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Field Monitoring of Skewed Integral Abutment Bridges

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ABSTRACT
Integral abutment bridges (IABs) have come of interest due to their decreased construction and maintenance costs in comparison to conventional jointed bridges. Because their structural behavior is still not fully understood, two IABs (of different geometry and soil properties) were selected for field monitoring. The monitored bridges, located in northern Illinois, include a 30-degree skew four-span continuous IAB and a 42.5-degree skew single-span IAB, both of which have steel superstructures and incorporate pile top relief for HP14 piles. Construction of the bridges began in the spring of 2013, and instrumentation data has been collected since May 2014. Field instrumentation includes pile, girder, and concrete embedment strain gages, as well as displacement transducers and tiltmeters. This paper presents findings from the first full year of data collection, with the bridges experiencing a summer-to-winter-to-summer temperature cycle. Comparisons are also made with 3-D finite element models of each instrumented IAB. As expected, results of pile strains, girder strains, global movements, and abutment rotations all show clear trends with temperature. The absolute and relative values of these findings help to enhance the detailed understanding of IAB behavior, as well as improve IAB analytical modeling.

KEYWORDS: *Integral Abutment Bridges, Field Monitoring, Skewed Bridges, Pile Strain, Bridge Movement*

1. INTRODUCTION

Integral abutment bridges (IABs) are gaining popularity amongst departments of transportation within the United States because of their lower maintenance and construction costs, as well as longer service life. Unlike conventional bridges, IABs act as a continuous unit when subjected to thermal loads. This is achieved through the monolithic casting of the abutment and deck, as well as via the embedment of the girders and the pile foundations into the abutment (Figure 1.1). However, the behavior of these bridges, especially in regards to the superstructure, is still not fully understood. To address this knowledge gap, a team in the Department of Civil and Environmental Engineering at the University of Illinois is currently conducting a comprehensive IAB research program, including a thorough parametric study with three-dimensional (3-D) finite element bridge models and extensive field monitoring of two IABs located in northern Illinois. This paper details the instrumentation and data collection scheme associated with the field monitoring of these two bridges, and provides select results from the field data. Ultimately, these field data and complementary analysis results will be used to develop more consistent substructure and superstructure design guidelines for future IAB construction, with potential for expanding the current bridge length and skew limits that are in place for IABs.

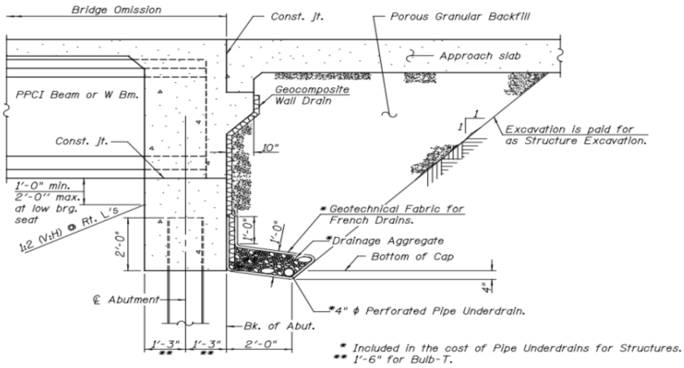

Figure 1.1 General Illinois Department of Transportation (IDOT) Integral Abutment Detail

2. FIELD INSTRUMENTATION

2.1 Site Descriptions

As part of the current research, two skewed integral abutment bridges were selected for field monitoring. Both carry eastbound Illinois Tollway (I-90) traffic in northern Illinois, and the configurations are: a four-span continuous bridge over the Kishwaukee River in Cherry Valley (“Kishwaukee”) and a single-span bridge over the Union Pacific Railroad in Gilberts (“UPRR”). The steel H piles at each site have a weak-axis orientation, with the web perpendicular to the longitudinal axis of the bridge, as per IDOT standards. Construction of each bridge began during the spring of 2013. Basic geometric properties of each site are listed in Table 2.1 below.

Table 2.1 Geometry of the two bridges being monitored as part of this study.

