Performance of an Accelerated Constructed Bridge: The 24th Street Bridge Council Bluffs, Iowa:

1. INTRODUCTION

Construction of bridges that last longer, are less expensive, and take less time to construct are becoming a larger part of bridge design. Overall, the movement toward bridges with increased cost-effectiveness are resulting in designs that increase durability, are easier to construct, and have minimal disruptions to the traveling public. Although there may be many ways to achieve this design, currently, one of the most commonly discussed ideas includes using precast segmental construction. This type of construction has the advantage that the individual components are manufactured off-site where increased quality is usually achieved. Further, because much of the work is completed away from the bridge site, it is anticipated that user disruptions would be minimized since the amount of labor intensive on-site work would be reduced which leads to reduced on-site construction time.

Using accelerated construction methods the 24th Street Bridge, a prestressed-precast bridge, was constructed the by Iowa Department of Transportation (IADOT). The design involved the use of precast deck components that were grouted compositely with the steel girders. This application of precast deck panels represents a step forward from previous systems in Iowa and strives to further improve the design concept. The first accelerated precast deck bridge in the state of Iowa was built in 2006. The development of the 24th street bridge deck panels evolved from the knowledge learned during the design, fabrication and construction of the 2006 bridge.

This paper presents the results of the 24th Street Bridge field testing, which monitors component behavior during construction and the performance of the completed bridge. The successful implementation of the 24th Street Bridge project has far reaching implications across the nation as it will allow for continuation of developmental work initiated through previous Innovative Bridge Research and Construction (IBRC) (AbuHawash et al, 2007) program projects.

2. BRIDGE DESCRIPTION

The 24th St. Bridge was constructed in two phases at an existing bridge site. The west lanes were constructed during Phase I, which allowed the existing structure to be used for traffic. Upon completion of Phase I traffic was moved to the Phase I bridge and the existing bridge removed in order to construct Phase II. After both phases were complete a closure pour was cast to connect the bridges. The completed bridge carries six lanes of 24th street traffic (four through lanes and 2 turning lanes) over Interstate 80 in Council Bluffs, IA. The bridge is two spans with a total length of 353 ft-6 in. from center-line of abutment bearings. The south and north spans are 178 ft-6 in. and 175 ft-0 in. respectively. The girders are continuous over the center support and simply supported at the abutments. The abutments consist of one row of vertical piling and one row of battered piling with a concrete cap and backwall. The spill through slope is terraced with concrete retaining walls down to the interstate level. The center pier has two rows of vertical piling support with a continuous concrete pier cap.

The superstructure of the 24th St. Bridge is comprised of 70 pre-stressed precast deck panels supported and composite with high performance steel plate girders. The completed bridge has 12 lines of girders spaced at 9 ft-0 in. on center. The girders have a depth of approximately 60 in. The flange thickness varies from 1 in. near the abutments to 2 in. at the pier. Each girder has two bolted field splices with a maximum girder length between splices of 121.75 ft. Figure 1. shows the pier and girders during Phase I construction.

Figure 1. Bridge Pier and Girders during Phase I construction

The precast deck panels were designed to act compositely with the girders, requiring the deck be field connected to the girders through the use of grouted shear connectors. The deck panels are 10 ft long x 52 ft-4 in. wide x 8 in. thick. A schematic of the deck is shown in Figure 2. Each panel has 28 1 in. by 3 in. embedded ducts to house four longitudinal post-tensioning strands which run the full length of the bridge. Each panel is transversely prestressed with 10 1/2 in. dia. strands. The deck to girder composite action was obtained with the use of 1 ft 4 in. by 11 in. shear stud pockets longitudinally spaced 2 ft on-center. Shear studs were welded on the girder within the pockets of the panels; the pockets were then filled with grout that was also used to fill the haunch between the girder and deck.

