Field Monitoring Of Positive Moment Continuity Detail In A Skewed Prestressed Concrete Bulb-T Girder Bridge

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ABSTRACT

This paper presents the results from a recently completed field study in Louisiana investigating the performance of a skewed prestressed concrete Bulb-T girder bridge made continuous. The bridge is part of the design-build John James Audubon Project that connects the cities of Saint Francisville and New Roads across the Mississippi River in Louisiana. A 96-channel monitoring system was designed and installed prior to bridge construction. The bridge utilized the positive moment continuity detail recommended in NCHRP Report 519. The Louisiana Department of Transportation and Development (LA-DOTD) chose to monitor the skewed segment with Bulb-T girders because both attributes were not within the scope of the experimental program covered in Project 12-53 that produced NCHRP Report 519. This paper presents details of the monitoring system developed for this project, which has been in service for over two years. Temperature, strain, rotation, and elongation readings are also presented and data preprocessing challenges are described. The bridge continuity is assessed based on the acquired cleaned readings. Conclusions, lessons learned and recommendations for future research are also presented.

Keywords: prestressed concrete, bridges, structural health monitoring

INTRODUCTION

The precast prestressed concrete girder bridge alternative is considered one of the most economical construction choices. Erecting precast PSC girders eliminates the need for cumbersome and costly formwork, which usually negatively impacts daily activities around the construction site. As a result, construction speed benefits from the use of PSC girder construction, especially since the girders are only erected after being cured in the casting yard. Precast elements are separate by definition and therefore are not monolithically connected by default. Therefore, many of the existing precast PSC girder bridges are constructed as simply supported spans as can be seen in Fig. 1-a.

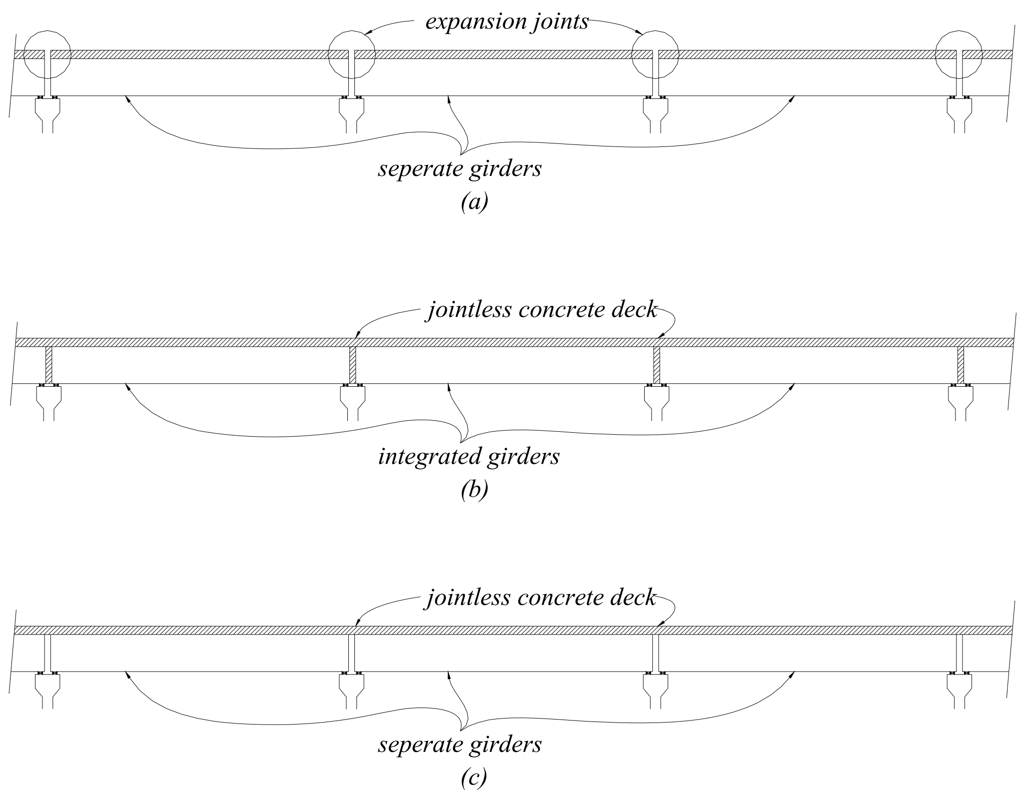
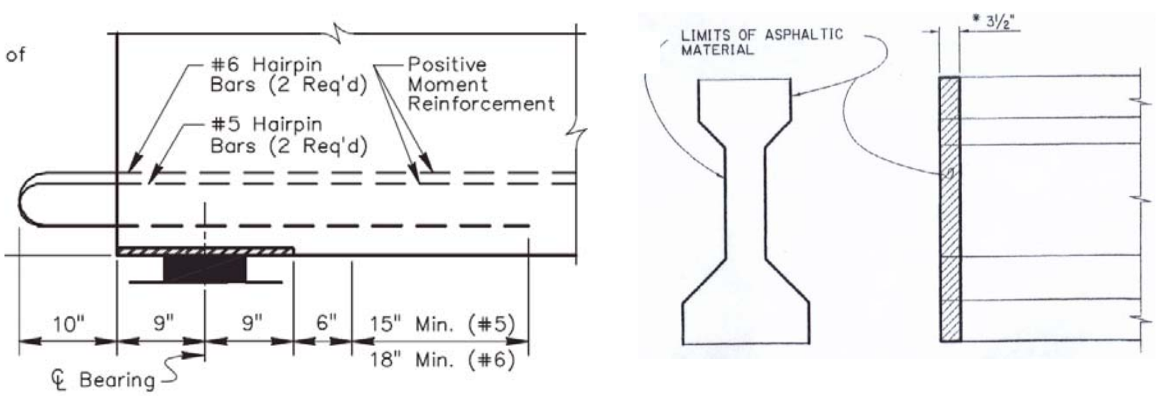