	Kishwaukee	**UPRR**
Spans	38.1 m, 46.3 m, 46.3 m, 36.6 m (125 ft, 152 ft, 152 ft, 120 ft) West to East	56.2 m (184.5 ft)
Overall Length	167.3 m (549 ft)	56.2 m (184.5 ft)
Width	21.0 m (69 ft - 1/4 in.)	21.0 m (69 ft - 1 in.)
Abutment Skew	30°	42.5°
Number of Girders	8	10
Girder Size	Plate Girder: 1524 mm (60 in.) Web	Plate Girder: 1829 mm (72 in.) Web
Number of Piles	30 (15 on each end)	38 (19 on each end)
Pile Designation	HP14x117	HP14x89

The Kishwaukee River Bridge was completed in October 2013, and data collection began on May 24, 2014. This bridge employs the largest pile size currently in service for IDOT IABs, as well as extends only one foot short of the current maximum permissible IDOT IAB length of 167.6 m (550 ft) for multi-span bridges. Pile relief has been employed, with the top 3.05 m (10 ft) of each pile encased by a corrugated pipe with a bentonite fill. All other aspects of the bridge are relatively standard in regards to IDOT details and specifications.

The UPRR Bridge was completed in April 2014, and data collection began on June 18, 2014. This bridge exceeds the current IDOT limit set on the length for a single span IAB of 51.8 m (170 ft), as well as nears the current 45 degree limit for skew. Additionally, this bridge was constructed with mechanically-stabilized earth (MSE) walls at each abutment, which are no longer permitted for use with IABs according to the latest IDOT Memo regarding use and design of IABs (IDOT ABD Memo 12.3). Pile relief was employed on this bridge as well, with the top 3.05 m (10 ft) of each pile encased with a corrugated pipe with a loose sand fill. All other aspects of this bridge are relatively modest or are based on standard IDOT details.

2.2 Instrumentation Scheme

All instruments and data acquisition equipment were procured from Geokon, Inc. To minimize the number of instruments needed to monitor the complete behavior of the IABs, symmetry of the two bridges was utilized in the instrumentation layout. The instruments used are summarized in Table 2.2, all of which (except for the multiplexers) have temperature sensors.

Table 2.2 Instruments used at each bridge site.

Instrument Name	Quantity	
	Kishwaukee	UPRR
Arc-welded Strain Gage (for piles)	20	20
Spot-welded Strain Gage (for girders)	51	30
Embedded Strain Gage	21	11
Tiltmeter	8	7
Displacement Transducer ("Crackmeter")	6	0
Multiplexer	7	5

General plan views and elevations of the bridge instrumentation can be seen in Figures 2.1 and 2.2. A similar scheme as depicted in Figure 2.2 was used for the UPRR Bridge.

