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INSTRUMENTATION AND LONG-TERM MONITORING OF AN INTEGRAL-ABUTMENT BRIDGE SUPERSTRUCTURE

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ABSTRACT

Strain gauges were installed in the girders and deck of an integral abutment bridge to determine the behaviour of the superstructure to daily and seasonal thermal fluctuations. The two-span, 76 m-long bridge has no expansion joints; rather, the prestressed-concrete girders are cast directly into the abutments which are in turn supported by flexible pile foundations. Details of field instrumentation along with monitored data over a period of eight years are presented in this paper. Gauges measured the longitudinal strain in the girders and deck and were positioned at different elevations to provide a strain distribution across the superstructure depth. From the strain data, moments were computed at each of the installation locations. Data indicate that the superstructure responds to thermal fluctuations as expected with midspan moments smaller in magnitude when compared to the moments measured at the abutment and pier locations. Furthermore, data confirm that the level of sun exposure affects the thermally-induced moments measured in the girders.

Keywords: integral abutment bridge, field monitoring, superstructure, girder moment, girder strain

1. INTRODUCTION

Integral abutment bridges are jointless bridges that accommodate thermal superstructure expansion and contraction through abutment movement. These structures have several advantages over traditional bridge design such as lower initial construction costs, fewer maintenance costs, and greater earthquake resistance (Clatyon et al. 2006). Despite widespread construction of these structures, a nationally-accepted design method does not exist. Rational guidelines on details such as maximum length, skew angle limits, and girder live load distribution factors for integral abutment bridges are lacking in the Canadian Highway Bridge Design code and American Association State Highway Transportation Officials LRFD Bridge Design Specifications (Nikravan 2013). Each transportation department is left to rely on the experience of its engineers and limited research to formulate design guidelines (Kunin and Alampalli 2000, Civjan et al. 2004, Hassiotis et al. 2005). In New Brunswick, the Department of Transportation and Infrastructure (NBDTI) relies on guidelines given in a variety of research articles including the Ministry of Transportation of Ontario (MTO) handbook (Husain and Bagnariol 1996).

Research on the behaviour of integral-abutment bridge superstructures has been limited and the types of structures instrumented are quite varied. For example, superstructure strains have been monitored in a concrete deck and steel frame bridge (Elgaaly et al. 1992, Sandford and Elgaaly 1993), a concrete deck and steel girder bridge (Shoukry et al. 2006), concrete deck and prestressed girder bridges (Lawver et al. 2000, Huang et al. 2005, Barker and Carder 2001, Fennema et al. 2005, and Kim and Laman 2012), and a concrete deck and precast concrete voided plank bridge (Ooi and Lin 2006). However, even with the instrumented concrete deck and prestressed girder bridges, the foundation type varies for each with one having a shallow foundation, several having deep foundations, and two having mixed foundations, e.g., one abutment supported on a shallow foundation and the other abutment supported on a deep foundation. Furthermore, much of the data presented from these studies is solely focussed on axial strains in the girders rather than a strain profile from which girder moments may be computed. Given the limited and varied field data and the fact that, to the authors' knowledge, no such information for integral-abutment bridges in Canada exists, this research was undertaken.

2. PROJECT DESCRIPTION

2.1 Route 2 High-Speed-Connector Underpass

In excess of 100 sensors were installed during the construction of an integral abutment bridge in Fredericton, NB. Instrumentation consisted of sensors to monitor longitudinal strains in the superstructure, abutment movement, abutment-foundation-pile strains and earth pressures acting on the abutment. The performance of the substructure of this bridge has been reported by Huntley and Valsangkar 2008, 2013, and 2014. The instrumented bridge structure is a three-lane underpass with two 38 m spans connected by the deck and intermediate diaphragms to create a single continuous span of 76 m. The bridge has no skew, but it is unsymmetrical due to a sidewalk along the north side of the deck. Eight prestressed concrete girders are provided per span for a total bridge width of 17.6 m. The girders are New England Bulb Tees with a height of 1.8 m (NEBT 1800). Details for the girder section are given in Table 1.

Table 1: AASHTO type NEBT 1800 girder details.