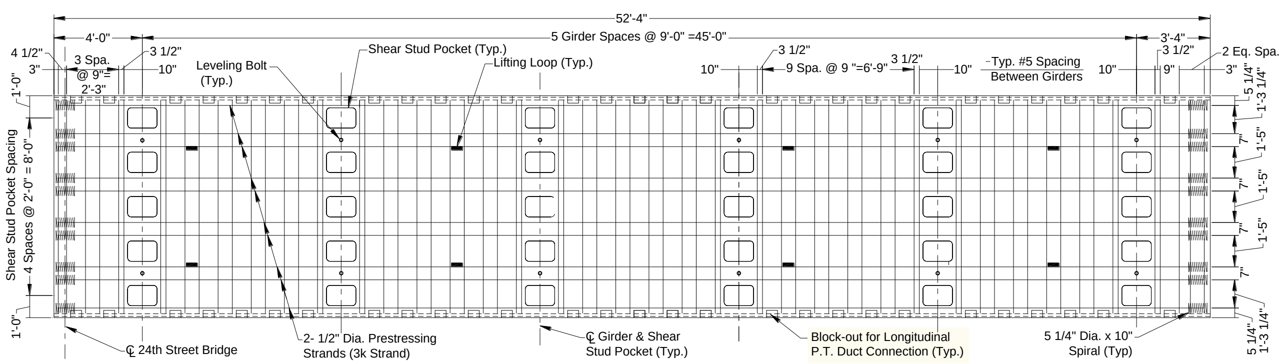

Figure 2. Precast deck panel configuration

3. FIELD TESTING

3.1. Strand Corrosion Monitoring

Corrosion monitoring of both pre-stress and post-tensioned strands was completed with the use of Vetek V2000 corrosion monitoring system. Six transverse pre-stressed strands were instrumented with the corrosion monitoring systems during panel fabrication. The post-tensioning strands were also monitored by placing three sacrificial ducts with two non-stressed strands in two conjoining end panels near the north abutment. The actual strand could not be monitored due to the congestion within the post-tensioning duct.

The V2000 monitor works by measuring the electric potential between the strand and the electrode with the pore water of the concrete acting as an electrolyte between the two. An increase in the electric potential indicates that corrosion activity is taking place. Both the pre-stress and post-tensioning strands selected for monitoring were instrumented their full length. Electrode readings for both strands were taken during their respective placement in 2008. Strands were again read in 2009. All electrode readings indicated that no corrosion is taking place in the strands.

3.2. Deck Panel Behavior during Handling

In order to better understand the behavior of the deck panels two deck panels were monitored during their placement in the field. Each of the two deck panels were instrumented with 16 BDI strain gauges. The strain gauge locations and labels (gridlines A, B, and C for Panel No. 1 and D, E, and F for Panel No. 2) are shown in Figure 3. The strain gauges were placed on top of the deck panels. Only half of the transverse length of the panels was instrumented. The strain gauges are oriented on the deck panels with 12 of the gauges transverse to the directions of traffic and 4 longitudinal to the direction of traffic; these are denoted with an "L" after the gauge number. Grid lines 2 and 4 are pick point locations for picking up and moving the panels.

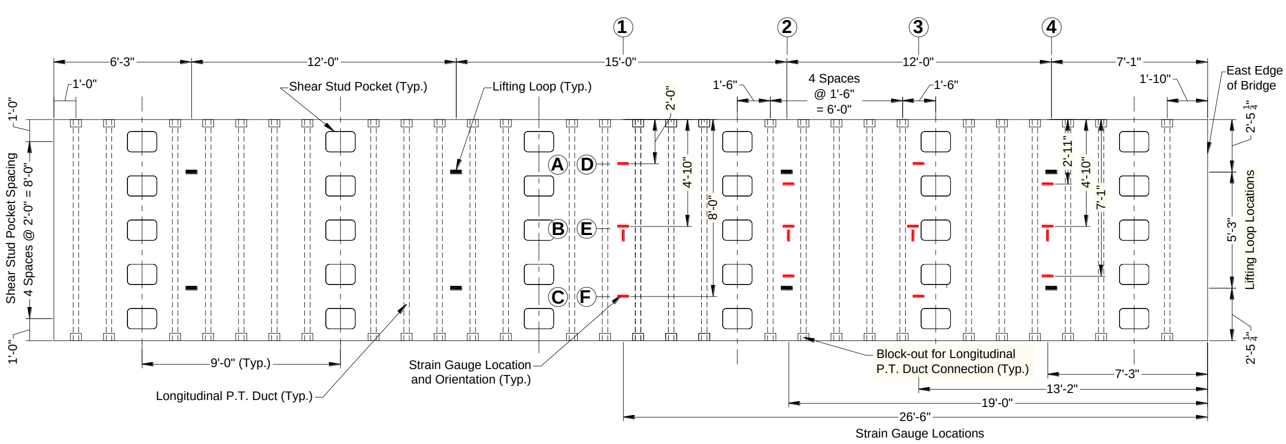

Figure 3. Strain gauge layout on precast deck panels

The deck panels were instrumented in the staging yard at the construction site. The panels were then lifted using 8 pick points. The panels were lifted with a crane and maneuvered into place as shown in Figure 4. The monitored panels had final placement over the pier during the phase II construction. Each panel took about 20 minutes (1200 seconds) to place. The strain gauges continuously recorded data during the 20 minutes