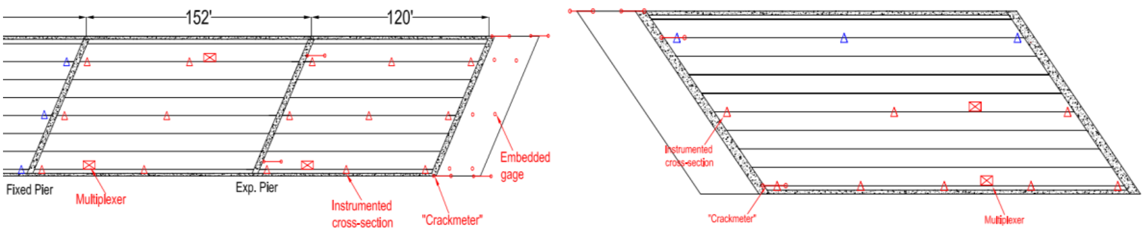

Figure 2.1 Plan View of (a) Kishwaukee River Bridge and (b) UPRR Bridge

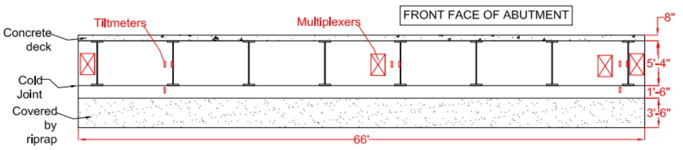

Figure 2.2 Elevation of East Abutment at Kishwaukee

2.2.1 Strain Gages

Three abutment piles of each bridge were instrumented with arc-welded strain gages prior to being shipped to the bridge sites and placed. Based on the MSE wall details, the instrumented portions of the piles at UPRR were spliced into placed, whereas the piles at Kishwaukee had to be driven. The instrumented piles were chosen so as to represent the North, Middle, and South portions of the bridges, and they are located at the east end of Kishwaukee and west end of UPRR. Each bridge was planned to have 2 exterior piles with 8 gages each and 1 interior pile with 4 gages. However, when the piles were being installed at Kishwaukee, the south exterior pile and the interior pile were switched. The details for the UPRR Bridge can be seen in Figure 2.3, with a similar scheme at Kishwaukee.

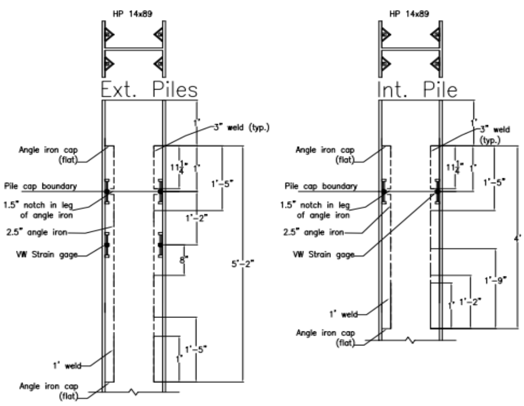

Figure 2.3 Details of pile instrumentation for the UPRR Bridge

Because the piles were to be instrumented before being placed, the gages were protected using steel angles, and the piles for the Kishwaukee Bridge had a tapered end-cap to prevent the protective angle from being dislodged during driving. The same Geokon gages and a similar protection scheme were used by Olson et. al (2009) in an earlier detailed study of IAB piles. The arc-welded strain gages were connected using two steel mounting blocks which, combined with set screws, held each gage in place. A hose clamp secured the coil housing to the gage, and the wires exited the protective angle through a small notch that was later sealed off using spray foam insulation.

Spot-welded strain gages were used on the three girders corresponding to the instrumented piles. These were placed at various cross sections along the lengths of each bridge, in addition to embedded strain gages within the 203.2 mm (8 in.) concrete deck (Figure 2.4). Six embedded gages were placed in the approach slab of the Kishwaukee Bridge as well, in addition to the 15 installed in the deck.

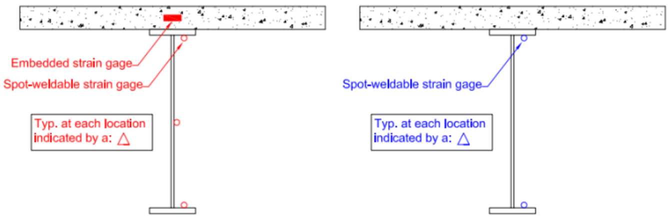

Figure 2.4 Detailed Cross Sections of Superstructure Instrumentation on the Kishwaukee River Bridge

The spot-welded girder gages were attached at the steel fabricators for each bridge in July 2013 before being shipped. They are less durable than the arc-welded gages, but do have a cover plate to protect them against damage during construction activities and general elemental exposure. Girder gage wires were routed along bottom flanges and zip-tied to nuts that were epoxied to the flange.

Embedded strain gages were tied off to rebar and isolated with wooden blocks so that they would measure the strain in concrete. Coil housing was connected to the gage, with wiring run along rebar and then through the forms down to multiplexers connected to the girders.

2.2.3 Tiltmeters

The abutment and girder ends on each bridge were instrumented using tiltmeters. These measure rotation about only one axis, which was oriented as the transverse axis of the bridge (i.e. rotation in the longitudinal direction). The tiltmeters were attached with brackets either bolted to the steel girders or anchored to the concrete abutment. A set screw on the bottom of the device was removed to initialize the instrument. Because the tiltmeters are flush with the abutment face and the girder, rotation measurements of the abutment and girder are not parallel to one another, and an offset equal to the abutment skew must be used when analyzing the field data.