Rte. 2 High Speed Connector Underpass: type NEBT 1800 girders	
Depth (mm)	1800
Area (mm^2)	625 x 10^3
Weight (kN/m)	14.74
Neutral axis height from bottom of girder (mm)	855.1
Moment of inertia (mm^4)	274.9 x 10^8
Minimum strength at 28 days, f_c (MPa)	50
Minimum strength at time of release, f_c (MPa)	39
Prestressed cable diameter (mm)	12.7
Design tensile strength of prestressed steel, F_{pu} (MPa)	1860
Initial tension per cable (kN)	138
Number of prestressing strands	56
Number of draped prestressing strands	12
Number of straight prestressing strands	44
Centroid of straight strands (mm)*	135.5
Centroid of draped strands at 2 m from girder ends (mm)*	1353.4
Centroid of draped strands at midspan (mm)*	190.0

*Measured from the bottom of the girder

Girders are cast directly into the abutments creating a rigid connection and eliminating any expansion joints. Abutment walls are approximately 4 m high and 15 m in length and are connected to wingwalls. Approach slabs are also provided and are rigidly connected to the abutment walls. Each abutment is supported by fifteen HP 310x132 steel piles oriented for weak-axis bending. Piles were driven in a pre-excavated trench and backfilled with loose granular material to increase flexibility. Such a construction is unique and deviates from the common practice of pre-drilling holes prior to driving integral-abutment foundation piles in stiff soils; however, due to ease of construction the method was approved by the NBDTI.

At the centre pier, bridge girders are supported on elastomeric bearing pads. The centre pier is supported by 22 HP310x132 battered steel piles oriented with either the weak axis or strong axis parallel to the longitudinal axis of the superstructure. Essentially, the intention is that the centre pier remains stationary while thermal expansion and contraction of the superstructure occurs at the abutment foundations. Approach slabs are provided and are rigidly connected to the abutment walls.

2.2 Instrumentation

While a variety of sensors have been installed on the Route 2 underpass, this paper is focussed on the data provided by the strain gauges installed in the bridge superstructure. A total of 40 Geokon model VCE-4200 vibrating-wire concrete embedment strain gauges were installed to measure the longitudinal strain in the bridge girders and deck. These gauges also include a thermistor allowing for a temperature measurement in addition to the sensor reading. Thirty gauges were installed in four girders at ten locations (Figure 1). At each location, strain gauges were installed at three different elevations as shown in Figure 2. An additional gauge was installed in the bridge deck above each of the ten locations. Installation of the gauges in the girders took place at the prestressing concrete plant, while the deck gauges were installed on site just prior to the deck pour.