Figure 4. Final maneuvering and placement of precast deck panel placement

Strain profiles along gridlines A, B, C, D, E, and F are shown in Figure 5.when the strain of the panel is approximately maximum. At location zero of Panel No. 1 (i.e., A, B, and C), which is gridline 1 and center line of the panel, the strain is at its maximum of approximately 68 $\mu\varepsilon$. In general, the strain decreases as it moves away from the center of the panels. The strain at and between the pick points have very little consistency between gridlines. The strain profile for Panel No. 2 (i.e. D, E, and F) starts at approximately 170 $\mu\varepsilon$ then increases to the pick point at gridline 2. The maximum strain of 230 $\mu\varepsilon$ occurred at location F2. The strain then decreases to gridline to 4.

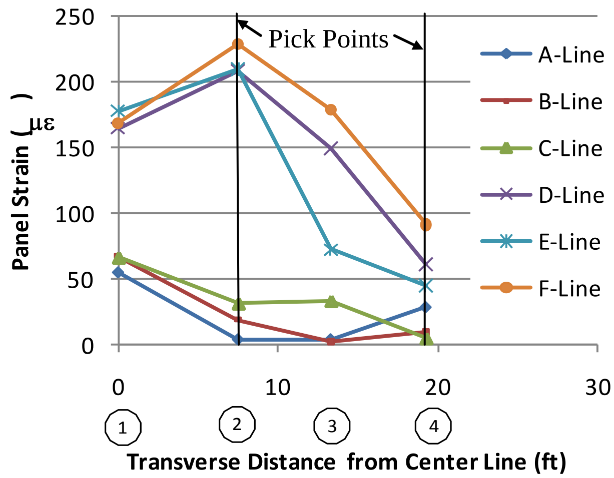

Figure 5. Strain profile along gridlines Panel No. 1 and 2

3.5. Diagnostic Live Load Testing

A diagnostic live load test was conducted on the completed bridge to compare structural performance with expected design performance. Strain gauges and deflection transducers were installed on critical superstructure members. Bridge Diagnostic Inc. (BDI) strain gauges were placed at the pier, mid-span, and abutment of the north span as shown in Figure 6. The gauges had three different configurations as illustrated in Figure 7 and depicted by the number next to the gauge location in Figure 3.4. Deflection transducers were placed on girders F through L at the mid-span of the north span only. The deflections transducers were placed on the bottom of the girder similar to configuration 1.

