STATIC TRUCK LOAD TESTING OF AN ADJACENT BOX BEAM BRIDGE CONTAINING ULTRA HIGH PERFORMANCE CONCRETE (UHPC) LONGITUDINAL JOINTS

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ABSTRACT

Adjacent precast prestressed box beam bridges have often been preferred for medium and short span bridges for economy and speed of construction. However, adjacent box beam bridges may exhibit longitudinal cracks in the shear keys which can propagate into overlays due to poor connections between adjacent beams. In this research, partial depth shear keys were grouted with ultra-high performance concrete (UHPC) and contained equally spaced dowel bars for a bridge constructed during the summer of 2014 in Fayette County, Ohio. The bridge, which is the first adjacent box beam bridge in the USA containing UHPC shear keys and transverse dowel bars, was instrumented and monitored. The bridge did not have transverse post-tensioning, a composite deck, or transverse tie bars. This paper presents the results of static truck load testing performed just prior to the bridge opening to traffic. Finite element modeling was also performed using ABAQUS/CAE. The model was calibrated with the experimental data and showed the ability of the finite element modeling to predict the behavior of the bridge. The bridge behaved as a unit, which emphasized the ability of the new shear key configuration with UHPC and dowel bars to satisfactory perform load transfer.

Keywords: Adjacent Box-Beams, UHPC, Truck Test, Shear Key, Dowel Bars, Finite Element Modeling

INTRODUCTION

Adjacent precast prestressed box beam bridges have often been preferred for medium and short span bridges.. The box beam bridge is constructed by placing box beams side by side and grouting the longitudinal, partial or full depth shear key joint with non-shrink grout. Bonded or unbonded transverse tie rods are used to pull beams together during construction. In addition, post tensioning bars or strands can be used to help in load transfer between beams. A composite bridge can be created with a reinforced concrete deck on top of the boxes and with sufficient connection to the beams. This also aids in load transfer between beams. However, the issue with these bridges is longitudinal cracking of the shear keys and reflective cracking in the overlays which usually leads to leakage and accelerated corrosion of the reinforcement. Load transfer between beams may also be reduced or lost (Russell [1]). Repair of cracked and leaking longitudinal shear keys is expensive.

Much research has been done to determine the causes of the longitudinal cracks in the shear keys between adjacent box beams. Huckelbridge et al.[2] tested two bridges in Ohio. One of these bridges was a two lane simply supported, non-composite box beam bridge. The second bridge was a four lane, four span, non-composite box beam bridge. The results showed that the first bridge exhibited a 0.02 in. relative displacement in the joint adjacent to the center girder, which indicated leaking of the joint. However, the joint still transferred the shear load to the adjacent beam. The second bridge also exhibited a relative displacement, especially for the joint adjacent to the wheel load position. New shear keys were cast during repair of the bridge. The joint that exhibited the largest relative displacement before repair also had the largest relative displacement after repair. The authors concluded that the intact shear key should have relative displacement less than 0.001 in., otherwise the shear key fractured along at least part of the length. The observed relative displacement in the joint was between 0.003 in. to 0.02 in. and indicated partial fracture. There was no instrumentation in the shear key to measure the strain that caused the cracks.

Some researchers have shown that the cracks in the longitudinal joints are caused due to thermal stresses and propagate due to applied load. Miller et al. [3] studied shear key configurations and grout materials in four full scale side by side box beams which represented part of a bridge. A top shear key was tested either with non-shrink or epoxy grout and a mid-depth shear key was tested with non-shrink grout. The top depth shear key with non-shrink grout was cast in November and just minor cracks (shrinkage cracks) were observed after inspection. The test was done in January, and due to cold, snowy weather, the shear key cracked before applying any load. The bridge was tested under cyclic loading, and the cracks from the temperature propagated without any new cracks forming from loading. The top shear key was also tested in May. The shear key cracked after one week, and temperature was found to be the main cause of the cracking. No cracks were observed in the epoxy grout under temperature effect or loads. However, epoxy is undesirable due to the high difference in thermal expansion between epoxy and concrete. The mid-depth shear key was constructed with non-shrink grout. The cracking was reduced because the throat was not grouted which helped to reduce temperature stresses. The authors suggested using post-tensioning to stop joint movement, mid-depth shear keys with non-grouted throat, or full depth shear keys.

Another study was done by Grace et al.[4] that investigated the behavior of an adjacent box beam bridge due to the combination of thermal and traffic load. Field observations, experimental testing, and numerical modeling were used in the research. The results showed that cracking was unlikely to occur due to the traffic load when transverse post-tensioning was used. However, when the bridge was subjected to a positive temperature gradient, cracks were observed. In addition, cracks from temperature propagated due to traffic loads. The thermal effect was studied numerically and utilized the temperature recorded from the field.

Recently, Attanayake and Aktan [5] inspected in service Michigan bridges to identify the changes in the design of side by side box beam bridges. The design changes were full depth grouted shear keys, high level of transverse post-tensioning, and a cast in place deck with seven day moist curing. However, the Michigan box beam bridges still experienced longitudinal reflective cracks irrespective of age. While under construction the inspected bridge showed that cracks developed at the grout-beam interface within a couple of days after grouting. The cracks developed in the shear key three days after being grouted and before applying post-tensioning. The shear key still experienced cracks even after applying post-tensioning. When the bridge was inspected after three weeks, before the deck placement, cracks were observed in every shear key along the beam shear key interface. The reflective cracks were observed within the first 15 days after casting the deck and before the deck was subjected to barrier and live loads. The observed crack was above the supports (abutments) and developed from top to the bottom through the deck thickness over the shear keys. The authors concluded that redesign should be employed to consider the effect of live load and intrinsic loads.

