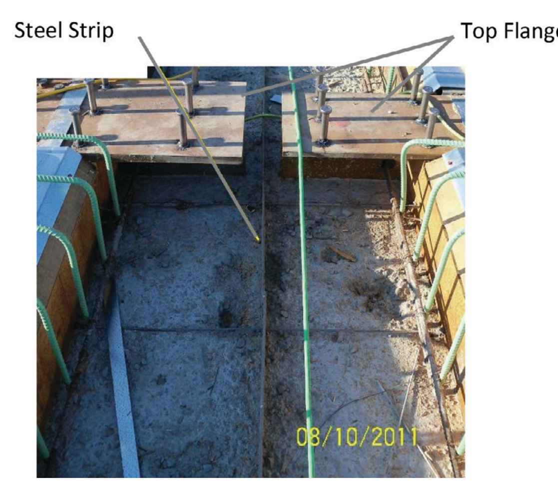


Fig. 10. Detail to induce cracking between girder ends, resulting in reduced continuity.

Table 3. Distribution Factors				
Lanes Loaded	Multiple Presence Factor	Distribution Factor		Measured/AASHTO
		AASHTO	Measured	
1	1.20	0.76	0.73	0.96
2	1.00	0.83	0.89	1.07
3	0.85	1.04	0.89	0.85
	Maximum	1.04	0.89	0.85

$$RF = \frac{C-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)\pm(\gamma_P)(P)}{(\gamma_L)(LL+IM)} \tag{4}$$

where

RF = rating factor
C = capacity
DC = dead load effect due to structural components and attachments
DW = dead load effect due to wearing surface and utilities
P = permanent loads other than dead loads
LL = live load effect
IM = dynamic load allowance
γ_{DC} = LRFD load factor for structural components and attachments
γ_{DW} = LRFD load factor for wearing surface and utilities
γ_P = LRFD load factor for permanent loads other than dead loads
γ_L = evaluation live load factor

Consideration of the bending moment and shear capacity of an interior and exterior girder resulted in a calculated inventory design rating factor (RF_C) of 1.02, governed by the shear capacity. Because this factor is >1, the bridge has sufficient capacity for the HL-93 loading. This also implies that the bridge has adequate capacity for all AASHTO legal loads, so no load posting is required.

A refined rating calculation was also carried out using live load test results. The procedure for rating bridges using live load testing is provided in Chapter 8 of the MCE (AASHTO, 2003). Following is the equation to be used to obtain load rating using live load data:

$$RF_T = RF_C \times K \tag{5}$$

where

RF_T = load-rating factor for the live load capacity based on the load test result
RF_C = load-rating factor based on calculations prior to incorporating testing results
K = adjustment factor resulting from the comparison of measured test behavior with the analytical model

The K factor is calculated directly from load testing results using the following equation:

$$K = 1 + K_a K_b \tag{6}$$

where

K_a = accounts for both the benefit derived from the load test, if any, and consideration of the section factor resisting the applied test load
K_b = accounts for the understanding of the load test results when compared with those predicted by theory

Through diagnostic load testing, the K_a and K_b values were found to be 0.55 and 0.8 respectively. The K value from Equation 6 is, therefore, 1.44, resulting in an adjusted design load-rating factor (RF_T) of 1.47.

A rating factor was also calculated for each of the AASHTO legal loads, Type 3, Type 3S2 and Type 3-3. The results of this analysis are listed in Table 4.

Long-Term Monitoring

Performance of the bridge was evaluated by continuously monitoring the response of the bridge, with data taken once every hour. Monitoring began with casting of the deck on the morning of October 29, 2003, at 8:30 a.m. and continued for the next 6 years. This was used as a starting time for the long-term monitoring portion of the project. The readings of the gages at that time used as zero values.

Variations in the data were observed to occur on several different timescales. Many of the variations were cyclic in nature and were largely attributed to temperature or other meteorological phenomena, such as humidity. These were observed to occur on daily (solar heating), multi-day (weather fronts) and annual (seasonal) bases. Note that the hourly monitoring was not intended to pick up short-term transient responses due to traffic loading.

Given that the objective of the monitoring was to evaluate the long-term performance of the structure, a filtering technique was employed to filter out the noise of short-term effects. The goal of filtering was to isolate a quiescent period of time during the day and obtain the temperature and bridge responses for this period. The values are from

Table 4. Legal Truck Load Rating					
Loading	**Weight (kips)**	RF_C	K	RF_T	**Capacity (tons)**
Type 3	50	2.92	1.44	4.21	105
Type 3S2	72	2.16	1.44	3.11	112
Type 3-3	80	2.00	1.44	2.88	115

a period each day when the thermal gradient through the depth is at a minimum. Days during which the temperature is changing rapidly have been discarded, and central averaging over the day has been utilized to further smooth the response variables. The result of temperature filtering reduced the full data set into a single temperature and the corresponding response data for each day.

Observations

In order to assess the long-term bridge behavior from the strain gages, the general trend of the strain data is of interest. Specifically, in the case that the measured strains, or their periodic fluctuations, do not significantly change over the data-logging period, it can be concluded that the bridge responses to loading remain stationary.