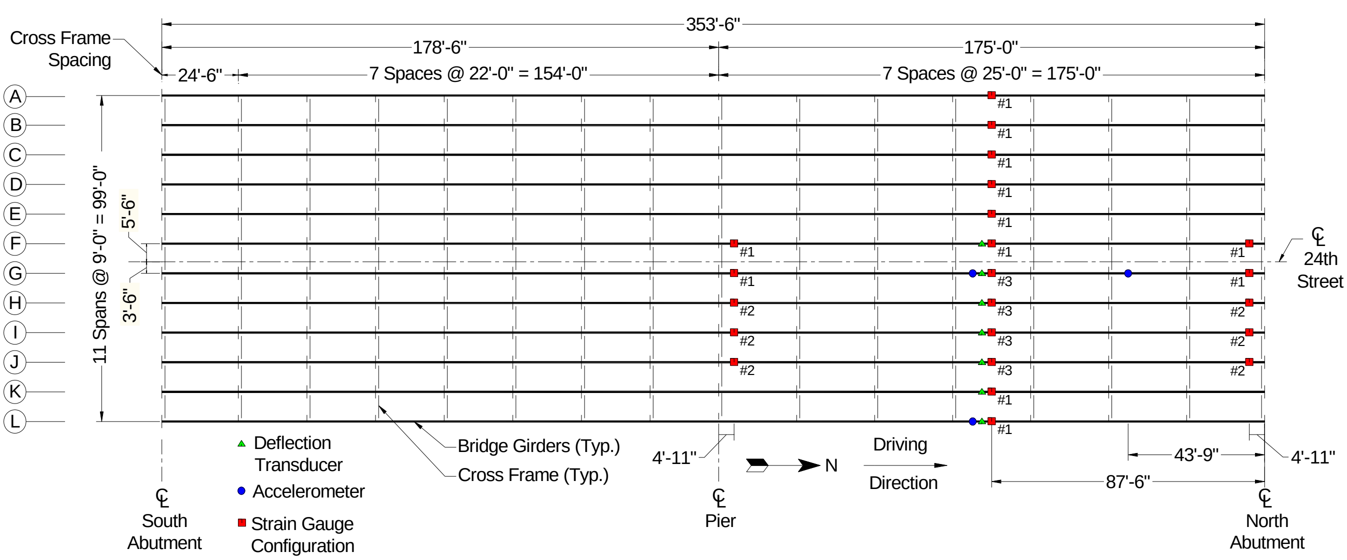

Figure 6. Gauge location on bridge

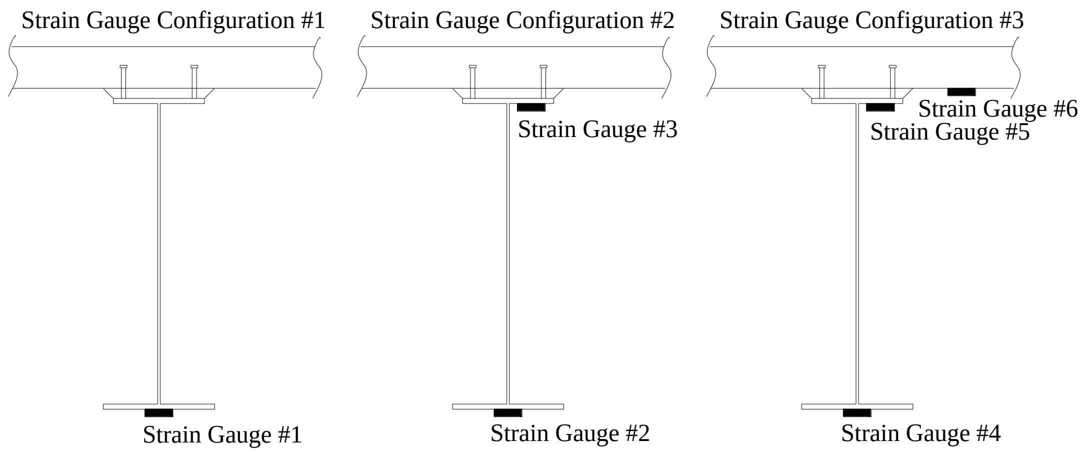

Figure 7. Cross-sectional strain gauge configurations

Field testing took place on November 12, 2008. The testing consisted of point in time live load testing with a fully loaded three axle dump truck that was driven over the bridge at slow speeds. The transverse position of the truck varied with six load cases (LC) 1 through LC 6, as shown in Figure 8. The loaded dump truck used for testing had a gross weight of 49 220 lb. The axle configurations and weights can be seen in Figure 9. The testing was conducted at night with only the loaded lane being closed to ambient traffic. Although tests were conducted when ambient traffic was low, there were load cases with heavy trucks driving over the bridge.