A grout material with high mechanical and bond strength along with superior durability could be used in the joint to eliminate the longitudinal cracks in shear keys of box beam bridges. Ultra-high performance concrete (UHPC) represents a new class of concrete, which has high strength and durability characteristics. UHPC has been defined as a cementitious composite material composed of an optimized gradation of granular constituents, a water-to-cementitious material ratio less than 0.25, and a high percentage of discontinuous internal fiber reinforcement. The mechanical properties of UHPC include compressive strength greater than 21.7 ksi and sustained post-cracking tensile strength greater than 0.72 ksi. UHPC has a discontinuous pore structure that reduces liquid ingress, significantly enhancing durability compared to conventional concrete (Yuan and Graybeal[6]). The superior mechanical properties and durability of UHPC has led it to be used as a grout material for other types of bridge connections. Perry et al.[7], documented the UHPC in the longitudinal and transverse joints of the 3-span side-by-side Eagle River Bridge in Canada. The UHPC was placed successfully and the required strength was provided after the curing period. However, there was no instrumentation used to provide data on the bridge's performance.

Because of the superior properties of UHPC, it has been suggested that it be used as grout material in the shear keys with dowel bars of box beam bridges. Yuan and Graybeal[8] studied two full-scale adjacent box beams to evaluate four connection details. The first configuration included a partial or full depth shear key grouted with non-shrink grout in combination with transverse post-tensioning. The second configuration included a partial or full depth shear key grouted with UHPC and dowel bars. Cyclic loading was applied and the results showed that the shear key did not crack and the non-shrink grout and the UHPC resulted in the same behavior. However, the cyclic loading did propagate a pre-existing crack in the conventional grout connection independent of the level of post-tensioning. The UHPC connection exhibited superior performance even when direct tension was applied on the top to create cracks. Cracks were observed in the box beams, but no cracks developed in the shear keys. The UHPC shear key with dowel bars was tested in a lab environment and only included two adjacent box beams. Therefore, a full scale bridge in a field environment utilizing the UHPC shear key with dowel bars would further help understand the behavior of the new shear key design.

Steinberg et al.[9] investigated the behavior of a pair of box beams under live and temperature gradient loads by using finite element modeling. Partial and full depth shear key configurations with 4 in. dowel bar spacing were modeled to connect the pair of box beams. The partial depth shear key configuration model was calibrated by using experimental data collected from testing at the Turner Fairbank Highway Research Center (TFHRC). The full depth shear key was modeled by using the same parameters used with the partial shear key model because experimental data was not available. Maximum principal stresses in both shear keys and dowel bars were investigated considering temperature gradients and dowel bar spacing. The results showed that maximum principal stresses increased in the partial depth shear key after applying the temperature gradients. However, there was a decrease in the maximum principal stresses for the full depth shear key. In addition, shear keys exhibited larger principal tensile stresses as dowel bar spacing increased. However, the partial depth shear key exhibited less tensile stress than the full depth key. The principle stresses in the dowel bars increased when including temperature gradients for both partial and full depth shear keys. Both shear key configurations experienced differential deflection lower than 0.02. A partial depth shear key with dowel bar spacing of 12 in. was suggested by the authors.

In the present research, a full-scale bridge was used to monitor the field performance of the new shear key configuration with UHPC and dowel bars. The bridge was the first adjacent box beam bridge utilizing this shear key in the USA, and was constructed during the summer of 2014 in Fayette County, Ohio. The bridge was instrumented by using strain gages embedded in box beams, shear keys, and on dowel bars. In addition, exterior sensors were used at the bottom of the bridge at mid-span to monitor the short and long-term behavior of the bridge. This paper presents some of the truck load test and finite element modeling results.

BRIDGE DESCRIPTION

The bridge was constructed on Sollars Road in Fayette County, Ohio near the town of Washington Court House. The bridge consisted of seven box beams adjacent to one another. The bridge length was 61 ft. long and had a width of 28 ft. Each beam was 48 in. wide and 21 in. deep as shown in Figure 1. A total of 28 half inch diameter seven wire low relaxation strands with a 270 ksi ultimate strength were used in each box beam. The mild shear reinforcement had a 60 ksi yield strength. There was a 33 in. long diaphragm at each end of each box beam. The beams rested on two bearing pads at the end of each beam and were connected to the abutment with a vertical dowel bar. The bridge did not have transverse tie rods, which is common practice in Ohio. In addition, transverse post-tensioning or a composite deck, which can be used to help the bridge behave as a unit in the transverse direction and reduce the differential deflection between beams, were not utilized. Both of these methods to improve transverse behavior can add costs to the bridge and are not welcomed by smaller bridge owners such as counties.

The bridge utilized the new shear key design that was developed and tested at the Turner Fairbank Highway Research Center (TFHRC). The new design consisted of UHPC as the grout material with equally spaced dowel bars in each joint. The new shear key was larger than the typical shear key and used dowel bars, which were threaded into the beams and were staggered at a 4 in. spacing. The testing at TFHRC involved connecting two box beams together and applying a concentrated load. The results show that the new design was sufficient to make the bridge behave as a unit and no cracks were recorded in the shear keys even after numerous loading cycles. Finite Element Modeling of the laboratory testing, along with a parametric analysis were also performed and it was shown that larger spacing of the dowel bars could be used (Ubbing [10]). However, the design was already underway and field behavior can sometimes greatly differ from laboratory conditions. Therefore, a conservative decision was made to utilize the same dowel spacingas the TFHRC study. Each beam had shear keys on both faces except for the exterior beams, which only had one interior face shear key. The dowel bar system had two parts. The first part is embedded in the beam 18 in and contained a female threaded end. The part that is embedded in the shear key had a length of 4.75 in and a male threaded end allowing it to be screwed into the part embedded in the beam. Similar shear key designs have been used in bridges that are in service in Ontario, Canada. Figure 2 shows details of the shear key.