2.2.4 Displacement Transducers (“Crackmeters”)

The girder-pier, abutment-approach slab, and approach slab-transition slab interfaces at the north and south ends of the Kishwaukee Bridge are instrumented with crackmeters. The largest available crackmeters were used so as to ensure the ability to measure displacements at the approach slab-transition slab interface, which hold the strip seal. To install the crackmeters, each end was anchored into the concrete with a small spacer. At the girder-pier interface (for the east expansion pier), the crackmeter was installed using brackets, with one end bolted to the cross frame connection plate and the other end anchored to the concrete pier.

2.2.6 Data Acquisition

Multiplexers are used as junction boxes, with a maximum of 16 instruments connected to each. These instruments then connect to the central datalogger at each site. A daisy-chain was utilized to connect multiple multiplexers to each other in series before connecting to the datalogger, in order to reduce the amount of wiring required. The multiplexers are mounted either to the steel girders or the face of the concrete abutment.

Geokon Micro-1000 Dataloggers were used for data collection. The datalogger works with LoggerNet software to store and send data. In order to remotely collect data from the datalogger, a Sierra Wireless Airlink LS300 Verizon cellular modem was placed at each site. A null modem adapter needed to be used so that the modem could directly communicate with the datalogger without having a dedicated computer at each site. A Campbell Scientific template was uploaded to the modem to give it the correct configuration, and the modem's IP address was input into LoggerNet. An antenna was also used with each modem to increase the signal strength. To supply power to the datalogger and modem, a 130 W solar panel was installed at each site, along with 2-110Ah batteries in parallel.

Data is sent to the office from each datalogger at a user-defined interval. This interval was set to 15 minutes at the start of the data collection period. In March 2015, the collection interval at the Kishwaukee Bridge was decreased to every 5 minutes. It takes approximately 2 seconds to read a vibrating wire gage, which means that it takes a few minutes for all of the data to be collected and sent through the datalogger. For this reason, smaller collection intervals on the order of seconds instead of minutes are not feasible. The intervals used, however, are adequate for looking at thermal load effects. Then, to process this data, a macro was created that copies the new data to a field data Excel Workbook, deletes the old file, and sorts the incoming data so that it can be used for analysis.

3. FINITE ELEMENT MODELING OF THE SITE BRIDGES

Finite element (FE) models were created of both site bridges using the structural analysis program SAP2000 v14 (CSI 2009) to establish a baseline for comparison with the field data. The field monitoring is part of a concurrent parametric study on IABs, and the same modeling scheme and assumptions detailed by Fahnestock et. al. (2015) were used. Shell elements were used to represent the abutments, concrete decks, and piers, whereas frame elements were used for the piles, girders, and parapets. Soil resistance on the piles was modeled using nonlinear springs, whose p-y curves were calculated with *LPile* software (Ensoft 2005). Boring logs were used to determine the foundation soil properties at each site, and pile relief was employed by reducing the stiffness of the top soil springs. There is more uncertainty associated with the UPRR model because parameters such as the sandy soil and MSE wall are more difficult to model accurately. Models were run under dead load, thermal load, and live load conditions. For the thermal load cases, the FE models impose a staged dead load before applying an incremental thermal loading. They include both a positive and negative thermal load, which are temperature changes of ΔT = +/- 44.4 °C (80°F). To obtain results from purely thermal loads, the results from the dead load only case are subtracted from the thermal load case.

4. SITE MONITORING RESULTS

4.1 Average Superstructure Temperature

To examine and compare the effect of temperature with the results obtained from the sensors, a definition of the average superstructure temperature needed to be made. After observing temperature trends from the gage readings on each bridge, it was determined that the exposed girder strain gages were most indicative of the superstructure temperature. Further investigation led to the definition of average superstructure temperature as being the average of the temperature readings from all gages on the middle girder. For the UPRR Bridge, the temperature readings from the gages closest to the west abutment were not included in the average because the temperature variations at that location are inconsistent with the trends of the rest of the bridge.