Consider the data shown in Figure 11, which comes from a gage located within the diaphragm at the pier. Cyclic, seasonal fluctuations are quite evident, with peaks occurring near the start of each year. On top of this pattern can be seen a trend of decreasing values that is strongest at the beginning and diminishes over time. The trend is nearly indistinguishable after 2 years, and only the seasonal fluctuations remain. This initial downward trend can be attributed to concrete creep and shrinkage. The observed seasonal pattern can be attributed to thermal effects resulting from either partial restraint at the girder ends or differential expansion and contraction of the materials. All gages from similar locations showed similar behavior.

The long-term monitoring shows that the structure behavior becomes consistent over time, with no significant deviation from the predicted bridge behavior.

SPRAGUE STREET OVER I-680 I-GIRDER

The Sprague Street Bridge over I-680 is the second bridge to be constructed using the simple for dead load–continuous for live load pier connection detail developed at the University of Nebraska–Lincoln. Similar to the previous structure, the bridge was instrumented and then monitored during and after its construction.

Structure Description and Design Details

This bridge is located in Omaha, Nebraska, and replaces a four-span steel-girder bridge. The new bridge consists of two 97-ft spans. There are four lines of girders, spaced 10 ft 4 in. apart. The girders are HPS-50W high-performance weathering steel, W40×249 rolled sections. The overall width of the bridge is 41 ft 8 in., which includes a 7-ft pedestrian walkway on one side and a 3 ft 8 in. overhang on the other. A continuous composite bridge rail exists on both sides of the bridge. Figure 12 shows a cross-section of the bridge. Cross-frames, in the form of bent-plate separators (not shown), are located at each midspan and at the quarter points of each span.

SDCL Details: Bearing Plate, Reinforcement

The pier connection detail is shown in Figure 13. Each girder was set on a steel bearing plate. Transverse reinforcement, #4 bars, was placed through the holes in the webs at the end of girder. Each girder had a 1.5-in.-thick end plate over the full depth and an additional 2-in.-thick bearing block at the bottom of the end plate. The bearing blocks transfer the compression between the bottom flanges.

The cast-in-place concrete deck is a 7.5-in.-thick deck with an integral 0.5-in. wearing surface. The width of the deck is 41 ft 8 in. Longitudinal reinforcement consists of #4 bars at 12 in. on center in the top layer and #5 bars at 12 in. on center in the bottom layer. Transverse reinforcement consists of #4 bars at 12 in. on center in the top layer and #5 bars at 12 in. on center in the bottom layer. Additional longitudinal steel, #6 bars at 12-in. centers, was placed in the top layer over the pier to transfer the tensile component of the bending moment.

Construction Sequence

Construction of the bridge began in February 2004. The pier was constructed first, followed by the west abutment. Once finished, the west girders were placed on the night of May 5, 2004. They were set at night in order to minimize disruption to traffic, as Interstate 680, below the bridge, had to be closed during that time. After setting the west girders, the east abutment was constructed. The east girders were then erected on the night of June 3, 2004. Again, it was done at night to minimize traffic disruption.

The girders of the first span of the bridge were set independently (see Figure 14). This sequence would not have been possible with the typical method of practice, which would require a field splice within the span, away from the support. Figure 15 shows the abutment end of the girders, which are set on channels spanning between piles.

After the east girders were set, the rest of the deck, diaphragm and abutment formwork were installed. The deck pour took place on July 10, 2004. The large overhang on the south side of the bridge was cast separately 6 days later to avoid placing excessive torsion on the exterior girder. The construction joint was located beneath the barrier. After the deck had cured, the barriers were slip-formed, and the pedestrian fence was installed. The bridge was opened to traffic the last week of July in 2004.

Construction Monitoring

The data acquisition system was installed and operational shortly after the girders were erected so that events such as deck pour, overhang pour and rail pours could be observed.

Casting of the deck presented an excellent opportunity to collect data to investigate the behavior of the structure during construction. Similar to what was shown in for the N-2 over I-80, the observed values obtained during the deck pour were compared with calculated values, assuming the structure behaved both as two simple spans and continuously. For the purposes of this investigation, the strains were assumed to be zero at the beginning of the deck pour so that the strains at the end of the pour were the result of the weight of the wet concrete being applied to the girders.

Strain

Figure 16 shows the strains in the top and bottom flanges along the length of the exterior girder at the end of the deck pour. Also shown are the strains predicted by assuming the system to behave as simple and continuous. In a simple-span condition, the entire top flange should show compressive strains, except at the supports, where the strains should be zero because there would be zero moment. However, Figure 16 shows tensile strains in the top flange near the abutments and near the pier. Additionally, the compressive strain in the top flange at both midspans is less than the expected value. This indicates that there is some degree of fixity at the abutments and at the pier. Similar observations can be made regarding the strains in the bottom flange.