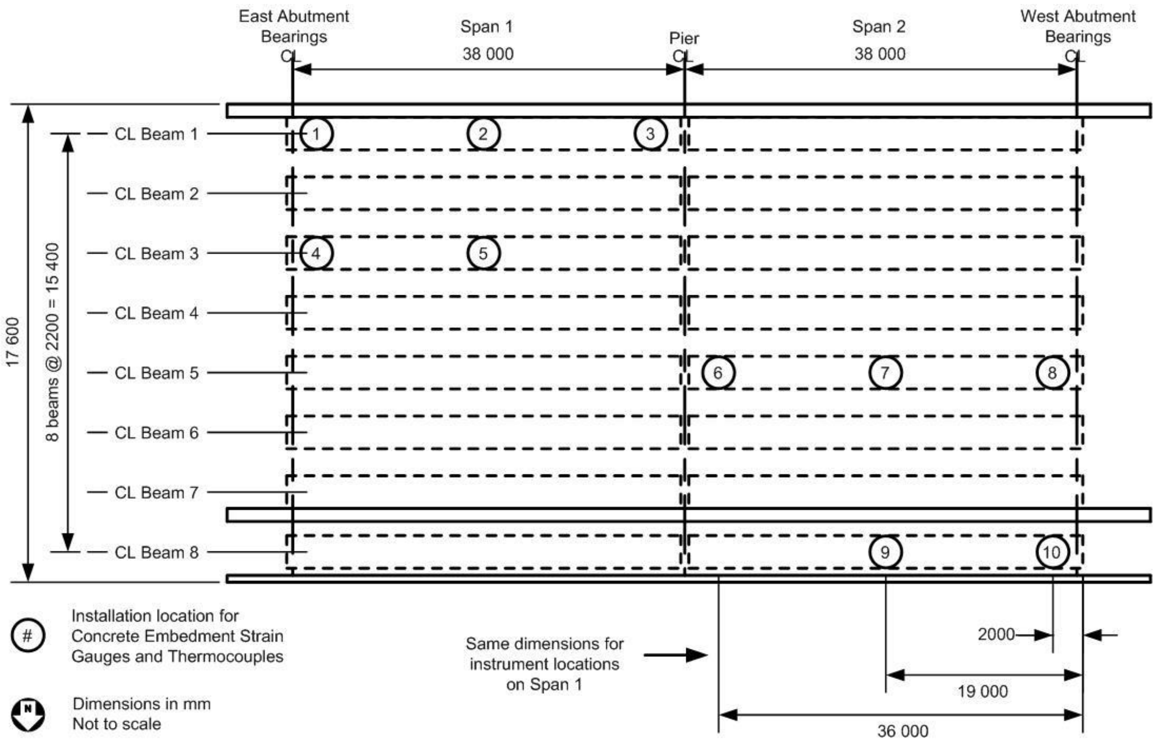

Figure 1: Instrumentation locations for the concrete strain gauges are denoted from 1-10.

Monitoring of the sensors began in October 2004 and is ongoing. In this paper, data are presented over a period of 8 years following construction. Data are collected three times every hour. Initial readings for the strain gauges were taken on a date following completion of the bridge so as to allow for the isolation of data that were generated by thermal changes rather than construction-related loading.

Installing the strain gauges at different elevations in the girders allows for the determination of the strain distribution across the depth. Fitting a line through the strain distribution gives the change in curvature from the initial reading (Equation 1). The change in moment of the girder-deck composite section can then be computed using Equation 2. Installation locations were selected to determine strains and moments at the ends and mid spans of both exterior and interior girders.

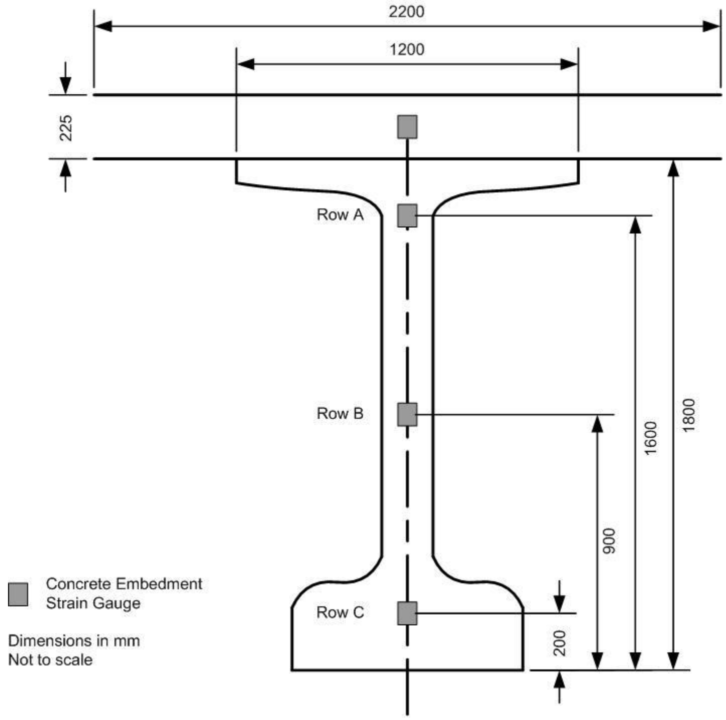

Figure 2: Cross-sectional view of the composite section showing the concrete strain gauge installation locations.

[1] $$\frac{1}{\rho} = -\tan\left(\frac{\Delta \text{ strain}}{\Delta \text{ height}}\right)$$

where $\frac{1}{\rho}$ = change in curvature in the composite (girder-deck) section (1/mm)

$\left(\frac{\Delta \text{ strain}}{\Delta \text{ height}}\right)$ = slope of best-fit line of strain readings at a given location

[2] $$\frac{1}{\rho} = \frac{M}{EI}$$

where M = change in moment in the composite section (N·mm)
E = modulus of elasticity of the concrete (N/mm^2)
I = moment of inertia of the composite section (mm^4)

3. RESULTS AND DISCUSSION

In addition to the moments resulting from dead and live loads, the thermal expansion and contraction of the superstructure also generates moments along the length of the bridge. Thermal bridge movements have been measured using deformation metres installed on the back face of each abutment. Two sensors were installed on each abutment at 1/3 and 2/3 of the abutment height. Field data from these sensors is shown in Figure 3 and anticipated moments and forces from thermal fluctuations are shown in Figure 4. Further information on the deformation metre instrumentation has been reported in Huntley and Valsangkar (2008, 2013).