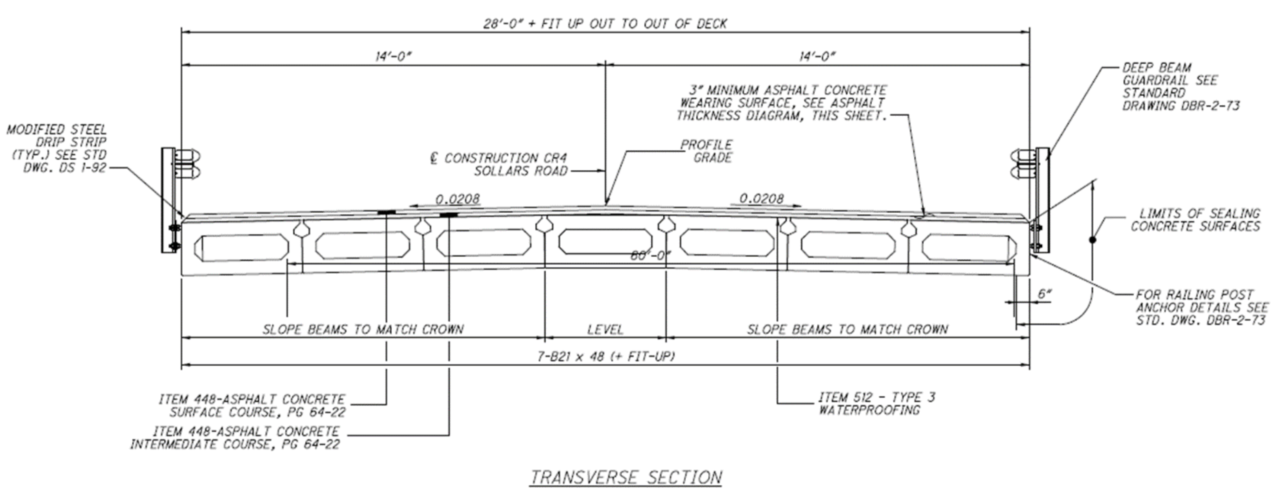

Fig.1 Bridge section

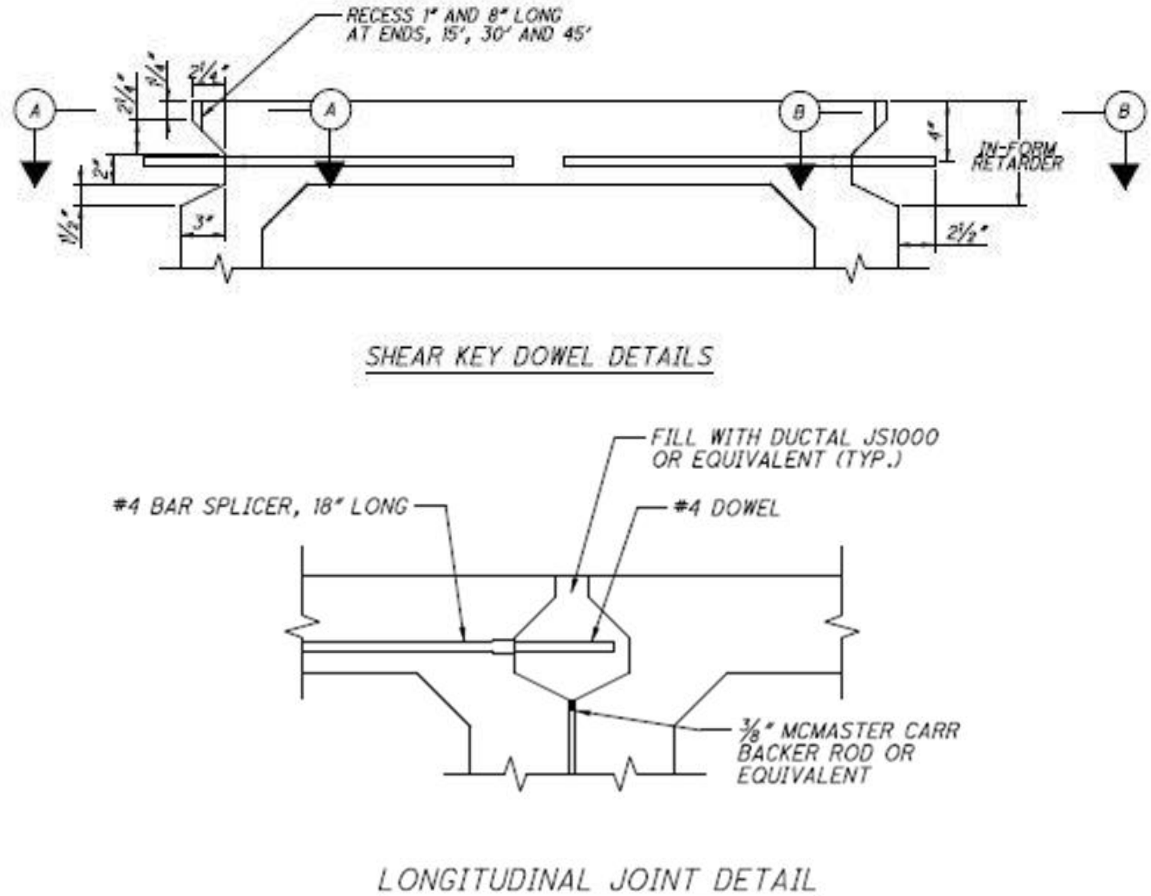

Fig. 2 Shear key detail

The box beams were fabricated in Kalamazoo, Michigan in May 2014 in a precast and prestressed concrete manufacturing facility. The typical box beam form was used, except the shear key shape was modified using wood. For exterior Beams 1 and 7, the shear key modification was used only on the inside face of the beams. For the remaining interior beams, the shear key form was used on both sides. The wood form for the new shear key can be seen in Figure. 3. The form was coated with a retarder and the female threaded ends of the dowel bar parts internal to the beam were placed on the red plastic tabs. Figure 4 shows the dowel bar parts internal to the beams. Figures 5 and 6 show the shear key upon removal from the box beam form and after power washing, respectively. The end result was a rough shear key surface with exposed aggregate that enhanced bond between the beams and UHPC.


Fig. 3 Shear key form


Fig. 4 Beam dowel parts in place


Fig. 5 Initial shear key


Fig. 6 Power washed shear key

INSTRUMENTATION

The first three box beams were instrumented with vibrating wire strain gages embedded in the beams and on the dowel bars. Thermocouples were installed in order to measure the temperature gradients. The first beam was cast on May 7, 2014. The following day, the prestressed strands were cut and the box beam was lifted from the form and loaded onto a truck. While the beam was on the truck, power washing with water was used to obtain an exposed aggregate surface on the shear key. The beam was then moved to the yard for the purpose of curing. The same procedure was used for the remaining beams.