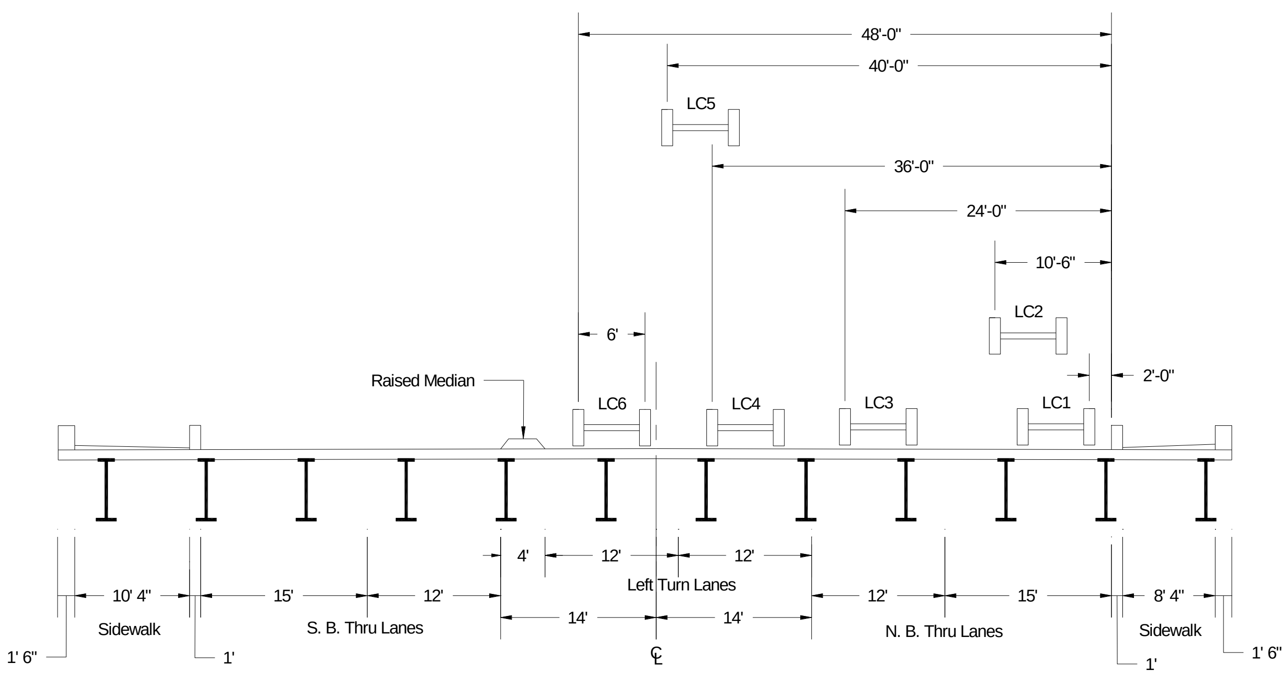

Figure 8. Truck load paths (the truck traveled north into the page)

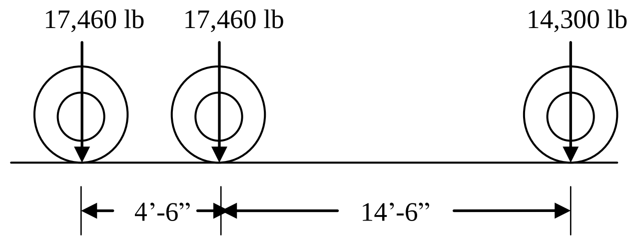

Figure 9. 2008 truck axle loads

3.5.1. Girder Deflection

Deflection transducers were installed on the bottom flange of the seven east girders. Representative time-history deflections for load case 2 at the north span mid-span with respect to front axle position are shown in Figure 10. The load truck was located in the center of the east driving lane for load case 2. In general all load cases produced the same deflection shape for the bridge with positive deflection when the truck was on the south span and negative deflection when the truck was on the north span. For all the load cases a deflection range of -0.32 in. to +0.21 in. was seen when the truck was on the south span and a range of -0.41 in. to +0.23 in. when the truck was on the north span for the seven girders. The maximum north span deflection of -0.41 in. corresponds to a span to deflection ratio of L/5120.

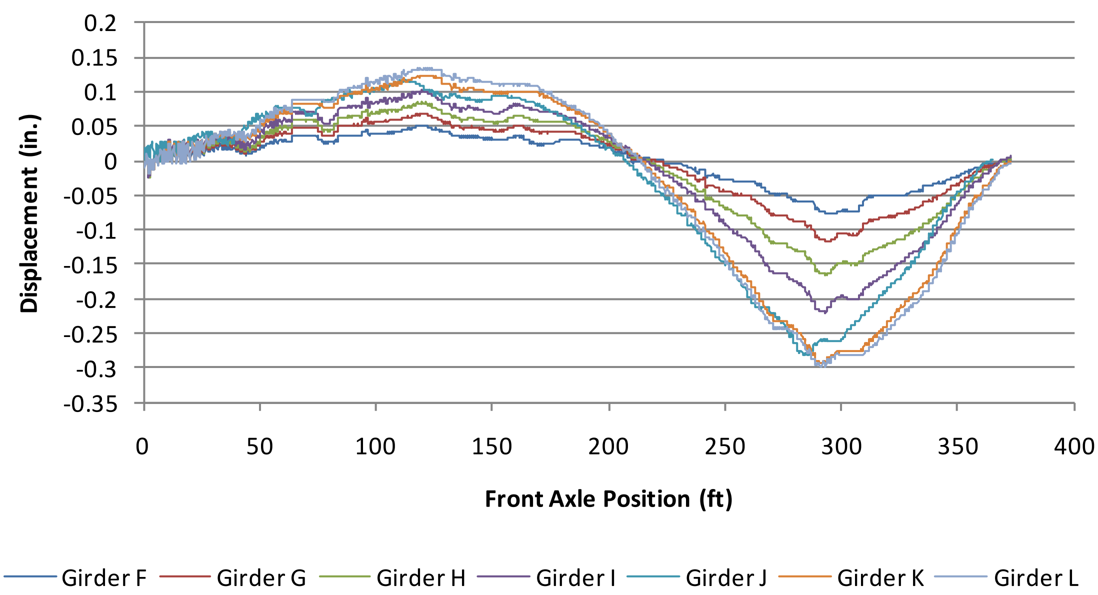

Figure 10. Representative time history deflections for Load Case 2

3.5.2. Girder Strain

A total of 36 strain gauges were placed on the bridge with 20 located at mid-span and 8 located at each the pier and north abutment. The full range of strain measured for all gauges can be seen in Table 1. The maximum strain was +66 με and occurred at girder L during LC1. The largest strain range was at the mid-span-bottom flange location with a range of 88 με during LC1.