While there is clearly some degree of continuity present, there is not enough to consider the system fully continuous. The actual degree of continuity was determined by fitting a curve with a shape prescribed by loading pattern through the observed data points. The curve is shown in Figure 16

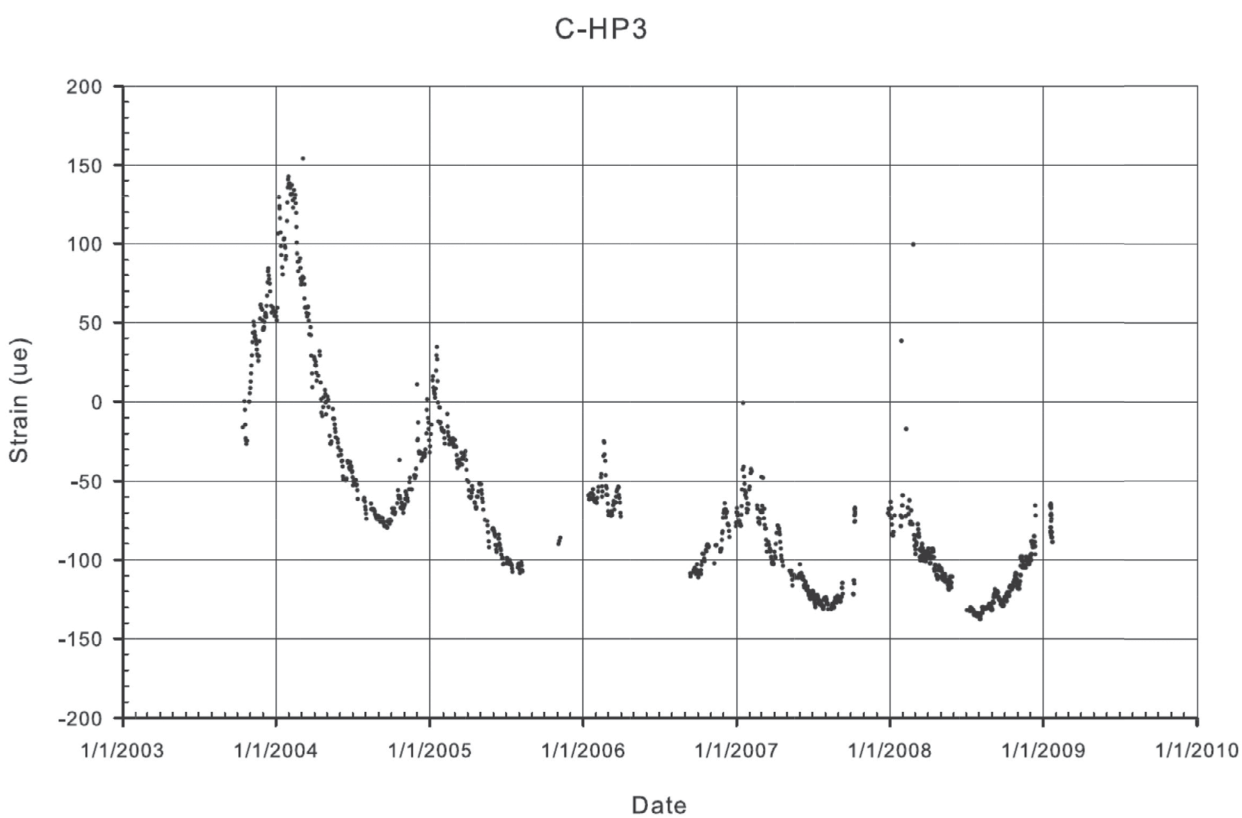


Fig. 11. Strain from concrete embedment gages located in concrete diaphragm between the girders over the pier near an exterior girder.

Table 5. Continuity Percentage for Positive and Negative Moment at Both Spans		
	South Span (%)	North Span (%)
Top flange stress—midspan	61	52
Bottom flange stress—midspan	58	49
Top flange stress—pier	8	17
Bottom flange stress—pier	46	42

as well. Table 5 summarizes the degree of fixity that was observed as calculated using Equation 1.

Deflection

During the deck pour, elevations of the screed rail at the midspans of all exterior girders were taken as concrete was placed along the length of the bridge in order to determine girder deflections and transverse structure rotation. The deflections were compared to calculated values to determine the difference. The expected deflection at midspan for the north exterior girders, points C and D, was 3.11 in., and for the south exterior girders, points A and B, was 2.04 in., assuming simply supported spans. The observed deflections at the south exterior girder at the conclusion of the deck pour were 1.32 in. at point A (west span) and 1.56 in. at point B (east span). These values are less than the expected values (2.04 in.), which is likely due to the restraining moments discussed earlier.

In the transverse direction, some rotation occurred due to the unbalanced loading from casting an overhang on the north side of the bridge, but not on the south side. There was

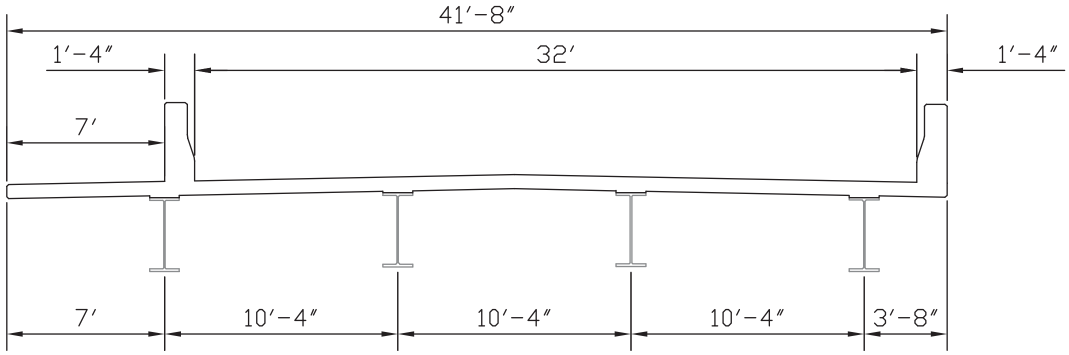


Fig. 12. Bridge cross-section.