One May 28, 2014, the contractor began to remove the old bridge and the new bridge's foundations and abutments were constructed. On Saturday July 12, 2014, the box beams were transported to the site. While the beam was on the truck, the dowel bars were screwed into the part of the dowel bar embedded in the beam. The beams were then moved from the truck and set on the abutments using a crane. Each end of each beam had bearing pads between the abutment and beam. In addition, one positioning vertical dowel bar at each end of each beam was placed through the beam into the abutment. The forward abutment vertical dowel bars allowed for expansion by using a joint sealer around the dowels. The rear abutment vertical dowel bars were grouted into place to create a fixed condition. For reference, the beams were numbered 1 to 7, from left to right while facing the forward abutment. Figure 7 provides a view of the shear key with dowels after beam placement.

Fig. 7 Shear key with dowels

On July 16, 2014, the joints were covered with plywood, except for larger openings at the quarter points along the shear key. On July, 17, the shear key joints were cast using ultra-high performance concrete (UHPC). Two mixers were used to properly mix the UHPC. The UHPC was moved to the joints in wheelbarrows and placed into chimneys made of plastic buckets located at the larger joint openings as shown in Figure 8. The UHPC flowed into the joints and the filling of the joints was assured by the hydraulic head of the UHPC in the chimneys. Instrumentation was connected to data acquisition systems in order to monitor the bridge as the UHPC cured. On July 22, the plywood forms were removed from the joints. No cracks were observed during inspection of the shear keys. On July 24, a waterproofing membrane was installed on the top of the bridge as work continued on the approaches. An asphalt wearing surface was paved on the bridge on August 5. The following day, frames were set up underneath the bridge at mid-span. On August 7, a total of sixteen strain gages, seven LVDTs and three thermocouples were installed to monitor the bridge. The seven LVDTs were connected to the frame using brackets, and the frame was used as a reference surface for LVDT measurements, Figure 9 shows a LVDT mounted to the frame along with one of the strain gages.

Fig. 8 UHPC placement


Fig. 9 Bridge instrumentation

BRIDGE TESTING

One August 8, 2014, two loaded trucks were used to test the bridge. The weights of the trucks were 56.1 kip and 53.4 kip. The axle dimensions of each truck were recorded and each axle load was determined using truck scales. The center-to-center axle dimensions and axle loads are shown in Table 1. For each truck, number 1 refers to the front axle and 2 and 3 represents first and second rear axles.

Table 1: Axle and Tire

Truck Number	Axle	Axle Width, (in.)	Distance Between Axles (in.)	Left Tire (kip)	Right Tire (kip)	Axle Load (kip)	Truck Total Load (kip)
1	1	83	156.5	7.45	6.75	14.2	56.1
	2	72	55	10.45	10.0	20.45	
	3	72		11.15	10.3	21.45	
2	1	83	156.5	7.95	7.0	14.95	53.4
	2	72	55	10.4	9.5	19.9	
	3	72		9.7	8.85	18.55	

Four static load configurations were used in the tests, and the trucks were positioned to obtain the maximum moment at mid-span. These load configurations were:

1. A single 56.1 kip truck load placed in the left lane
2. A single 53.4 kip truck load placed in the right lane
3. Two trucks placed side-by-side with a 109.6 kip total load

4. Two trucks placed back to back in the left lane for a 109.6 kip total load (see Fig. 10). Data was collected by data acquisition systems. Data acquisition took readings during the time the truck(s) were stopped on the bridge.

Fig. 10 Truck loading

FINITE ELEMENT MODEL

A 3-D finite element model was developed to investigate the behavior of the adjacent box beam bridge and to be used in the future for a parametric study. The model was constructed from different parts. The parts consisted of the box beams, shear keys, reinforcement, dowel bars, and end diaphragms. Each part was created using ABAQUS/CAE software and the dimensions of each part were taken as described in the bridge plans. Three dimensional linear eight node brick elements (C3D8R) were used to model each part. More than 290,000 elements and 500,000 nodes were used to mode the bridge. Linear elastic modeling was used to predict the behavior of the bridge under the nondestructive truck test.

All materials used in the modeling were defined as linear elastic materials. The modulus of elasticity and Poisson's ratio were defined for each part as shown in Table 2. The compressive strength of the box beam concrete and UHPC grout were determined from cylinder tests, and were found to be 11.037 ksi and 22 ksi, respectively. The modulus of elasticity of the concrete was calculated using ACI 318-11 [11.]

For the box beam, the modulus of elasticity is calculated by using:-

$$E = 57000\sqrt{f_c'} = 5{,}988\ ksi \tag{1}$$

For the UHPC, the modulus of elasticity was calculated using Equation (2) (Russell and Graybeal [12]):

$$E = 49000\sqrt{f_c'} = 7{,}268\ ksi \tag{2}$$

Table 2: Materials Properties

Part	Modulus of Elasticity, ksi	Poisson′s ratio
Beams and Diaphragms	5,988	0.2
Steel Components	29,000	0.3

Strand	28,500	0.3
UHPC Shear Key	7,268	0.18

Each part was meshed individually with the box beams, shear keys, reinforcement, and end diaphragms seeded to 5 in. along the longitudinal direction. The cross section was meshed individually to obtain more accurate result. The finite element model is shown in Figure 11.