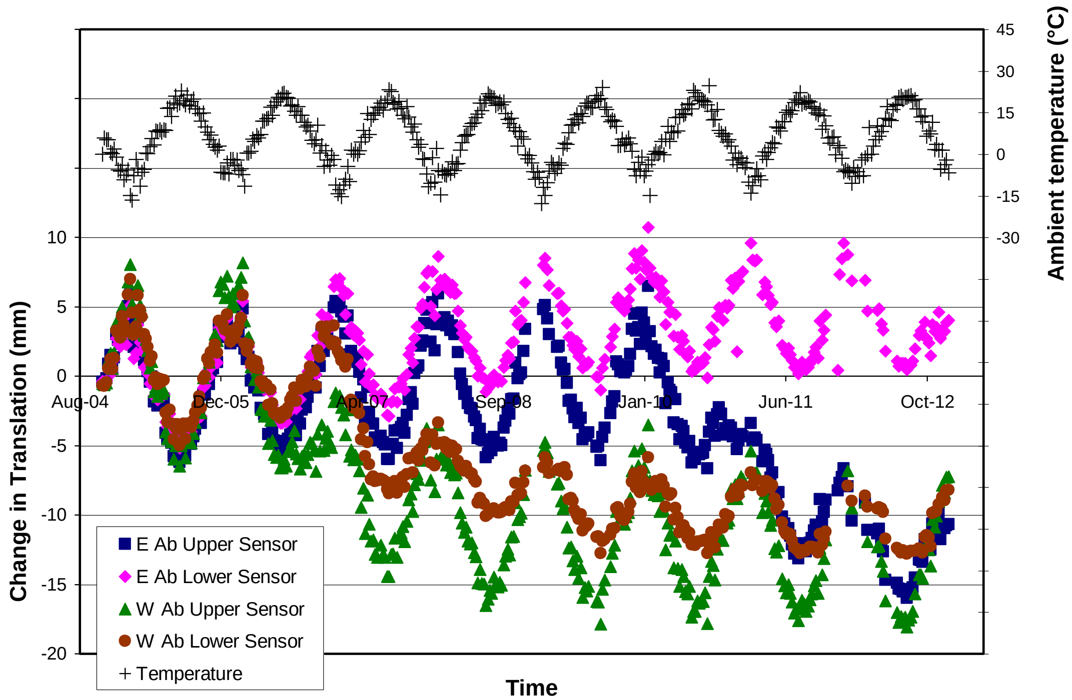

Figure 3: East and west abutment translation (E Ab and W Ab, respectively) and ambient temperature.