4.2 Pile Strains

The instrumented piles have strain gages located at the quarter-span of the pile flanges at the pile head, as well as at 355.6 mm (14 in.) below for two of the three piles on each bridge. Findings for the Kishwaukee River Bridge are much clearer than those for UPRR, most likely due to the larger effective expansion length (EEL). Strain readings were initialized at the first recorded data point, taking that as most indicative of the condition of the bridge when it first became integral. With this initialization and plotting change in strain versus time (Figure 4.1), gage locations 1 and 4 exhibit the largest magnitude of strain change. As bridge expansion/contraction occurs, these locations see the most compressive/tensile strain. Additionally, each gage location goes through cycles of tensile and compressive strain throughout the day, which correspond well with the variation in temperature.

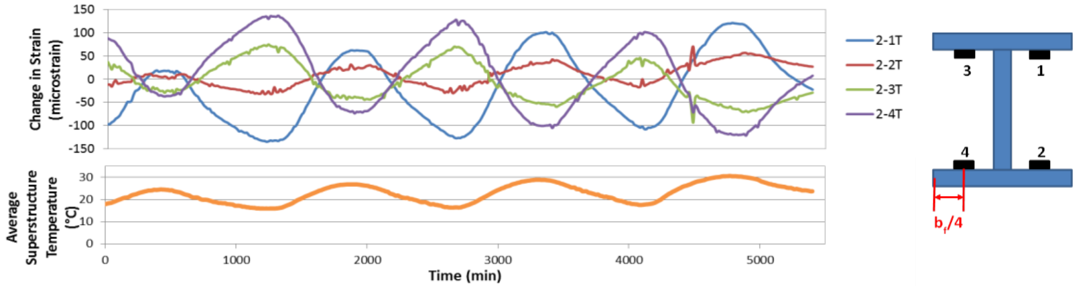

Figure 4.1 Kishwaukee Acute Pile Strains from 10:00am, 5/28/2014 to 4:00am, 6/1/2014

There is a strong linear trend of change in pile strain with change in superstructure temperature, as seen in both the field data and SAP model results. With the initialization scheme used, the effect of dead load is removed and strains purely due to thermal changes are measured. Because bridge construction was completed and data collection began in relatively warm weather, there is little data to represent the positive change in temperature region. Therefore, comparisons with the FE models are focused on thermal contraction of the bridges. In both bridges, the reported strains at the gage locations are substantially less than the 1724 microstrain required to reach yield, indicating that there could be significant pile deformation capacity available. Figures 4.2 and 4.3 show that the strain values predicted by the finite element models overestimate the field strains; however, the Kishwaukee model closely captures the extreme strain values seen at gage locations 1 and 4. Even though the models show greater strains, the overall trend of pile strain at each gage location is consistent between the field data and model results for negative change in temperature (contraction). It appears that the soil springs of the UPRR model are too stiff, which would explain why the model strains are significantly higher than the field strains. Slight differences between the model and field data are to be expected though, because of the nature of the loading. Focusing on contraction, the models employ an incrementally decreasing change in temperature. This is far from the case in the field, as the temperature goes through daily fluctuations of highs and lows, which can be considered more of a cyclic temperature change than a monotonic one.