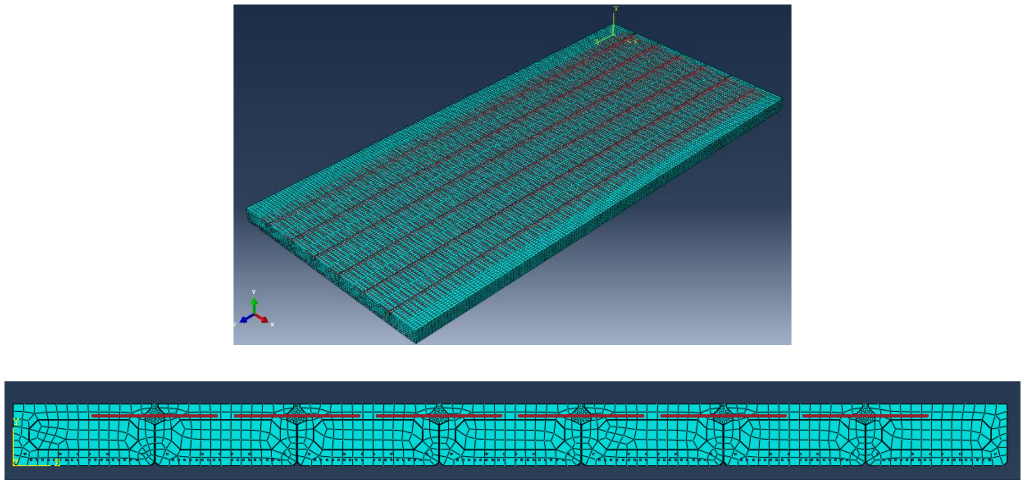

Fig. 11 Finite element model

Different interactions were used in the model. The interaction between the end diaphragms and box beams were modeled as a tie constraint where no slip or separation was allowed between two surfaces. The concrete and UHPC were modeled as host regions and the reinforcement was modeled as an embedded region. The interaction between box beams and shear keys was modeled as a surface-to-surface interaction. Both tangential and normal behavior was defined by using a friction coefficient of 1 and a stiffness of 10 kip/in.

The boundary conditions were defined in the model. A node, located at the bottom of the vertical dowel bar used to connect the beam to the abutment, was restrained at both ends of each beam. Three boundary conditions were consider for the model, pin-pin, pin-roller, and pin- roller with a longitudinal 15 kip/in spring.

The magnitude and position of the loading applied to the model was the same as the trucks used in the field test. The tire area was measured in the field and a uniform load was applied to elements used to represent the tire area in the model.

RESULTS

FINITE ELEMENT MODELING CALIBRATION

In order to calibrate the finite element model, the three different boundary conditions were used to compare results with field measurements. Truck load configuration 2 (one truck on right) was used for the calibration.

After modelling the three boundary conditions and comparing the results, the pin- roller with a longitudinal 15 kip/in spring boundary condition gave results that compared best to field measurements. The difference between field and finite element results was calculated by using Equation 3.

$$\% \, Difference = \left\{\frac{Field\ Results - Finite\ Element\ Results}{Field\ Results}\right\} * 100\% \tag{3}$$

By using the pin- roller with a longitudinal 15 kip/in spring boundary conditions, the maximum difference in deflection was found to be approximately 4%. The results show that the behavior of bridge is neither pin-pin nor pin-roller, but somewhere in between these two boundary conditions. The calibration results are shown in Figure 12.

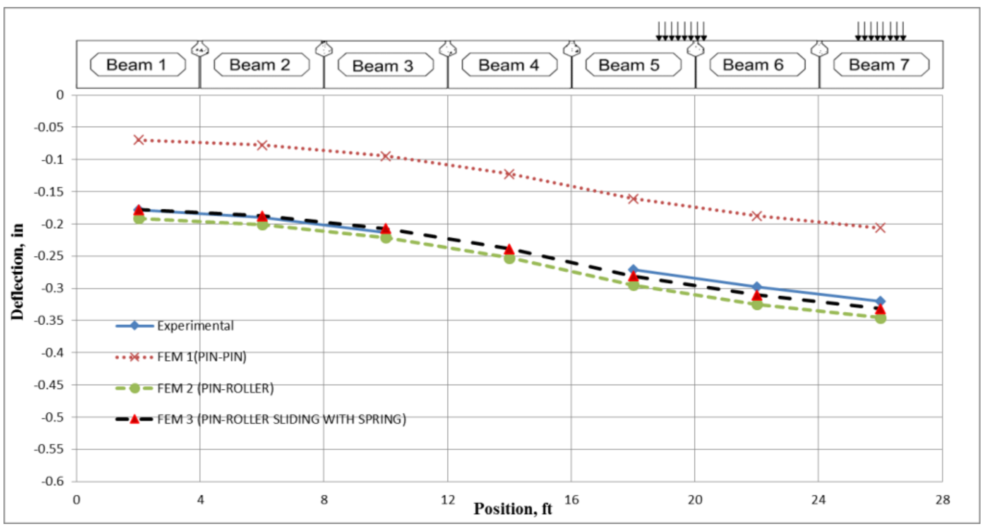

Fig. 12. Mid span deflection, versus bridge width for load configuration 2

MID SPAN DEFLECTIONS

The deflection was measured by using LVDT's when the bridge was loaded with the first load configuration (one truck on the left side). The total truck load was 56.1 kips. The results showed that the loaded beams generally exhibited the largest deflection (see Figure 13). Beam 1 should have the largest deflection because it was directly under the load. However, Beam 2 showed the largest deflection of 0.269 in. The lower than expected deflection of Beam 1 might have been due to an issue with the LVDT, its mounting, or the support frame, or Beam 1 might have had a higher stiffness. The LVDT on Beam 4 did not register a measurement. Beams 5-7 all deflected under the load configuration, which emphasizes the ability of the new shear key configuration to transfer the live load between beams, thereby causing the bridge to deflect as a system. The deflections from the experimental tests were compared with the finite element modeling results. The deflections were close to experimental results except for Beam 1. The maximum deflection from the finite element modeling was 0.35 in. and occurred in Beam 1 because it was directly underneath the load and an exterior beam.