Table 1. Range of strain at various locations

Location	Strain (με)		
	Bottom Flange	Top Flange	Bottom of Slab
Abutment	-4 to +14	-5 to +6	NS
Pier	-16 to +3	-1 to + 5	NS
Mid-span	-22 to +66	-5 to + 5	-2 to + 6

NS—No strain gauge at location

3.5.3. Closure Pour Movement

The close pour movement was determined by investigating the deflections of LC5 and LC6. The deflections used to monitor the closure pour are located on the bottom of Girder F and G, which are on each side of the closure pour. The differential movement for both load cases was less than 0.04 in. The load distribution of the girders during the load test could cause some differential reading to occur; therefore the actual movement at the joint would be less than the 0.04 in. The small movement in conjunction with the load transfer described herein indicated the closure pour was working properly.

3.5.4. Bridge Load Fraction and Load Distribution

Load fraction was calculated for each load case based on the assumption that the girders are of equal stiffness. The approximate load fraction for each girder can, therefore, be obtained with the following equation:

Where LF_i = load fraction of the ith girder, ε_i = strain of the ith girder, $\Sigma\varepsilon_i$ = sum of all girder strain, and n = number of girders.

Figure 11 shows the load fraction for the six load cases tested. Girder K was seen to have the largest load fraction of 0.25 during LC 1. The girder with the lowest load fraction, near zero, was Girder A, which was furthest away for the load truck. The load fraction was generally largest at the girders directly below the truck. The load fraction generally decreased as the transverse distance from the girder to the load truck increased. During LC 5, however, the load fraction was affected by ambient traffic on the west side of the bridge causing the load fraction to be near horizontal on the west side of the bridge.