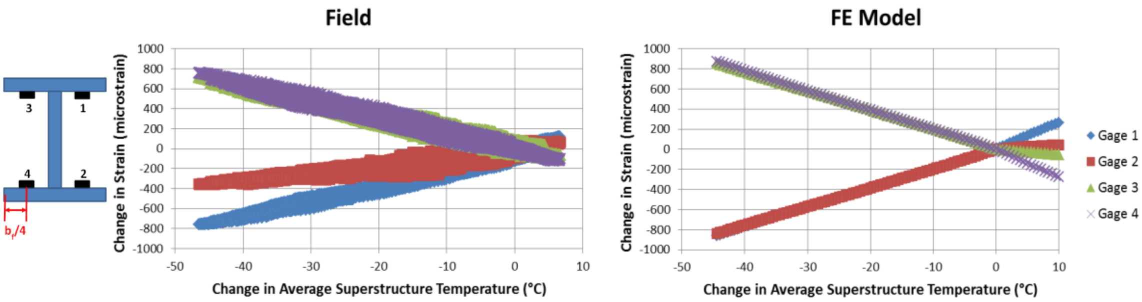

Figure 4.2 Kishwaukee Acute Pile Strains from Field Data and Pure Thermal FE Model Results

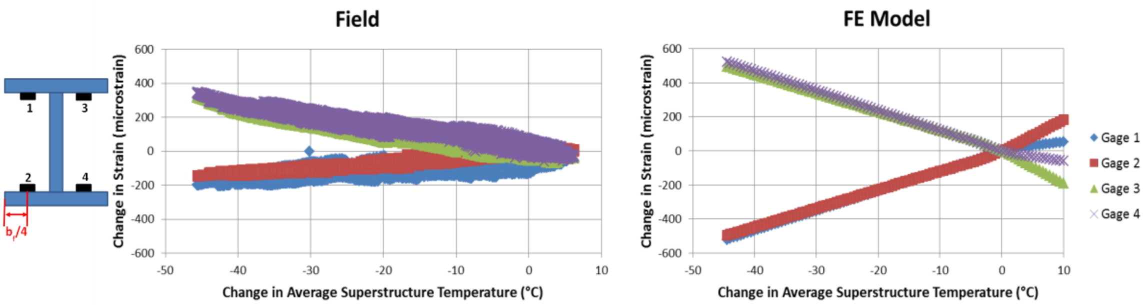

Figure 4.3 UPRR Acute Pile Strains from Field Data and Pure Thermal FE Model Results

4.3 Bridge Movement

Crackmeters (at Kishwaukee only) and tiltmeters were implemented to give a better picture of the overall bridge movement, in addition to surveying conducted at the Kishwaukee site. As with the pile strain gages, initial values were not recorded for the crackmeters or tiltmeters. Therefore, the same initialization scheme as described in the previous section was used.

With the tiltmeter readings, the differential rotation between the abutment and the girder can be examined, allowing the rigidity of the connection to be analyzed. Differential rotation can also be examined between the upper diaphragm and lower footing regions of the abutment, which would help to determine how rigid the cold joint connection actually is. The differential rotations seen in the field can then be used to better calibrate these connections in the finite element models. As can be seen in Figure 4.4, the rotations from the winter to summer period of expansion cycle back and follow the same trend as the initial summer to winter period of contraction. The figure also shows that the field data produces banded plots when differential rotation is compared to the ambient temperature. These distinct bands could be due to different live loads within the adjacent span and/or cracking occurring in the abutment causing the rotation to change. The plots of these rotations versus time are also banded, which further supports such conclusions. The middle band of points centered about 0 degree rotation has a greater weight of points than the outer bands. Live loads within the span cause greater abutment rotation than thermal changes and therefore, it seems plausible that the additional horizontal bands are caused by various live load scenarios. The central band of data points on both the differential rotation versus time and versus time plots can then be considered due to solely thermal changes.

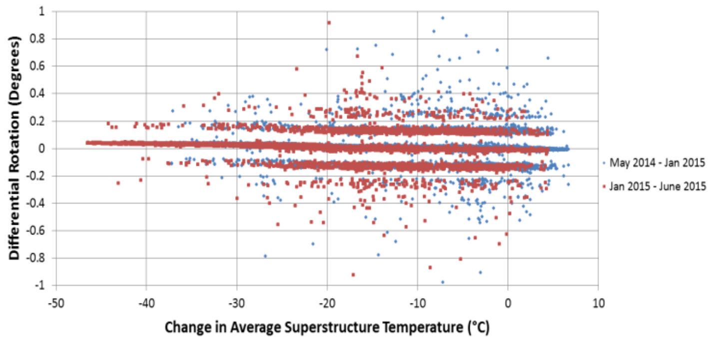

Figure 4.4 Differential Rotation Between the Abutment and Middle Girder at the Kishwaukee Bridge