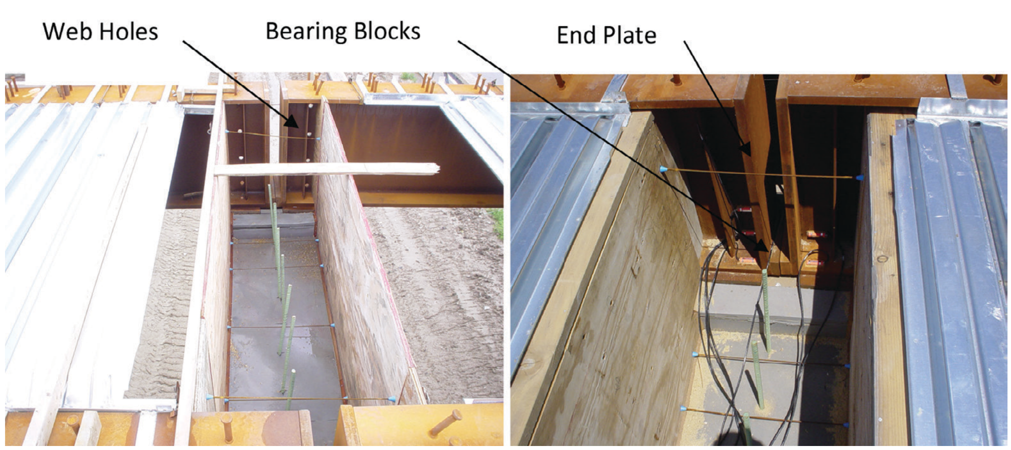


Fig. 13. Pier connection detail.

Fig. 14. Erection of first span of Sprague Street bridge over I-680.

Fig. 15. Erection of first span of Sprague Street bridge over I-680.

0.36 in. more deflection at point C than at point B, and there was 0.6 in. more deflection at point D than at point A.

The restraining moments obtained in the development of the curve fit shown in Figure 16, were used to predict deflections and resulted in values that were different from the observed by 2.9% at point A and 14.7% at point B. Note that the development assumed the end restraining moments are equal for both abutments, while in reality, they are not necessarily equal.

Girder Separation

Another item of interest during the deck pour was the data collected from crackmeters that were installed between the ends of the girders at the diaphragm. These were installed to measure the longitudinal separation between the girder ends during the deck pour. It can be seen in Figure 17 that as the pour progressed, the displacements increased, but at about 7:45 they leveled off and decreased briefly, then began increasing again. This is when the concrete placement crossed over the pier into the other span. The noted separation reduction is likely due to the finishing machine being directly supported by the pier for a period of time. The final displacements due to the deck pour were 0.19 in. at girder C and 0.17 in. at girder D. Note there was no sudden increase as would be expected if a large crack event had occurred.