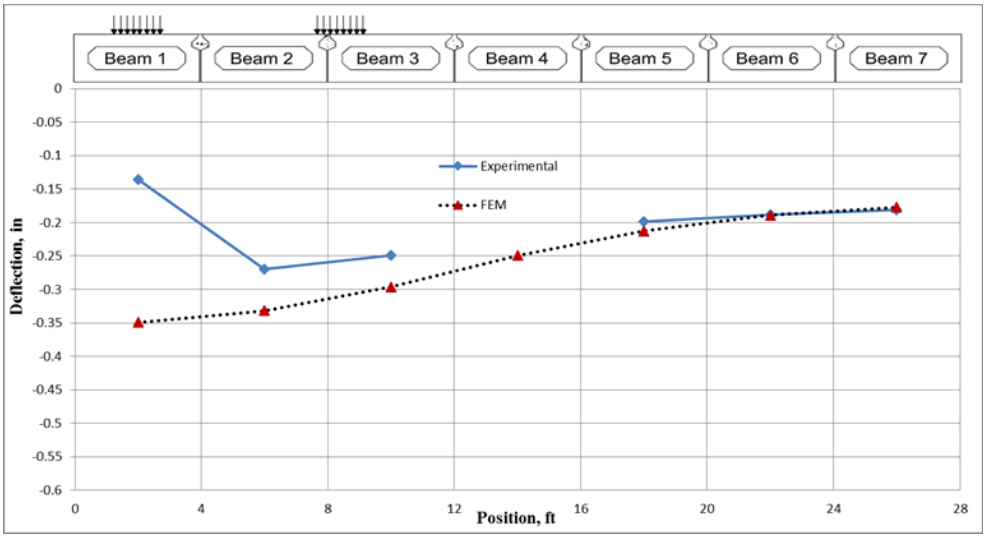

Fig.13 Deflection at mid span of the bridge for load configuration 1

The second load configuration (one truck on right side) was used in the calibration as shown in Figure 12. The total truck load was 53.4 kips. The results showed that loaded Beam 7 had the highest deflection of 0.320 in. However, beams not directly loaded still exhibited deflection due to the load transfer mechanism, as shown in Figure 12. The maximum deflection from finite element modeling was 0.33 in. and occurred in Beam 7.

For the third load configuration (two trucks side by side at mid span), the total truck weight was 109.6 kips. The maximum recorded deflection for this load configuration occurred on Beam 7 and was 0.482 in. as shown in Figure 14. The right side of the bridge exhibited more deflection than the left side. However, the truck on the right side was 53.4 kips and the truck on the left side was 56.1 kips. This may indicate a difference in the stiffness of the beams or issues with the instrumentation. The experimental results were also compared with the finite element results. The results show that there was a difference between experimental and finite element especially for first three beams. Again, this may attributed to issues with the instrumentation or differences in beam stiffness. The maximum finite element deflection was 0.533 in. and occurred in Beam 1, which was reasonable because the truck weight was higher on the left side. However, the finite element model assumes a uniform stiffness for all the beams. The bridge deflected as a unit, which emphasizes satisfactory load transfer between beams using the new shear key configuration.

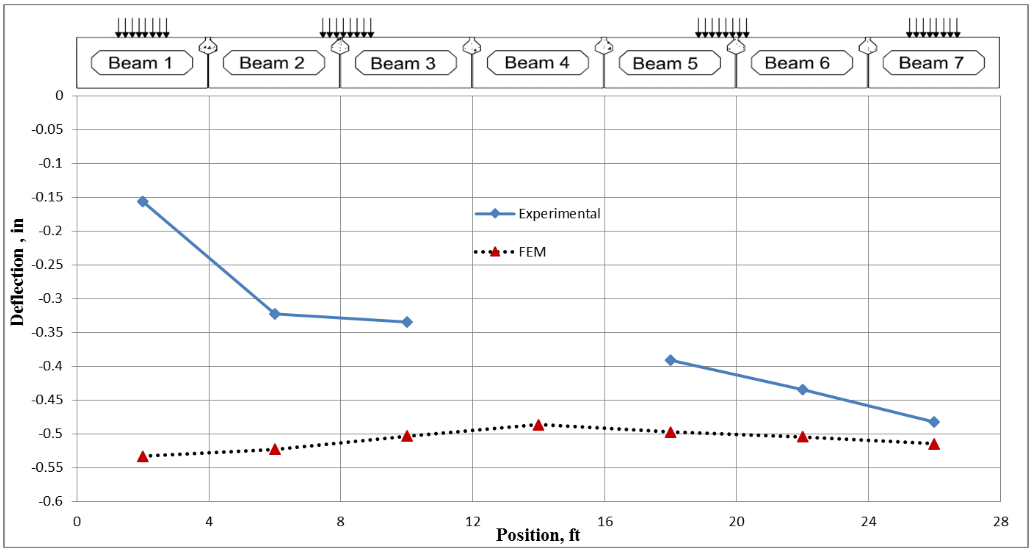

Fig. 14 Deflection at mid span of the bridge for load configuration 3

The measured deflections when the bridge was loaded with two trucks back to back on the left side are shown in Figure 15. The total truckload on the left side was 109.6 kips. The maximum deflection on Beam 2 was 0.381 in. However, Beam 7 still had deflection due to load transfer between beams by the new shear key configuration. The results from the finite element modeling are also presented in Figure 15, which shows that there was a difference between experimental and finite element especially for first three beams. The difference between the experimental and finite element results on the left side are likely due to the same reasons already discussed. However, the loaded side did deflect more than unloaded side, as expected. The maximum FE model deflection was 0.584 in. and occurred on Beam 1, which was reasonable because both trucks were on the left side. The bridge deflected as a unit, which emphasized the satisfactory load transfer between beams by using the new shear key configuration.

The differential deflection and joint opening from the finite element modeling under load configuration 4 are shown in Figures 16 and 17, respectively. This load configuration was chosen because the bridge was loaded on one side with total load of 109.6 kips. Joint 3 shows the maximum differential deflection because it exists between the loaded and unloaded part of the bridge. The maximum differential deflection was 0.021 in. The maximum joint opening occurred at Joint 2 because the load was applied on the shear key directly which create a higher joint opening. The maximum joint opening was 0.014 in. The results show that the joint of higher differential deflection exhibited lower joint opening.