As expected from the concepts of thermal expansion and contraction, global bridge movement is directly affected by temperature changes. This is seen in the linear nature of the plots of crackmeter displacement values versus change in temperature. Surveying was conducted at two points on both the north and south ends of the east abutment at Kishwaukee. The survey points were selected to be roughly aligned with the top and bottom of the girders. Figure 4.5 shows results from all crackmeters at Kishwaukee from the initial collection date until the present, in addition to the displacements measured from surveying. Field results indicate that the expansion joint at the approach slab – transition slab interface allows for the greatest magnitude of movement with change in temperature when compared to the other locations. The crackmeter readings fall within a relatively wide band due to the influences of live load patterns. As shown in the figure, the pier-girder displacements are less than what is expected from theoretical free expansion (using a coefficient of thermal expansion of 10.8 x 10^{-6} mm/mm/°C (6 x 10^{-6} in./in./°F) for the 46.3 m (152 ft) span of composite plate girder and concrete deck).

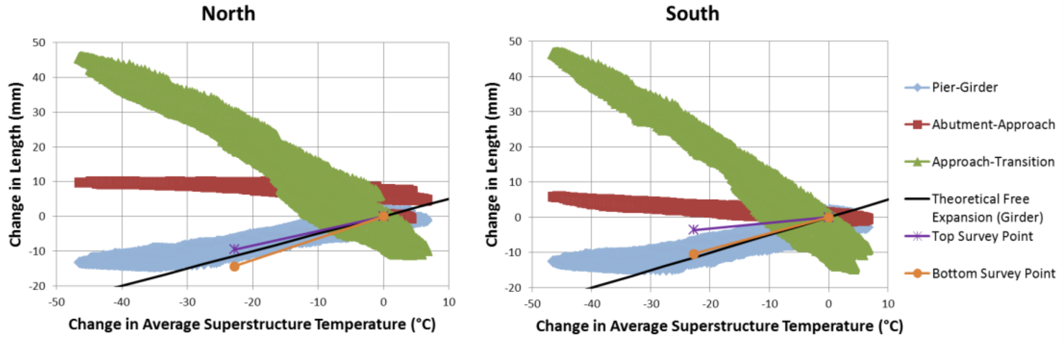

Figure 4.5 Crackmeter and Survey Results at the Kishwaukee Bridge East Abutment

5. CONCLUDING REMARKS

It is difficult to accurately model the complex field conditions of an integral abutment bridge (IAB), so it is beneficial to conduct an integrated study that links finite element analysis with field instrumentation and monitoring in order to gain a clearer picture of bridge behavior and to validate modeling assumptions. The instrumentation schemes described in this paper required extensive advance planning and detailed coordination with construction sequences and field conditions. Power, data transmission and wiring configurations were also essential elements that were developed in conjunction with the bridge owner and design / construction team. Despite a few setbacks and field complications, the University of Illinois research team successfully installed the data collection system and is now continuously monitoring the two bridges from almost all of the 106 channels at the Kishwaukee Bridge and 68 channels at the UPRR Bridge.

After the first year of data collection, a general understanding of overall bridge behavior is emerging. Specifically at the UPRR Bridge, low pile strain readings at the gage locations indicate that there is adequate pile capacity and piles are not expected to yield under typical service conditions. The tiltmeter readings show small differential rotations between the bridge girders and abutments, which is a sign that the connections are mostly behaving rigidly as commonly assumed. Furthermore, the superstructure movement as measured by the crackmeter displacement at the pier-girder interface is less than what is expected from free expansion, which is a trend that agrees with what has been observed in the companion parametric study. Other response parameters such as girder, deck, and approach slab strains are currently being investigated and compared with finite element models as well. Overall, the data being collected follows expected trends and will be a valuable resource for future finite element model updates and developing revised design provisions for IABs.

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