Fig. 1 Typical continuity conditions in precast PSC girder bridges: (a) simply-supported, (b) fully continuous, and (c) partially continuous [9]

Expansion joints between spans are known to cause serious problems (e.g. joint maintenance and deterioration of elements in their vicinity). Elimination of joints avoids many of these problems. Several continuity details have been used over the years in the bridge industry for slab-on-girder bridges with the goal of avoiding the aforementioned maintenance issues and reaping the benefits of continuity without the drawbacks of introducing it in large structures such as bridges (e.g., thermal movements). Two major categories of continuity solutions are commonly used, namely *full integration* details and *partial integration* details. Full integration details [Fig. 1-b] result in a fully continuous structure (both deck and girders) that can resist the bending moments that develop at the supports due to long term, thermal, and live load effects. Alternatively, partial integration [Fig. 1-c], where the expansion joints are eliminated by casting a continuous deck over the support while allowing adjacent girders movement with respect to each other, relieves some of the continuity effects [4; 9; 15].

Researchers have investigated the behavior of continuous bridge superstructures [1; 3; 6; 12-14]. Continuity details have to resist moments that develop as a result of establishing continuity. Negative moments develop due to live loads and superimposed dead loads. Positive moments develop mainly due to long-term effects such as creep, shrinkage and thermal variations. The existence of deck reinforcement over continuity diaphragms makes resisting negative moments an easy task. Conversely, special arrangements need to be made for resisting positive moments. Extending reinforcement from girder ends for development in continuity diaphragm is a common solution. In 1989, the National Cooperative Highway Research Program (NCHRP) published the findings from Project 12-29 in Report 322 [8], which is a comprehensive study on converting precast prestressed concrete girders into a continuous system. More recently, NCHRP sponsored Project 12-53 to investigate the performance of bridges made continuous and make recommendations. The recommendations were published in NCHRP Report 519 [7], which were adopted by the designer of a major project in Louisiana. The John James Audubon Project connects the cities of Saint Francisville and New Roads across the Mississippi River in Louisiana. Many of the spans of its eight bridges utilize hairpin bars to establish continuity between adjacent girders. Fig. 2-a shows the adopted hairpin detail, which is different than the Louisiana standard continuity diaphragm detail seen in Fig. 2-b. In addition to the lack of positive moment reinforcement, the Louisiana detail calls for a bond breaker to allow girder ends to move freely with respect to the diaphragm.



(a) hairpin bars [7] (b) Louisiana DOTD standard detail

Fig. 2 Difference between NCHRP 519 and Louisiana DOTD continuity details.

The LA-DOTD seized the unique opportunity offered by the construction of the John James Audubon Project and called for an investigation into the performance of this new detail on a full scale bridge to assess its long-term performance. One of the constructed bridges included a skewed segment with Bulb-T girders. Both attributes were not within the scope of the experimental program covered in Project 12-53 that produced NCHRP Report 519. The segment was therefore chosen for the study that lasted over two years. A structural health monitoring approach was used in the investigation. This paper presents the results from the study which includes monitoring data from a period of over 24 months. Also, results from a live load test are presented.

MONITORED BRIDGE AND MONITORING SYSTEM

The monitored bridge segment is a three span continuous superstructure, 242-ft long with a skewed layout. It constitutes Spans 23, 24, and 25 of Bridge #2. AASHTO Bulb–T girders (BT-72) were used for the construction of this segment. Because of the bridge's symmetry, only one of the identical intermediate bents (Bent 24 and Bent 25) was monitored. This segment was chosen because of its configuration, which has not been covered by the tests conducted in NCHRP Project 12-53; namely skewed configuration and Bulb-T girders. Fig. 3 shows a cross section of the monitored segment. More details about the bridge can be found elsewhere [10].