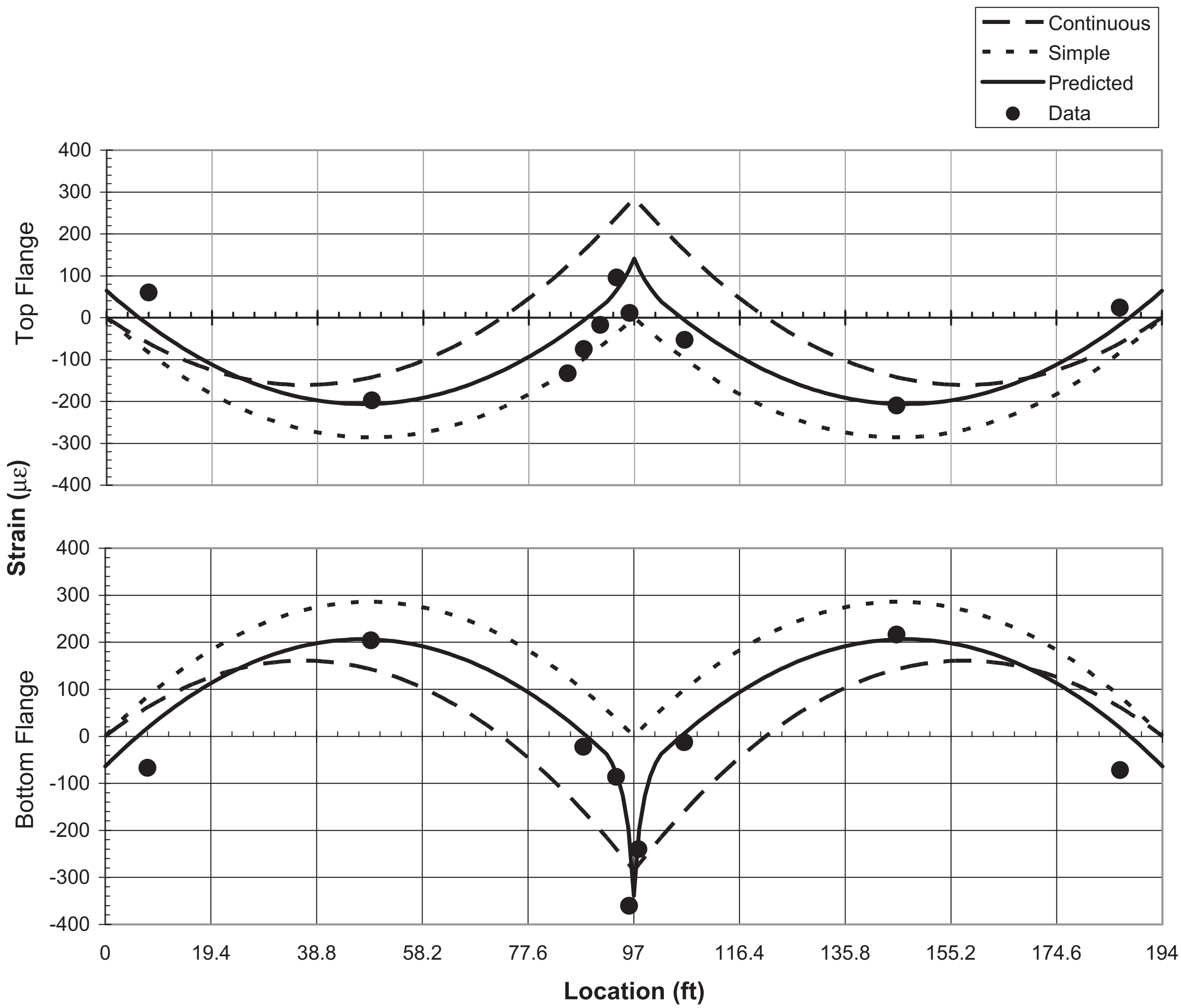


Fig. 16. Strains from analysis and measured values.

Long-Term Monitoring

The Sprague Street Bridge was continuously monitored for 5 years after completing construction. The long-term monitoring of the Sprague Street Bridge was very similar in nature and scope to the previous project. Casting of the deck began on the morning of July 10, 2004, at 6:00 a.m. This is used as a starting time for the long-term monitoring portion of the project. The readings of the gages at that time were used as zero values. Filter of the data to minimize the clutter caused by daily temperature fluctuations was again utilized. These procedures were briefly mentioned in the discussions for the N-2 over I-80 Bridge, described earlier. More detail information is provided elsewhere (Yakel and Azizinamini, 2009).

Observations

Note that the primary goal of performing the long-term monitoring of the structure was to ensure that the new, simple for dead load, continuous for live load design procedures did not exhibit unforeseen long-term performance issues. Therefore, the expected result is to see no change in gage responses other than possibly regular annual fluctuation attributable to seasonal variations in temperature, incident sunlight and relative humidity or precipitation levels. The results obtained from the gages were as expected.

A number of gages are embedded concrete. The responses of these gages display a logarithmic response characteristic of creep and shrinkage. This is most evident in the gages attached to sister bars located in the deck. The result from

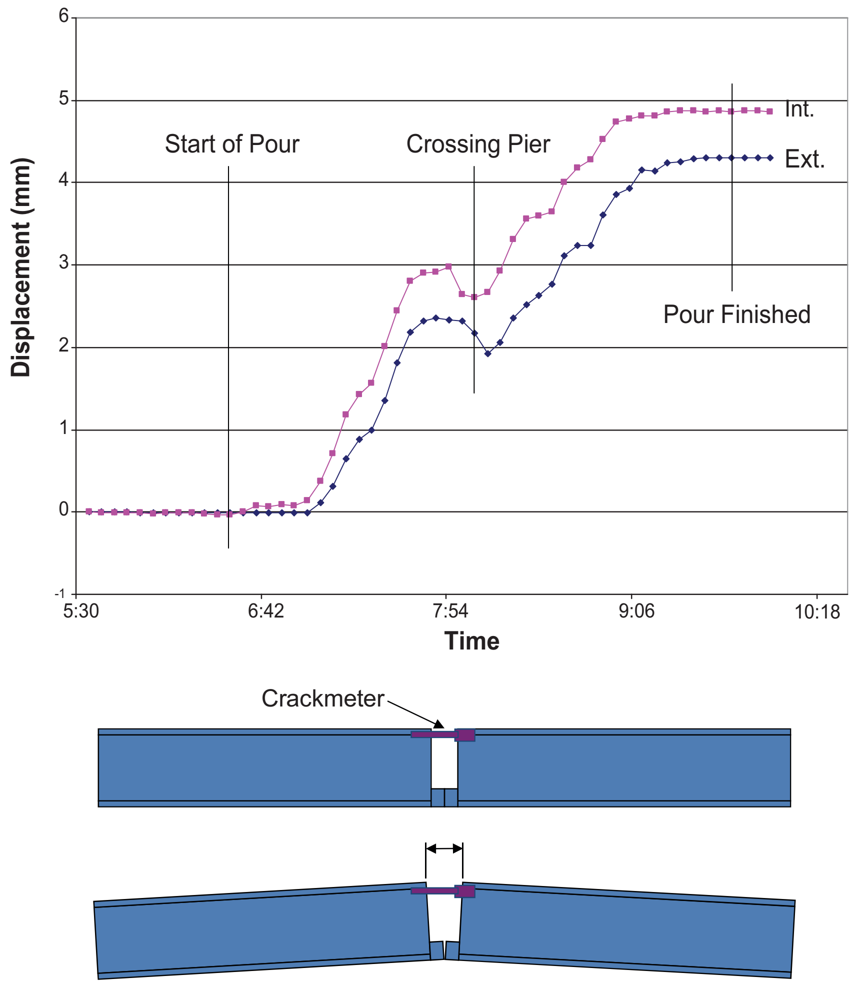


Fig. 17. Crackmeter displacements at diaphragm showing relative longitudinal movement of the top flanges.

one of the bars is shown in Figure 18 and is typical of the results observed in the other bars. After the first 2.5 years, the response had essentially flattened and remained fairly level, except for seasonal fluctuation, for the final 2 years.