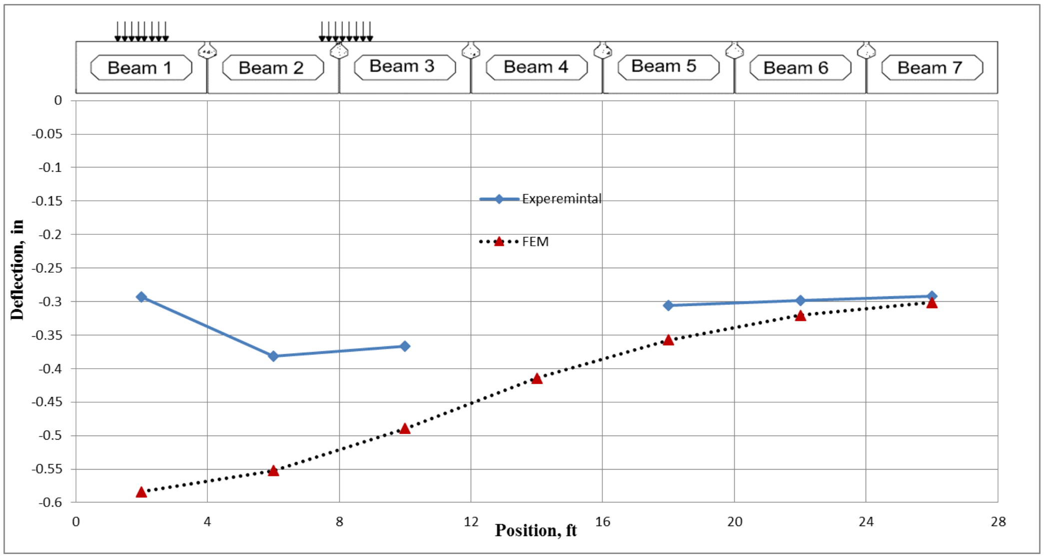

Fig.15 Deflection at mid span of the bridge for load configuration 4

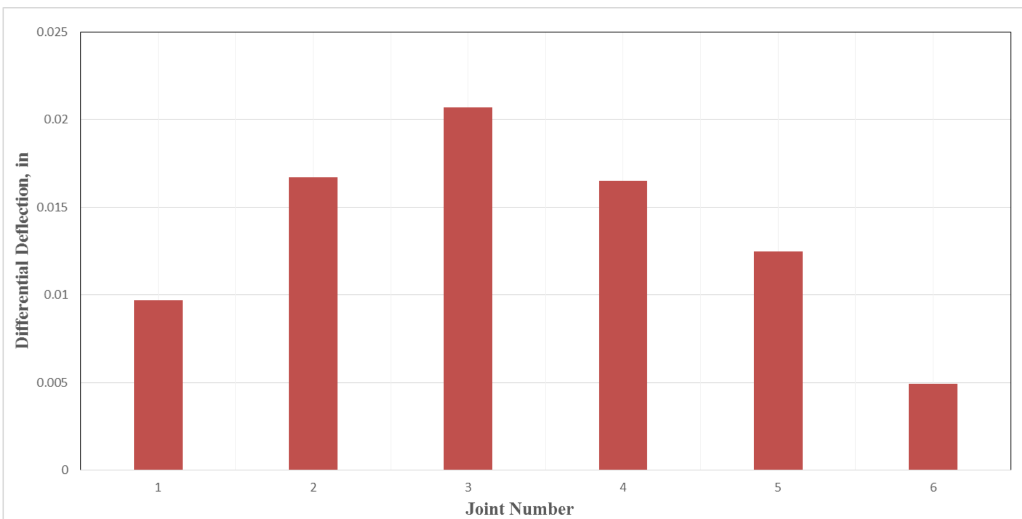

Fig.16 Differential deflection at mid span of the bridge load configuration 4

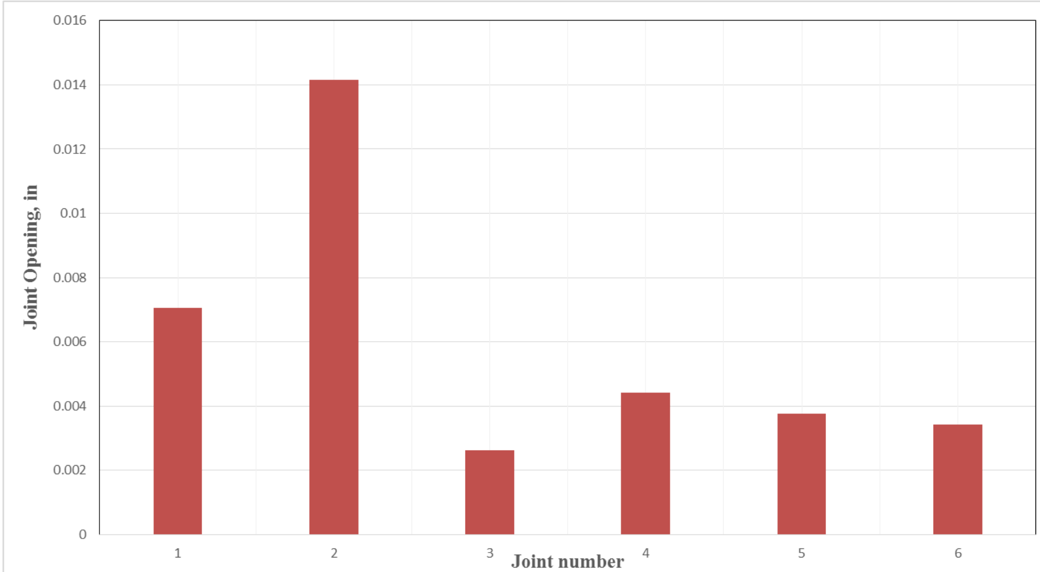

Fig. 17 Joint opening at mid span of bridge load configuration 4

A comparison between the deflections from experimental and finite element modeling for the different load configurations are shown in Figures 18 and 19, respectively. Figure 18 shows that load configuration 3 had the largest experimental deflection. However, the maximum deflection from finite element modeling was in load configuration 4. The deflections determined by finite element modeling were smooth for all load configurations which emphasizes the ability the new shear key configuration to transfer the load between beams.