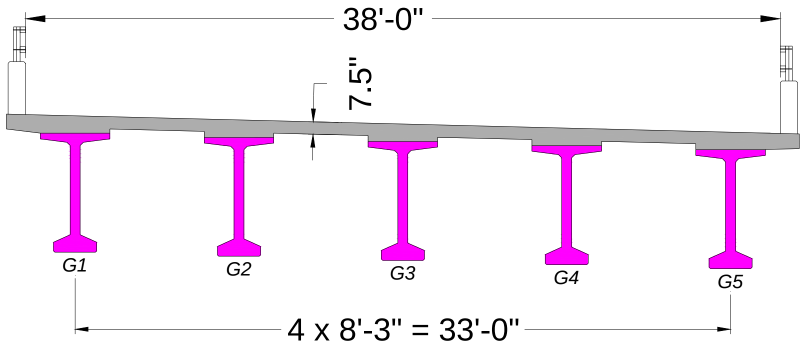
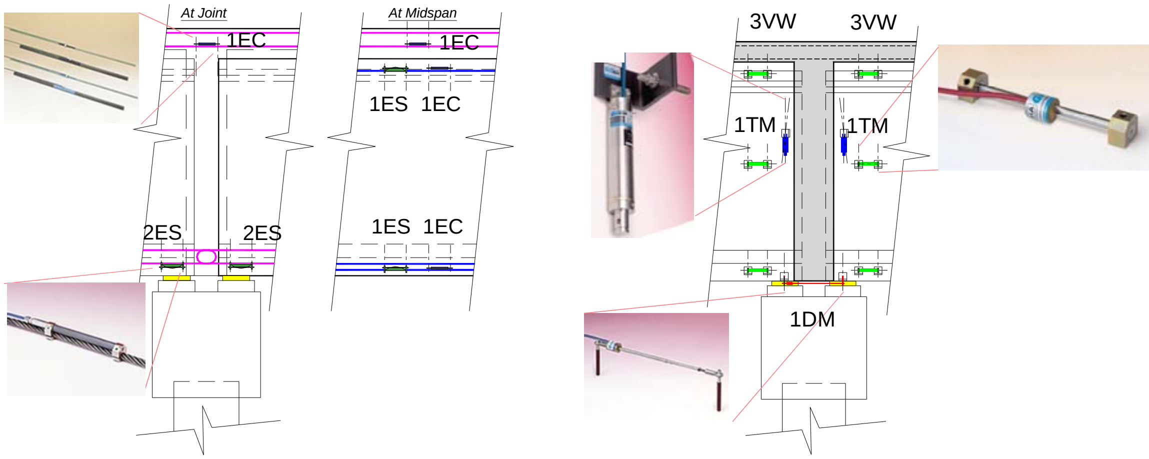


Fig. 3 Cross section of monitored bridge (Bridge #2).

A 96-channel monitoring system was designed to record essential performance measures to be used for evaluating the continuity detail. Several sensor types were chosen to measure strains, rotations, crack widths and gaps. All sensors utilized the vibrating wire technology which is known to be more suitable for long-term monitoring projects as they do not suffer from drifting [2; 5]. Embedded as well as surface-mounted sensors were employed. In all six types of sensors were utilized and the monitoring system included 66 active sensors. The sensors were strategically located at midspan and on both sides of the continuity diaphragm. Fig. 4 shows a schematic of the sensor locations. More details the bridge can be found elsewhere [10].

TEMPERATURE DATA

Fig. 5 shows the temperatures at midspan sections of one of the girders (G3) in Span 24 at three locations, namely deck, top girder flange, and bottom girder flange. It can be seen that the seasonal changes cause huge temperature fluctuations from a minimum of about 20ºF to 115ºF. The highest temperatures are always recorded in the deck because of the direct exposure to sunlight. Sunlight exposure also causes larger daily variations in the deck than the other sensors as evident by comparing the range of daily amplitude from deck and girder flanges. It should also be noted that the deck sensors were installed on the bottom mesh of the deck reinforcement to protect them during concrete casting. This position was at least 4 inches below the deck surface. The research team checked the temperature on the deck surface and compared it to those recorded by the deck sensors, which showed a difference of between 10-15°F.



(a) embedded sensors (b) surface-mounted sensors

Fig. 4 Distribution of sensors at each monitored location.

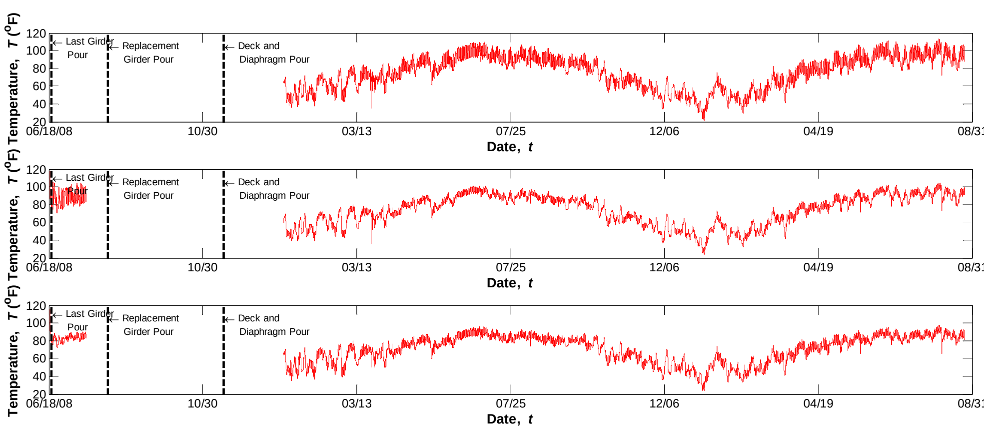


Fig. 5 Temperature readings in deck, top, and bottom girder flanges (Girder G3 – Span 24)

Fig. 6 shows a plot of the measured temperatures across the midspan section of Girder G3 in Span 24, which does not benefit from any shading offered by the barriers. Also shown in the figure is the design gradient as per AASHTO-LRFD specifications. It can be seen that the design gradient matches the measured temperatures well. The higher temperatures at the very top of the deck were not capture by the monitoring system because of the position of the deck sensors as discussed earlier. These results give confidence that designing precast PSC girder bridges using AASHTO-LRFD specified temperature gradient is adequate. It will be shown later that the temperature gradient effect has a significant impact on the performance of continuous precast PSC girder bridges.