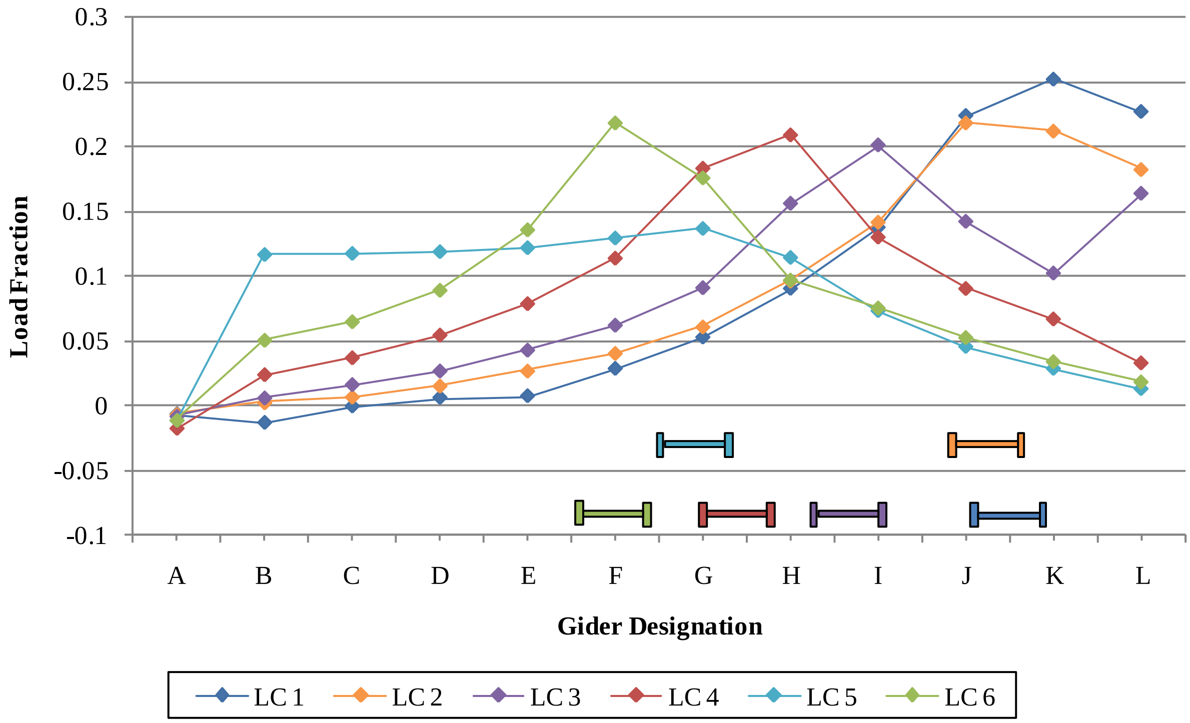

Figure 11. Experimentally obtained load fraction for six load cases

Load distribution was determined experimentally by adding the load fractions of complementing load cases. Combining LC2, LC3, LC4, and LC6 with the transpose of LC2 and LC3 (designated at LC2' and LC3') provided proper lane loads for determining load distribution. Figure 12. shows the experimental load distributions along with the code distribution factors from AASHTO LRFD 2004. The largest obtained load distribution factor was obtained at Girder H with a value of 0.63. The lowest girder distribution factor was 0.30, which was located at Girder A. Girder A and L, however, are located below the side walk areas therefore Girder B and K may be more representative of the load distribution at the edge girders. The lowest distribution at Girder B and K was 0.42. When comparing the experimental test results with AASHTO LRFD load distributions, the experimental results were less than code distributions.

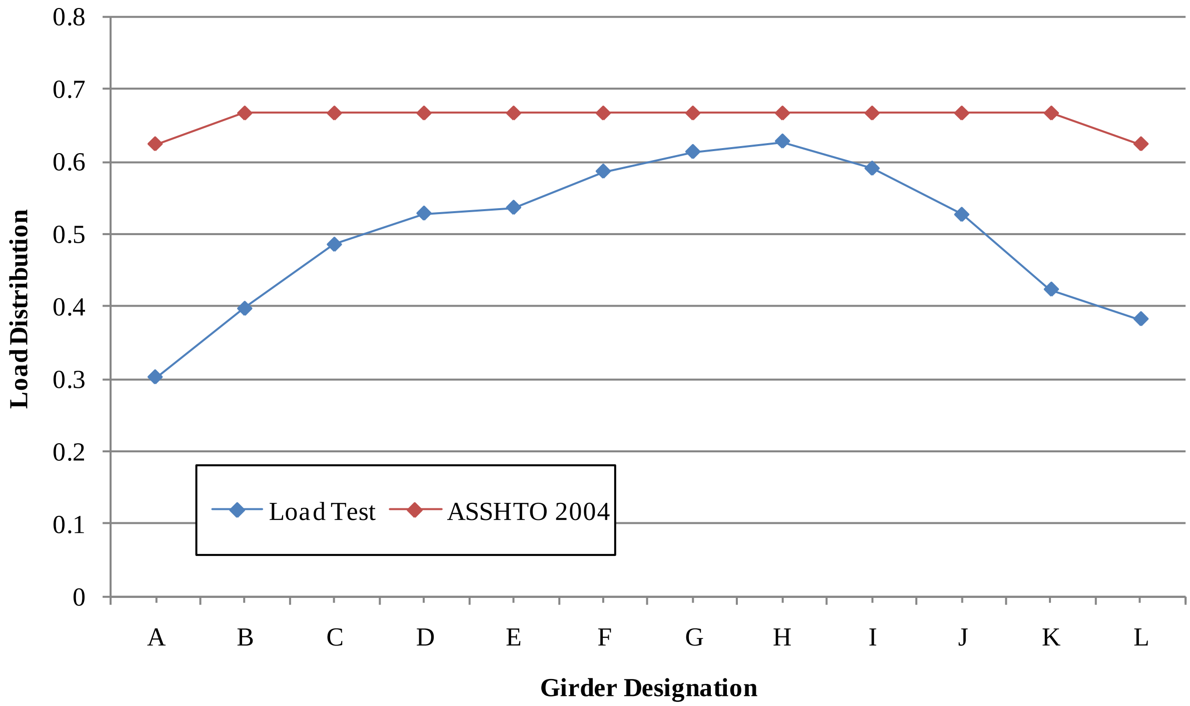

Figure 12. Experimental and codified load distributions

3.5.5. Evaluation of Neutral Axis Location

The girders with strain configuration #3 were used to estimate the location of the neutral axis during testing. Figure 13 shows the strain profile for girder G through J for LC2.The steel girder total depth was 61 in. with 1 in. flanges top and bottom. A girder haunch existed between the precast panel and girder of between ½ in. to 1 in. The location of the neutral axis during testing was approximately 58 in. to 61 in. from the bottom of the girder. The neutral axis location appears to be located near the top of the steel girder but not within the grouted haunch region or in the precast slab. In addition, the top flange strain and the strain on the bottom of the slab are very similar indicating composite action between the girder and slab is taking place.