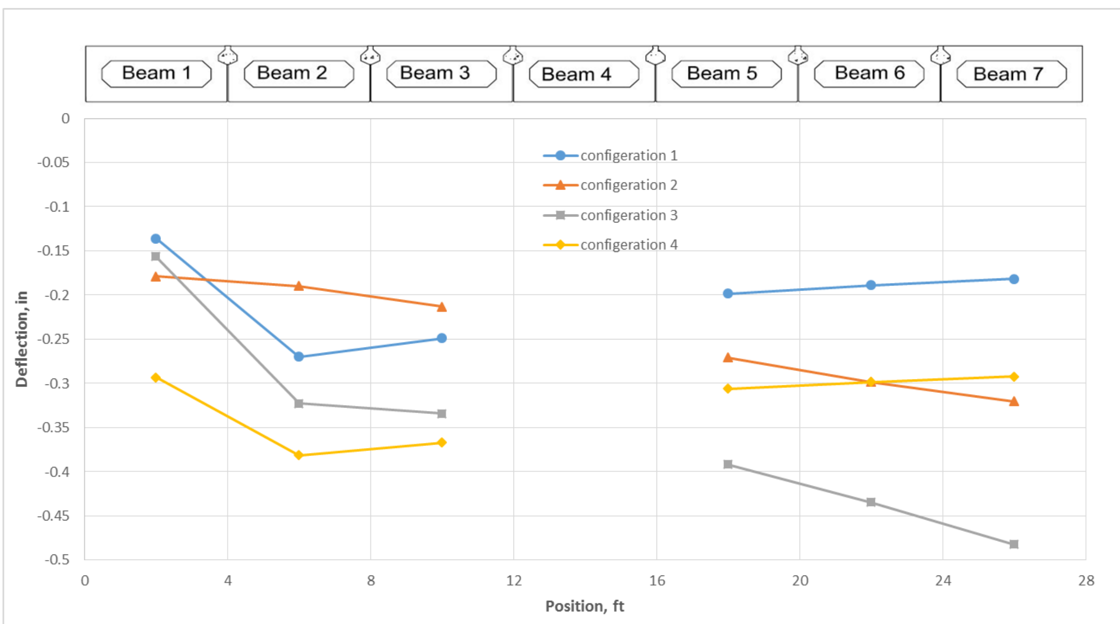

Fig. 18 Mid span deflection for all load configurations from experimental

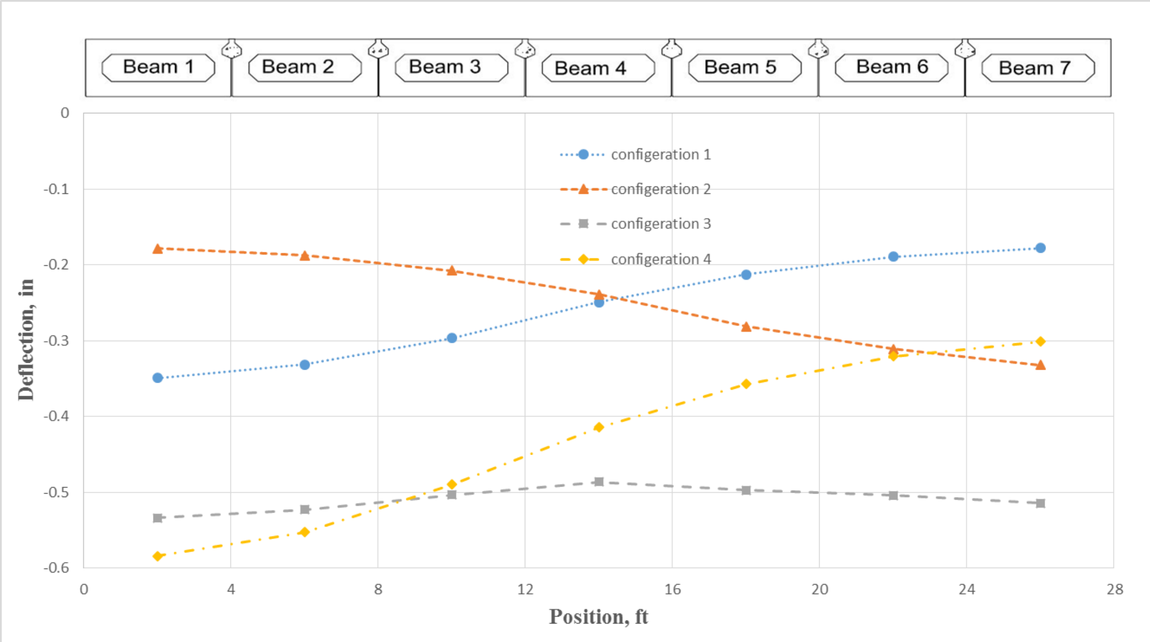

Fig. 19 Mid span deflection for all load configurations from finite element models

CONCLUSIONS

Based on the construction, truck loading, and finite element modeling, the following conclusions were drawn:

1. The new bridge was opened to traffic approximately 2 ½ months after closing the road. The new design did not change the relatively quick process of adjacent box beam bridge construction, which is one of its desirable attributes.
2. The loaded beams exhibited more deflection than the beams that were not loaded.
3. The bridge behaved monolithically, which emphasizes the ability of new shear key design to transversely transfer load across the bridge.
4. The maximum differential deflection was 0.021 in., which was approximately equal to the limiting value of 0.02 in. expressed in Huckelbridge et al.[2]
5. The finite element modeling results compared well to the field test results on the right side of the bridge but not on the left side. In addition, it appears that the beams did not all have the same stiffness based on a comparison of results and further refinement of the finite element model should be completed prior to a parametric study.
6. The joint that had the maximum differential deflection exhibited lower joint opening based on the finite element modeling. There did not appear to be any relationship between differential deflection and joint opening.
7. The maximum joint opening determined by the finite element modeling was 0.014 in. and occurred at Joint 2. This occurred because the load was directly applied on the shear key which created a larger joint opening.

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