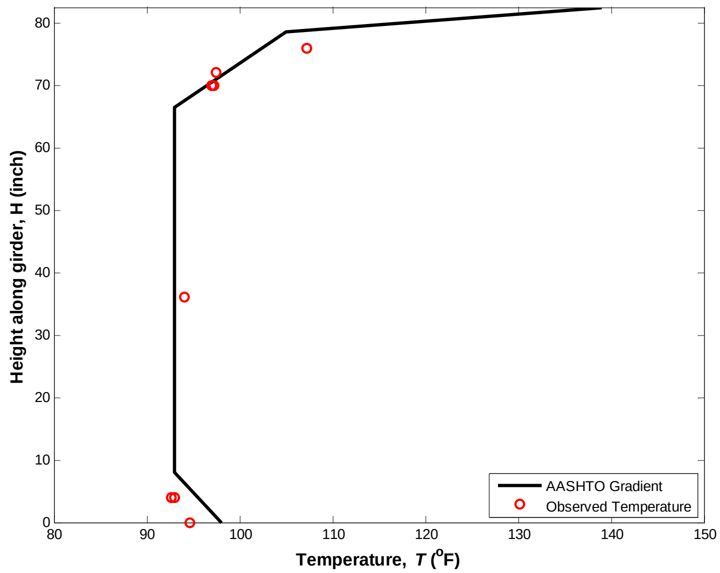


Fig. 6 Measured temperature gradient (3:00 PM – August 10, 2010 – Girder G3 – Span 24)

HAIRPIN STRAIN DATA

Strain sensors installed on the hairpin bars on both sides of the diaphragm (see Fig. 4-a) revealed a lot of important information about the performance of the continuity detail. Seasonal and daily temperature variations are clear to impose strains on the hairpin bars. By comparing strain readings shown in Fig. 7 for Girder G3 at two similar dates (e.g. 01/2009 and 01/2010), it seems that permanent residual strains take place due to creep effects. It should be noted that continuity was established after 101 days after the casting of Girder G3, which is more than the required 90-day period specified in AASHTO-LRFD specifications. As a result, the creep effect is small and seems to diminish with time. The more interesting observation from these strain plots is the daily strain variations. It is clear that the hairpin bars are subjected to large strains especially during summer months. This is due to the temperature gradient effect discussed in the previous section. Hundreds of microstrains are recorded as daily strain fluctuations. These strains are capable of initiating cracking in the vicinity of the continuity diaphragm, especially if combined with other sources of tension (e.g. creep and live load effects). It is therefore concluded that temperature gradient effects be considered in the design of continuous precast PSC girder bridges. Finally, Fig. 7 shows that strains on both sides of the continuity diaphragm seem to mirror each other, which is an indication of force transfer between adjacent girders. This observation will be corroborated by readings obtained from the live load test discussed later in the paper.

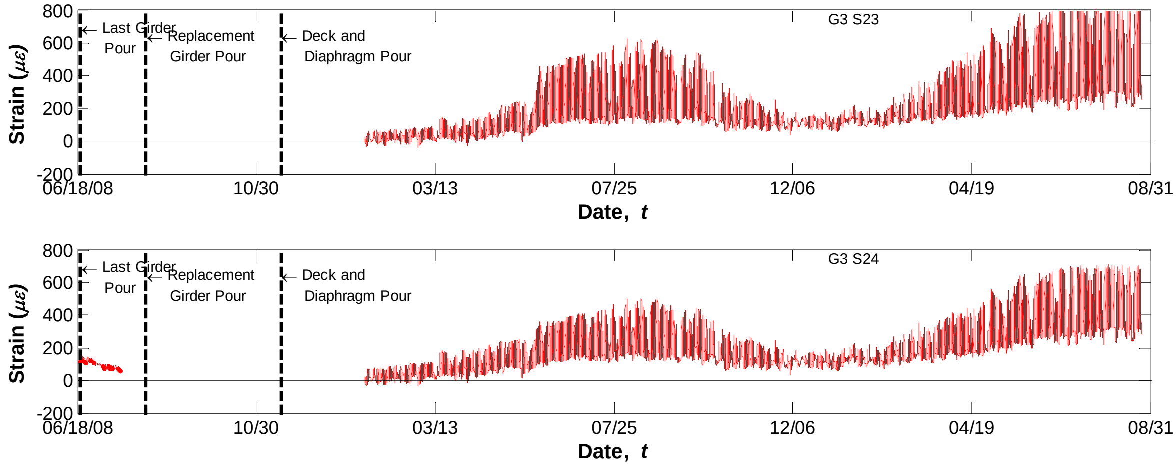


Fig. 7 Strains in hairpin bars at both sides of continuity diaphragm (Girder G3)

RELATIVE MOVEMENT BETWEEN ADJACENT GIRDERS

The relative movement between the bottom flanges at the ends of the adjacent girders on both sides of the continuity diaphragm was investigated using the gapmeters installed at Girders G1, G3, and G5. Positive displacements imply that the distance between the bottom flanges have increased, i.e., the girders are applying tension on the diaphragm, and vice versa. Fig. 8 shows a plot of the temperature corrected readings from all three gapmeters, which were installed on Girders G1, G3, and G5 across the continuity diaphragm at Bent 24. It can be seen from the figure that Girders G1 and G5 experienced far less seasonal and daily changes than Girder 3 due to the smaller temperature gradients experienced by G1 and G5 because of their vicinity to the barrier which shades the deck over these girders. Quantitatively, the gage lengths for these extended DM gages were 46.0 in., 43.0 in., and 45.5 in. for Girders G1, G3, and G5, respectively. This means that if a joint was cast monolithic with the girders, the resulting daily strain changes would have been equal to about 0.0150/46=326 $\mu\varepsilon$, 0.0325/43=756 $\mu\varepsilon$, and 0.0175/45.5=385$\mu\varepsilon$ for Girders G1, G3, and G5, respectively. It should also be noted that the daily changes are less in the cold months (December through February) than in the summer. This is due to the smaller temperature gradients during cold months, which in turn reduces girder rotations at the joint.