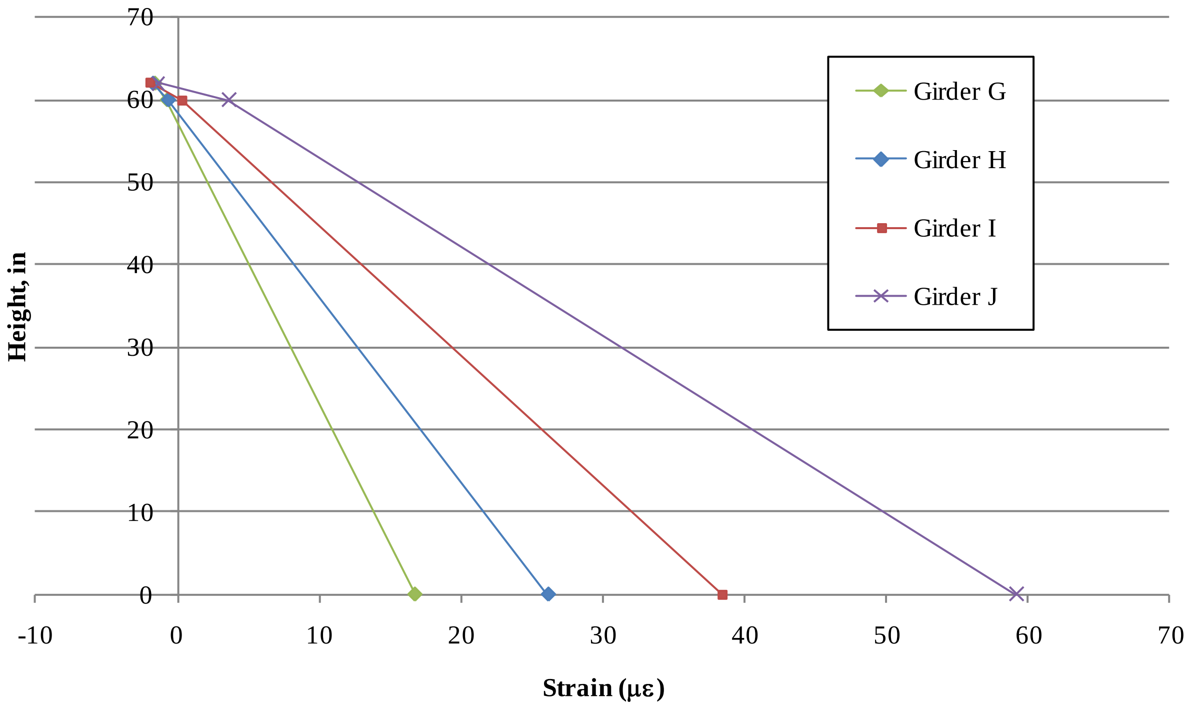

Figure 13. Strain Depth Profile for Girders G through J, Load Case 2

4. SUMMARY AND CONCLUSIONS

Using accelerated construction methods, a prestressed precast bridge, was constructed the by Iowa Department of Transportation (IADOT). The design concept involved the use of precast deck components that were grouted compositely with the steel girders. The successful implementation of this approach has far reaching implications in Iowa as well as nationwide, as there are many instances where proven rapid construction techniques could result in significant reductions in costs.

The overall objective of this project was to asses bridge components and evaluate the overall performance of the 24th Street Bridge. In order to complete the objective the project included monitoring strand corrosion, deck panel behavior during placement, and a diagnostic live load test.

The following conclusions were obtained during the testing:

1. All corrosion electrodes indicate that no corrosion is taking place in the strands.
2. During deck panel placement the peak strain in the panel was 230 με located at F2, which was near a pick point location.
3. The maximum north span deflection was observed to be -0.41 in., which corresponds to a span to deflection ration of L/5120.

4. The maximum strain of +66 $\mu\varepsilon$ occurred at the midspan of girder L during LC1. The maximum strain range of 88 $\mu\varepsilon$ occurred at the same location and load case.
5. The maximum strain range at the pier and abutment bottom flange locations were 19 $\mu\varepsilon$ and 18 $\mu\varepsilon$ respectively.
6. The closure pour differential movement was less than 0.04 in.
7. A maximum load fraction of 0.25 was seen at girder K during LC1.
8. The largest load distribution factor was found at girder H with a value of 0.63. The lowest load distribution factor of 0.30 occurred at girder A.
9. The load distribution factors obtained from field testing were within AASHTO LRFD load distribution requirements.
10. The neutral axis lies near the top of the steel girder but not in the haunch or deck slab.

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