A similar response can be seen in the results from the gages attached to the top flange directly over the pier. The strains are very low at this location due to the fact that the top flange is discontinuous over the pier. As a result, the response is more susceptible to influence of the concrete. In contrast, consider the bottom flange near the pier, which carries a significant compressive force and is not continuously connected to concrete. The strains from gages connected to the bottom flange nearest the pier do not show any changes in strain values other than seasonal fluctuations.

Summary

After an initial period of time when limited changes were observed, which have been attributed to creep and shrinkage of concrete, the response of the structure has been constant with only small, seasonal fluctuations. These fluctuations are expected and are generally attributable to changes in ambient temperature, relative humidity, incident solar radiation and ground freeze/thaw conditions.

262ND STREET OVER I-80—ACCELERATED CONSTRUCTION

The 262nd street bridge over I-80, located near Lincoln, Nebraska, incorporates several innovative concepts. The bridge utilizes a modular pre-top, steel, box-girder system, which allows much of the construction process to be performed prior to placing the girders and, therefore, reduces traffic disruptions. The bridge incorporates the simple for dead and continuous for live load system. The individual girders are simply supported while the pre-top deck is placed. Once in place, the modular units are joined together such that the resulting system is continuous for live load.

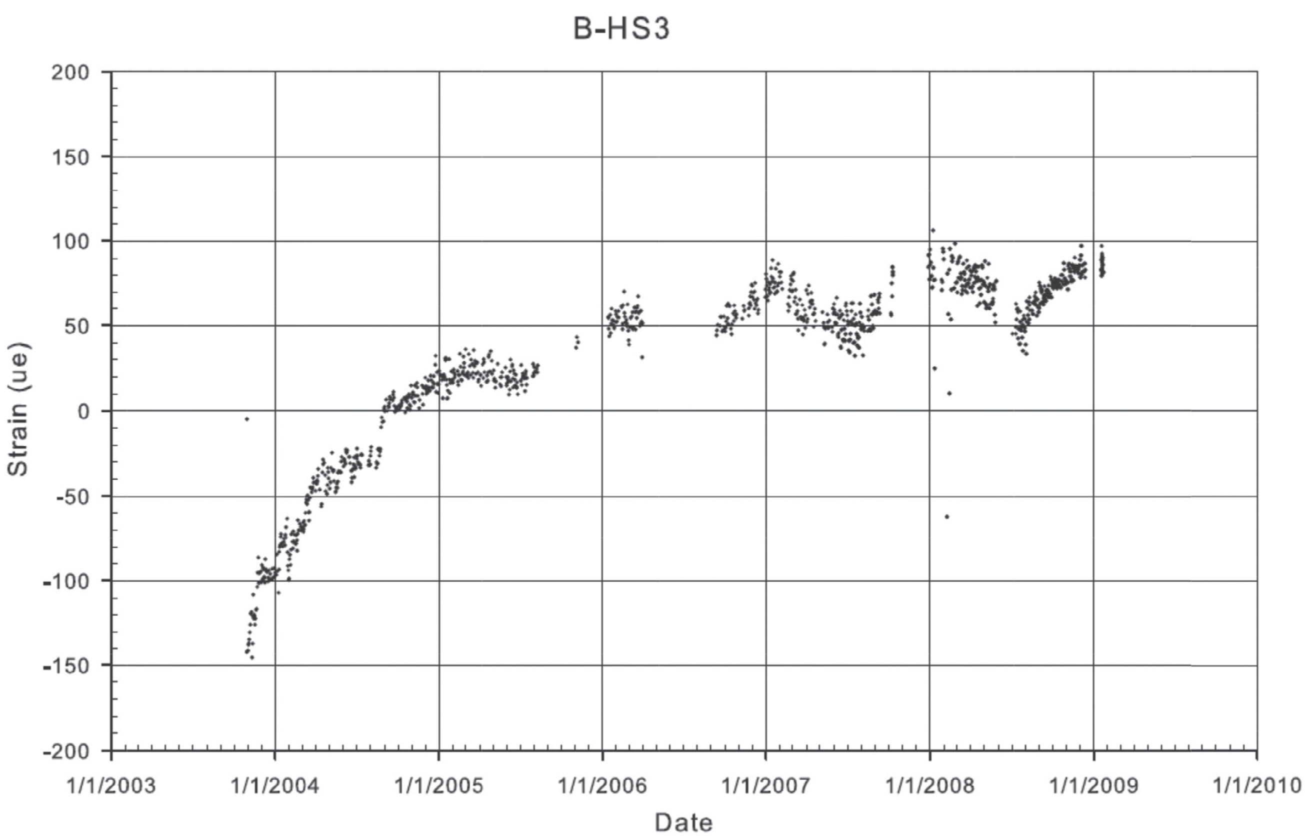


Fig. 18. Strain in concrete deck at pier (sister bar).

The steel box girders utilize high-performance steel in a hybrid configuration, 70 ksi in the bottom flange and 50 ksi in the top flanges and webs. The use of high-performance steel, combined with the simple for dead and continuous for live load system, allows eliminating plate-size transitions through the length of the structure.