GIRDER END ROTATIONS

Fig. 9 shows a plot of the recorded rotations from the tiltmeters installed on the webs at Girder G3 ends on both sides of the monitored continuity diaphragm (Bent 24). It can be seen from the plot that rotations on both sides of the continuity diaphragm follow the same trend. This means that the girders are rotating in the same direction, which indicates that the continuity diaphragm is doing its job of providing continuity between the girders.

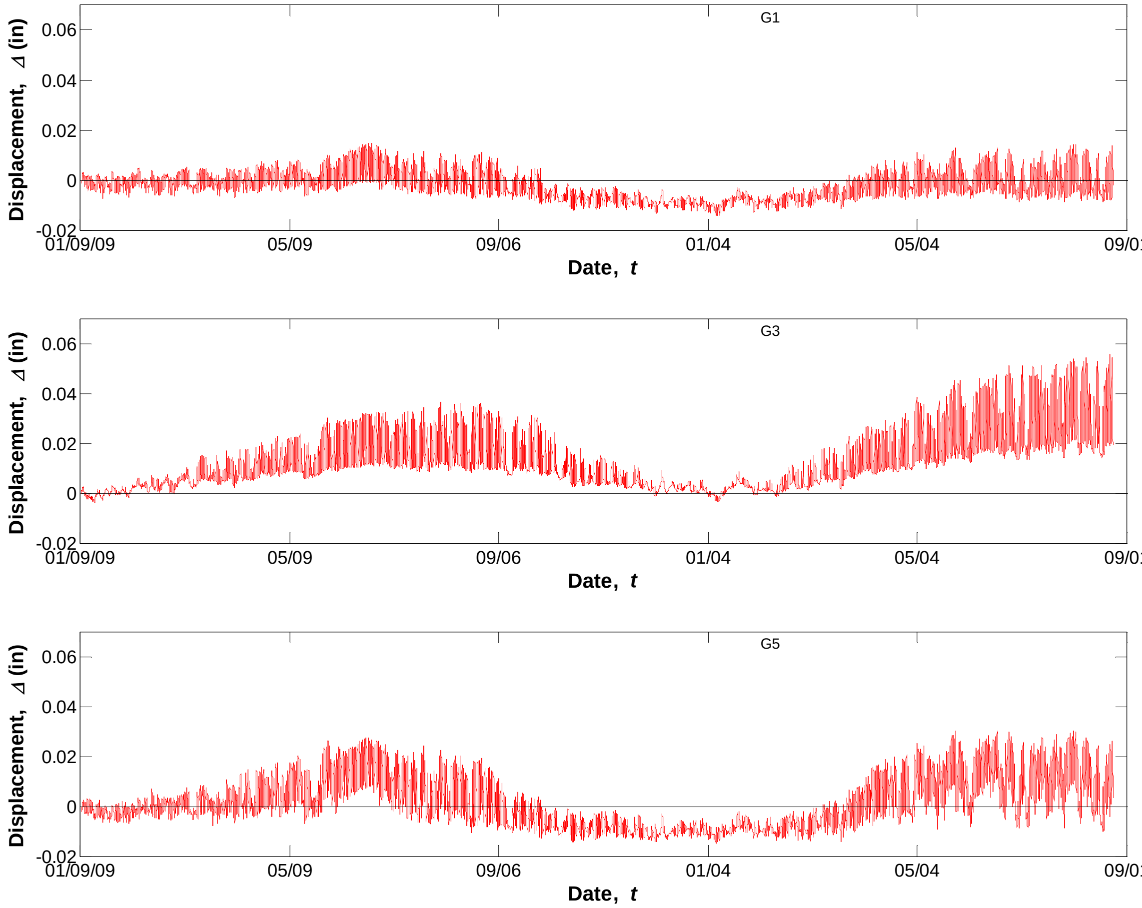


Fig. 8 Gapmeter displacements for Girders G1, G3, and G5

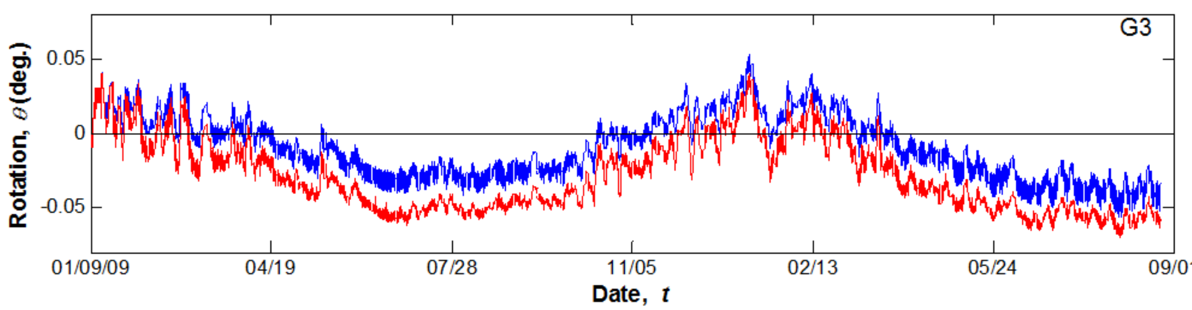
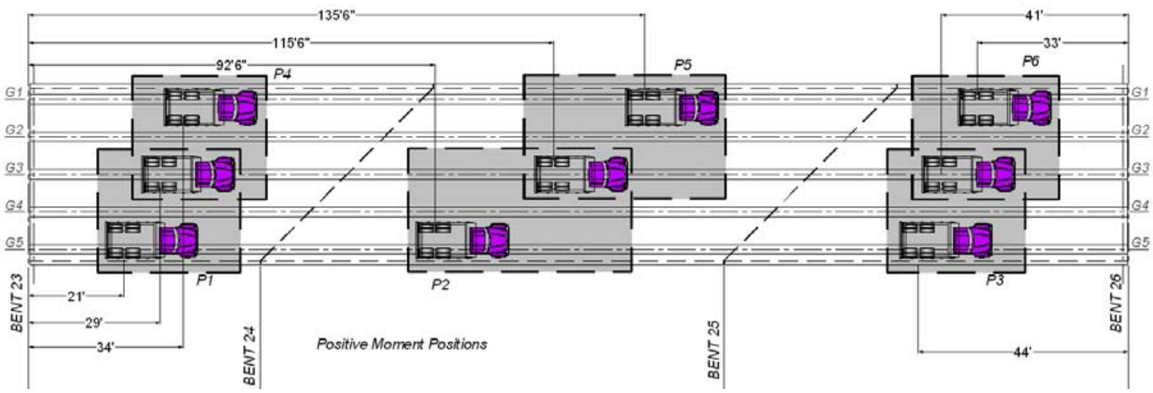


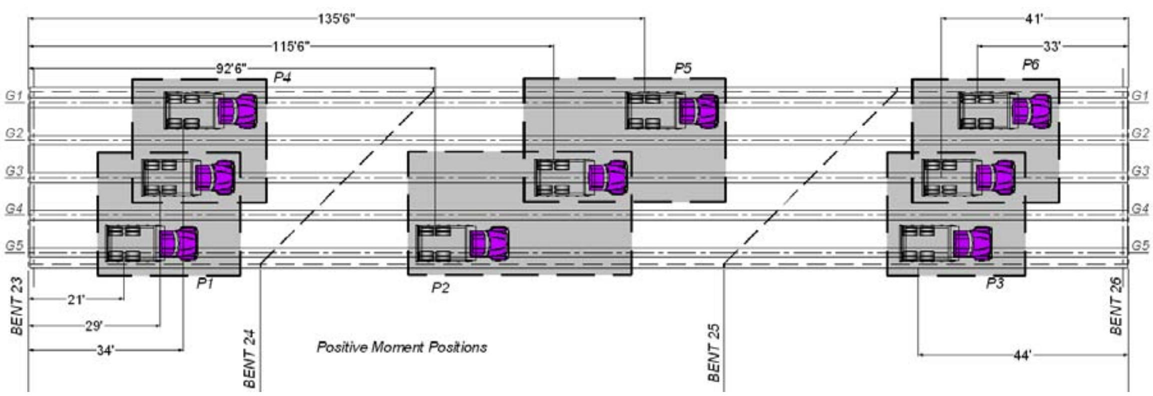
Fig. 9 Rotation of girder ends for G3

LIVE LOAD TEST

A live load test on the monitored segment was conducted to assess the continuity detail's performance under truck loads. Two dump trucks weighing 54.1 and 57.0 kips were used to load the bridge in 9 static loading cases. The trucks were positioned in tandem for 6 side-by-side midspan positive moment positions (P1 through P6) and 3 truck train negative moment positions at Bent 24 (N1 through N3) as can be seen in Fig. 10. Details of the truck positions, which were determined using finite element analyses of a full bridge model, can be found elsewhere [11]. Due to the slow nature of the installed long-term monitoring system, the rate of data recording was 24 readings per hour, which were then averaged and stored for retrieval. This rate translated into one reading every 2.5 minutes. To ensure that more than one data point was collected for each static load position, the trucks were asked to stay at each position for a period equal to or longer than 11 minutes. During this period, at least three and probably four readings were recorded for each loading case. In between loading positions (P1 and P2, P3 and P4, P6 and N1, N1 and N2, and N2 and N3), the trucks were driven of the tested bridge segment to help determine a reference datum for the recorded readings. Fig. 11 shows the actual trucks in two of the static loading positions, namely P1 and N1.



(a) Positions for positive moment at midspans



(b) Positions for negative moment at Bent 24

Fig. 10 Load test truck positions (distance with reference to middle of rear drive axle)

(b) Position P1 (c) Position N1

Fig. 11 Loading trucks in position for two of the nine static load positions

As can be expected, not all sensors were highly strained by all loading positions because of their locations with respect to the position of the trucks. Therefore, a minimum threshold strain was set to discard readings that were too small to be considered reliable. The results were compared to analytical results from the Global FE model. Fig. 12 shows a plot of the vertical displacement contours for the P5 load case. P5 is a case that causes positive moments in Span 24 (middle span). The two trucks were positioned over Girder G1, G2, and G3. As a result, downward deformations at this location could be seen in Fig. 12. At the same time, upward movement of girders in adjacent spans (23 and 25) took place.

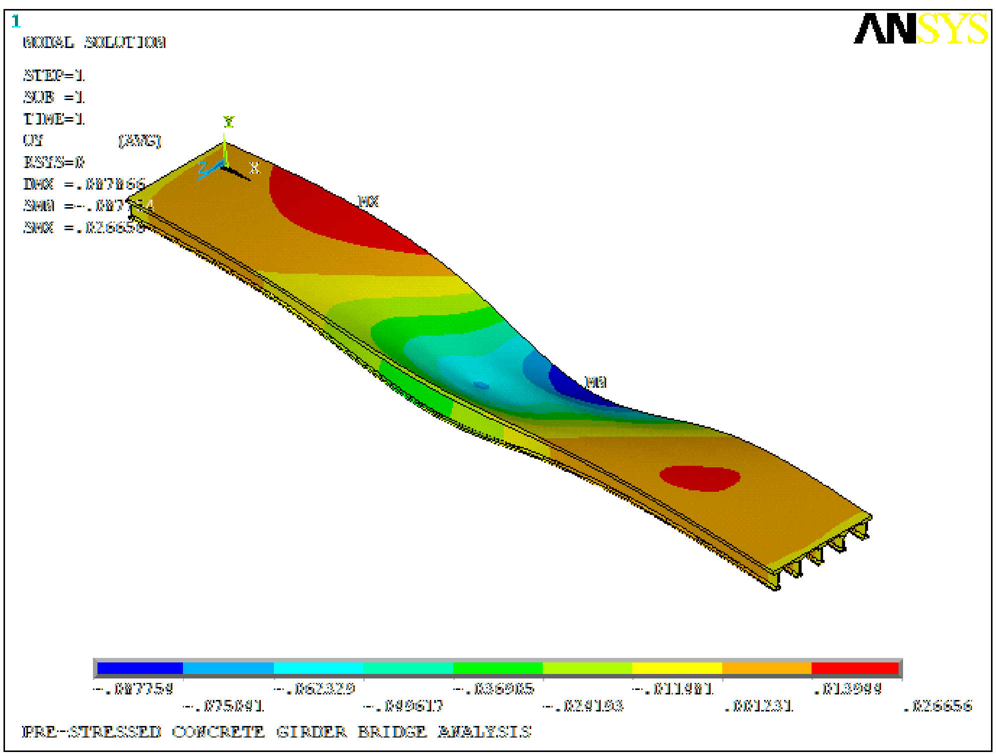
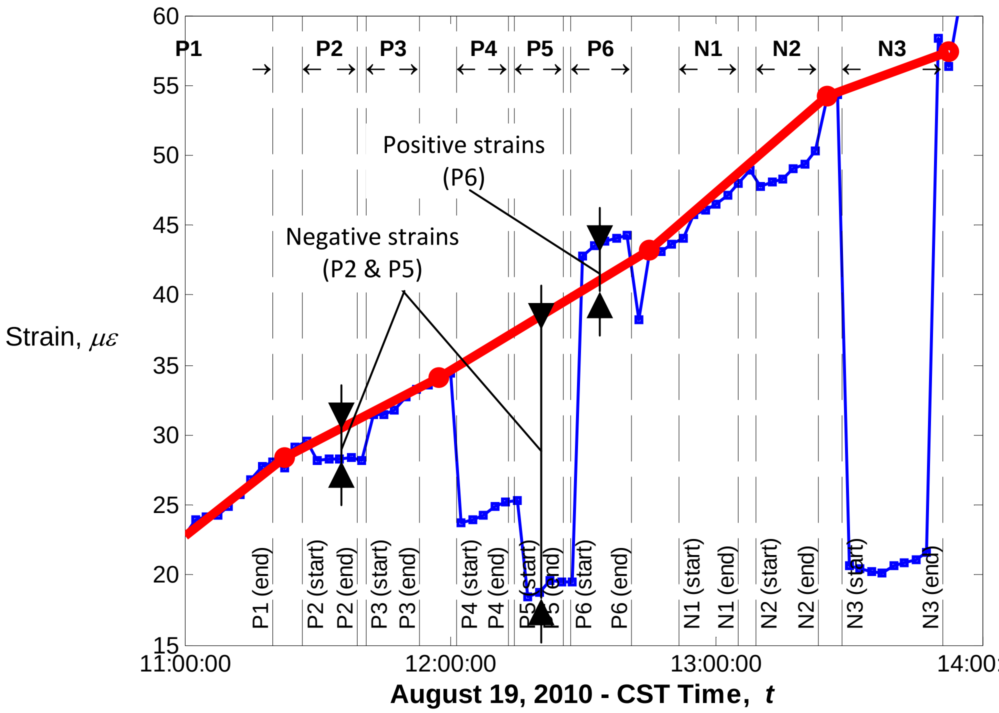
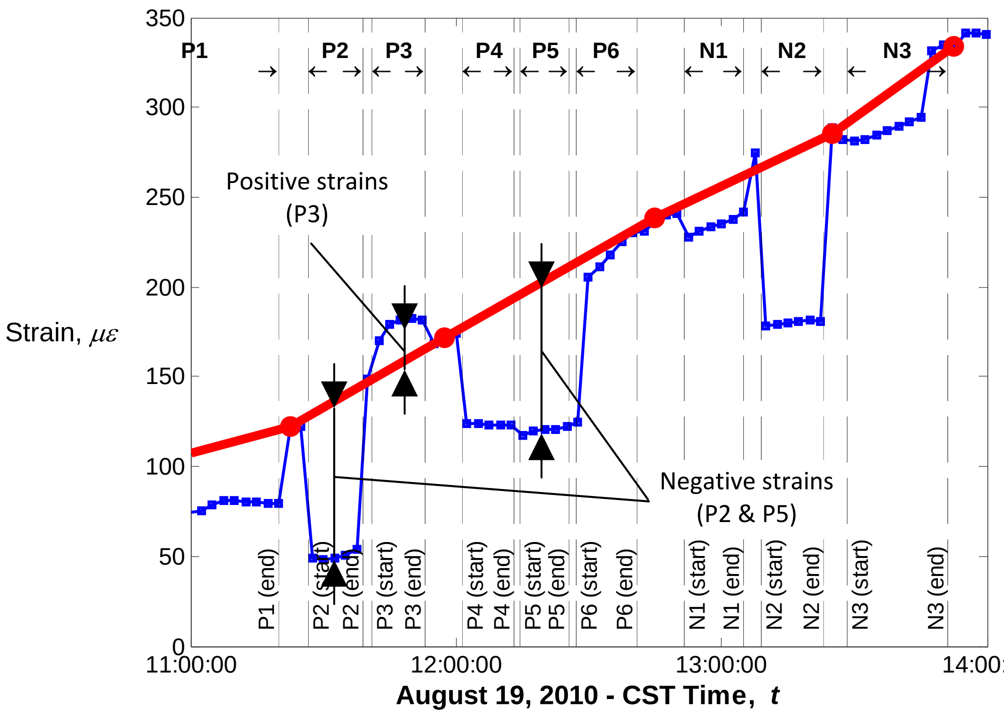