The modular units must be joined in both the transverse and longitudinal directions. For the transverse direction, headed transverse reinforcement extends beyond the edges of the slab. The adjacent pre-top girders were then connected through the use of longitudinal closure strips. For the longitudinal connection at the pier, longitudinal reinforcement extends from the ends of the units and is bent downward to be cast into the concrete diaphragm. Both the transverse and longitudinal details can be seen in Figure 19.

The resulting system performs exactly as the original SDLC concept envisions and enjoys all of the accompanying benefits. Additional details and results of monitoring from that structure can be found in the paper describing the accelerated construction system (Javidi et al., 2014).

DESIGN COMPARISON OF N-2 AND SPRAGUE STREET BRIDGES

Design comparisons for the N-2 and Sprague Street bridges were carried out using two methods: (1) as a continuous beam for dead, live and superimposed loads (conventional method) and (2) as two simple beams for dead load and continuous for live loads according to the new proposed concept. A summary of the design results is given in Table 6. The variation of flange thickness was limited to two thicknesses along the girder length to include practical fabrication considerations. The web depth and thickness remained constant in both methods to satisfy the deflection requirements. In weight calculations, the weights of stiffeners and cross frames were ignored because they are similar in both alternatives. The values in the table are presented as ratios in the form of demand/resistance. The flange thickness was varied to obtain a demand-to-strength ratio for the section close to 1.00. Recall that the designs were optimized in terms of steel weight only. Note that for each case, the weight of additional steel using the new concept is about 4% for a box girder and 3% for the I-girder over the conventional option. This additional steel is provided in the positive section, which is beneficial in controlling the deflection under live loads.

Cost–Benefit Analysis

In the previous section, it was stated that application of the new construction method could result in a slight increase in weight of the steel girder. In return, the field splices were eliminated in each girder. Although each of the design parameters somewhat affected the cost of bridge construction, the major factors remain the weight of the steel girders and the cost of the field splices. Following is a summary of a cost–benefit analysis.

The cost of a steel girder consists of material, labor and facility costs. The assumed average bid unit price of fabrication for each steel girder is listed in Table 7, which was

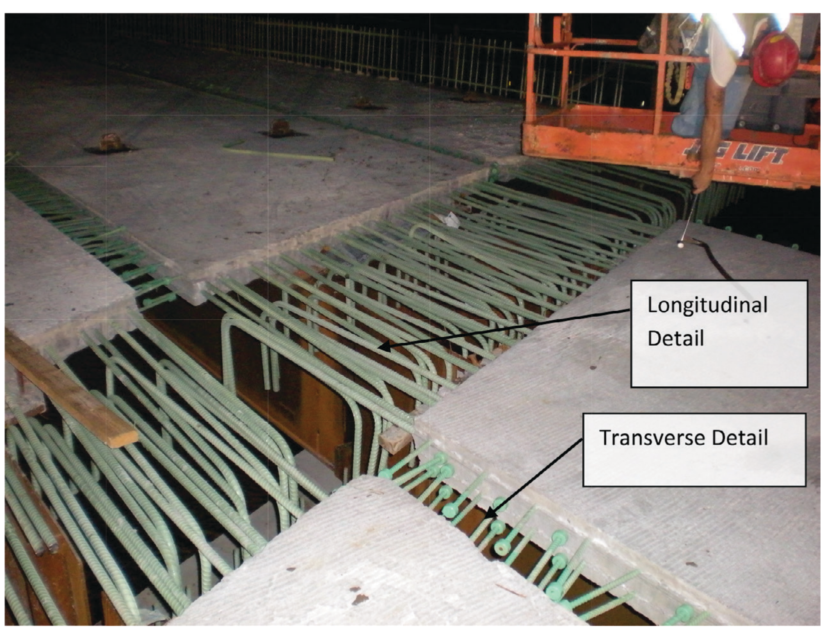


Fig. 19. Accelerated construction closure pier detail.

Table 6. Girder Sizing				
	N-2 (Box Girder)		Sprague (I-Girder)	
	Continuous for Dead and Live Loads	**Simple for Dead Loads, Continuous for Live Loads**	**Continuous for Dead and Live Loads**	**Simple for Dead Loads, Continuous for Live Loads**
Top flange sizes (in.)	16×0.64 16×1.3	16×0.7 16×0.8	15.8×0.5 15.8×2.6	15.8×0.5 15.8×2.6
Web sizes (in.)	50×0.375	50×0.375	36.56×0.75	36.56×0.75
Bottom flange sizes (in.)	72×0.64 72×1.3	72×0.7 72×0.8	15.8×0.5 15.8×2.6	15.8×0.5 15.8×2.6
Demand/strength ratio	1.00	1.00	1.00	1.00
Weight of one girder (kips)	52.47	54.63	21.46	22.07
Weight difference (percent)	4.13%		2.82%	

Table 7. Cost of Each Steel Girder				
	N-2 (Box Girder)		Sprague (I-Girder)	
	Continuous for Dead and Live Loads	**Simple for Dead Loads, Continuous for Live Loads**	**Continuous for Dead and Live Loads**	**Simple for Dead Loads, Continuous for Live Loads**
Weathering steel unit price per pound	$1.10	$1.10	$0.75	$0.75
Price of each steel girder	$57,712	$60,093	$15,627	$16,551
Splice cost (two per girder)	$12,800	$0	$4,000	$0
Splice schedule time	8 days	0 days	3 days	0 days
Total cost	$70,512	$60,093	$19,627	$16,661
Cost difference (percent)	15%		15%	

obtained from a local fabricator at the time of the study. The fabrication method—and, therefore, unit cost—is the same for both the SDCL and conventional systems. The increase in fabrication cost for the new concept is $923 for the I-girder and $2,380 for the box girder, as given in Table 7.