Fig. 12 Contours of vertical displacement (Case P5)

One interesting observation was that the hairpin strain records confirmed that forces were transferred from Span 25 (loaded span) to Span 23 (monitored span) for load positions P3 and P6. These two cases caused positive moments on the monitored continuity detail. Fig. 13 shows the magnitude of the strains, which are larger for Girder G3 than G1. A similar observation was related to the recorded negative strains for load positions P2 and P5. Both were load cases that target positive moment in the middle of Span 24. Therefore, trucks were positioned over Span 24. The trucks would, however, apply negative moments on the continuity diaphragm if it performed as intended. In both load positions (P2 and P5), the hairpin bars experienced negative strains that were, as expected, higher than the positive strains discussed earlier. Like the P3 and P6 load cases, the sensors were in one span and the loads acted on another for these two cases. Hence, the ability of the new detail to transfer forces between spans was confirmed. It should be noted that results from Girders G1 and G3 were only presented since Girder G5 was not instrumented on the hairpin bars in Span 23 and, therefore, a plot was not provided.



Girder G1 – Span 23 –Sensor #3



Girder G3 – Span 23 –Sensor #51

Fig. 13 Strains in hairpin bars showing positive values for faraway load cases

CONCLUSIONS

Based on the presented results, it can be concluded that:

- Positive moments develop in bridges employing the new continuity detail. They are caused by long-term effects such as girder creep and thermal variations.
- The continuity detail has the ability to transfer forces from one girder end to the adjacent girder end across the continuity diaphragm as evidenced by the recorded data under long-term effects as well as live load effects.
- Seasonal and daily temperature variations can cause large restraint moments in the bridge, especially temperature gradients. The level of restraint moment due to the combined seasonal and daily temperature is probably the most important factor in the design of this detail, since the designer has no influence on the temperatures at the bridge site. The other positive-moment causing factor, i.e., girder creep caused by prestressing forces, can be greatly reduced by not introducing continuity until a large portion of the creep takes place prior to pouring the diaphragm.
- Positive restraint moment can cause cracking in the diaphragm and/or girder ends if the total effect of positive moment causing factors are not considered in the design; i.e. creep in addition to thermal gradient. Girder cracking may have adverse effects on the durability and on the shear capacity of the girders. Therefore, special care should be given to the level of positive restraint moment during design. The authors are of the opinion that temperature gradient effects need to be considered in the design regardless of girder's age at establishment of continuity.

- The live load test revealed that the continuity detail transferred negative and positive moments across the diaphragm. The strains from the live load test were much lower compared to other long-term effects. Even if the actual design load were to be applied (approximately twice the test live load), the strains would still be small. Therefore, the live load case should be considered in the design; however, it is not the most demanding action on the detail.
- The monitored segment was skewed. Skewed configurations cause additional straining actions that do not develop in non-skewed bridge configurations. Therefore, the skew effect may have exacerbated the straining actions on the continuity detail. However, this hypothesis will need to be explored further through analytical or field investigations before it is confirmed.

Discussions with the precaster and the contractor revealed that construction cost of the detail is not substantial. Nevertheless, the precaster would rather build girders without the detail. The contractor’s critique of the new detail was stronger. The contractor is of the opinion that the continuity diaphragm, especially for skewed bridge configurations, is cumbersome and adds to the construction time mainly because of the diaphragm’s formwork. Simpler details that require less formwork would expedite the construction of slab-on-girder bridges.

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