The cost of fabrication and installation of each field splice needs to be estimated in order to determine the extra cost of each girder when designed using the conventional method compared with the new concept. The cost of fabrication and erection consists of the cost of bolts, holes, plates, an extra crane and steel workers for installation and inspection. The unit price and required time for each item was obtained from RS Means, *Open Shop Building Construction Cost Data* (2003). A 55% surplus was added to the total cost of material, equipment and labor to consider the overhead, profit and indirect costs of the contractor. The typical field splice designed for the I-girder bridge is shown in Figure 20, and the splice cost is given in Table 7. Each girder will require two splices—a short section is first placed over the pier, and each of the span sections are then placed with one end supported on the abutment and the other spliced to the pier section. The splice geometry and construction sequence is similar for the box girder. Note that the span lengths for the I-girder bridge are such that one splice could be a welded shop splice, but this added complexity to the cost analysis and has been ignored. The information given in Table 7 indicates that although the SDCL requires slightly heavier girders, the additional cost of the steel is more than offset by the savings realized by eliminating the splices. For both cases, the net savings is approximately 15%.

The extra time for the fabrication and erection of each splice was evaluated based on a crew consisting of two steel workers for fabrication and installation, one crane operator and one inspector. The total estimated time for the field splice of the I-girder is about 20 hours. This will extend the project time by approximately 3 days because the fabrication and erection of a steel girder is usually on the critical path of the project schedule. Similarly, the time estimation for the box girder indicates that using field splices can extend the project time by more than 8 days compared to the SDCL method.

OVERALL SUMMARY AND CONCLUSIONS FROM ALL THREE PROJECTS

Construction and long-term monitoring of three bridges provided an opportunity to monitor bridges constructed using three variations of the simple for dead, continuous for live load technique both during construction and beyond.

Monitoring of the strains and deflections during construction resulted in few surprises. The design concept assumes a simply supported condition exists during casting of the concrete deck. However, to provide lateral bracing, the diaphragm, or turndown, over the pier is cast and cured prior to casting the deck. As expected, encasement of the girders does provide some continuity over the pier during casting of the deck. The detail near the pier that utilizes a concrete slab cast onto the bottom flange to provide a stiffening effect to the compression flange behaved as expected.

Close observation of the construction activities allowed the overall constructability to be observed, and communication with the construction crews provided feedback and details on any challenges posed by the new type of construction. In general, the feedback was very positive. From a construction point of view, the system is very similar to that of a regular steel girder bridge.

Live load testing of the N-2 over I-80 bridge showed that the structure behaved as expected under real-life loading and that the simplified equations for distribution factors contained in AASHTO provide conservative results.

The long-term monitoring of the structures showed the behavior to be consistent over time with no significant deviations from the predicted bridge behavior. During the initial period of time of around 1.5 years, a slight overall change in strain values was observed. The rate of change slowed during this period and eventually ceased such that beyond this initial period of time, the responses of the structures have been constant with small seasonal fluctuations. These fluctuations are expected and are generally attributable to changes in ambient temperature, relative humidity, incident solar radiation and ground freeze/thaw conditions.

A cost–benefit analysis showed that although the SDCL system requires slightly heavier girders, the additional steel cost is offset by the savings realized due to elimination of the splices, resulting in a net savings of 15% over conventional construction.

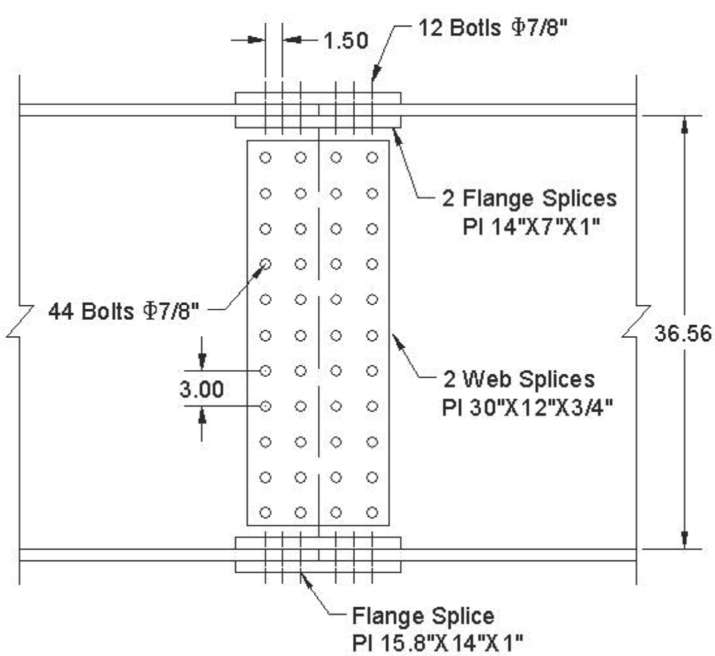


Fig. 20. I-girder field splice details.

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The opinions and conclusions presented in this paper are those of the authors and do not necessarily represent the viewpoints of the project sponsors.

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