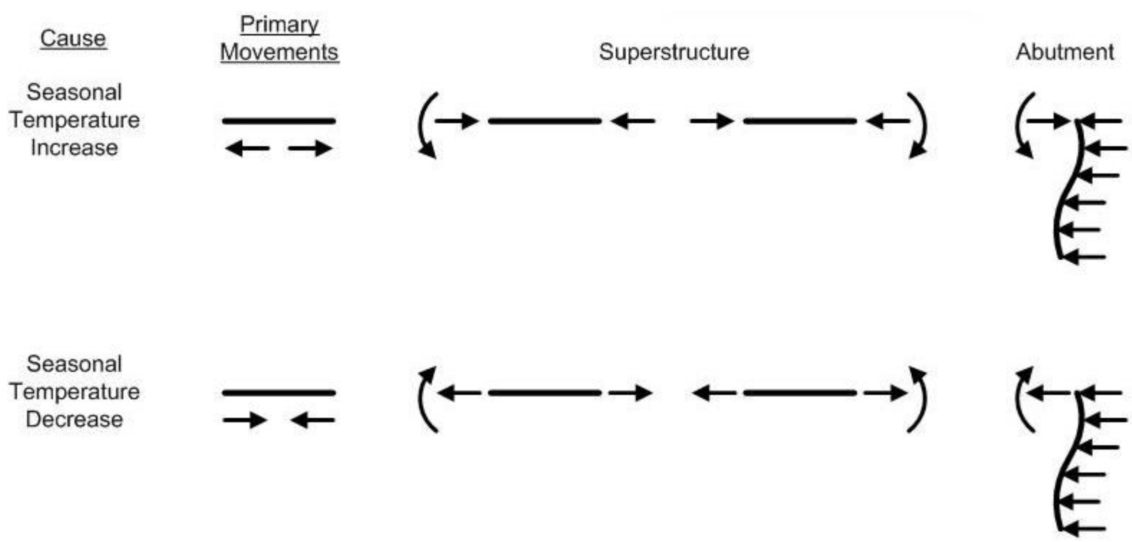

Figure 4: Forces and moments acting on the bridge superstructure due to seasonal thermal variations (after Russell and Gerken 1994).

In the summer, the superstructure expands and the abutments apply both a compressive force and negative moment to the bridge girders (top half of Figure 4). Given that the superstructure is similar to a continuous beam with a roller support in the middle, it is expected that near the abutments, the top of the superstructure will be in tension and the bottom will be in compression. Near the pier the opposite is expected with the top of the superstructure in compression and the bottom in tension. The reverse occurs in the winter when the superstructure contracts and the abutments apply a positive moment to the superstructure (bottom half of Figure 4).

Figures 5 – 9 show the variation in moment measured within the bridge-deck composite section at each of the 10 installation locations. The ambient temperature during this same time is also plotted. Each data point represents the average moment variation or ambient temperature during a one week period. Smaller moment variations were measured at the installation locations on the interior girders which is not unexpected given that the exterior girders would receive more sun exposure.

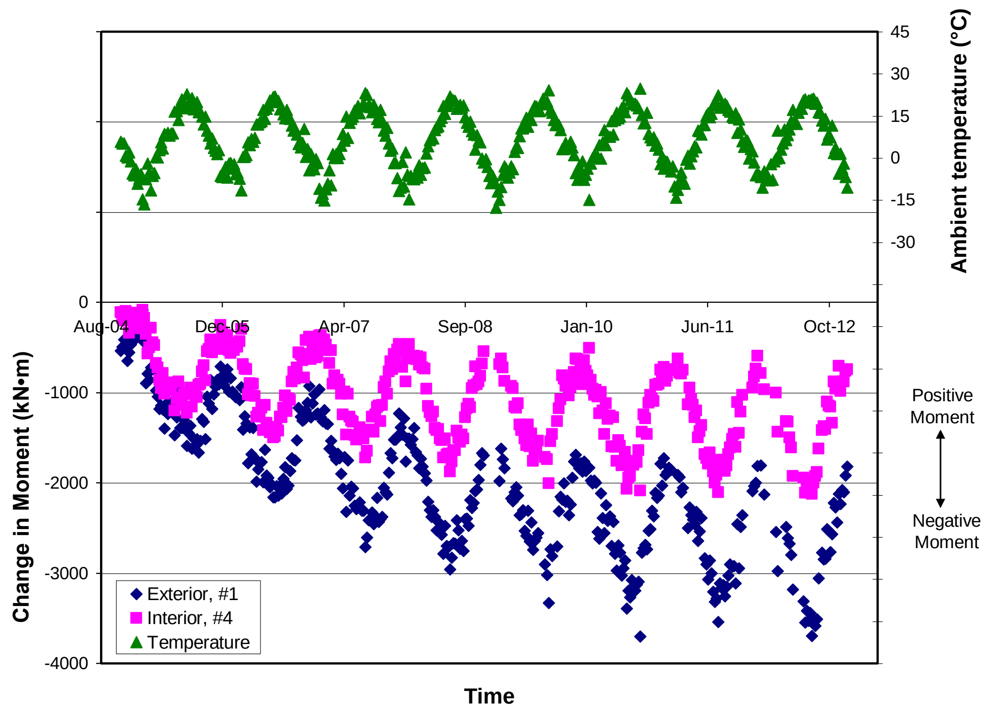

Figure 5: Moment response of the exterior and interior composite sections near the east abutment, specifically locations #1 and 4; the ambient temperature profile is also shown.

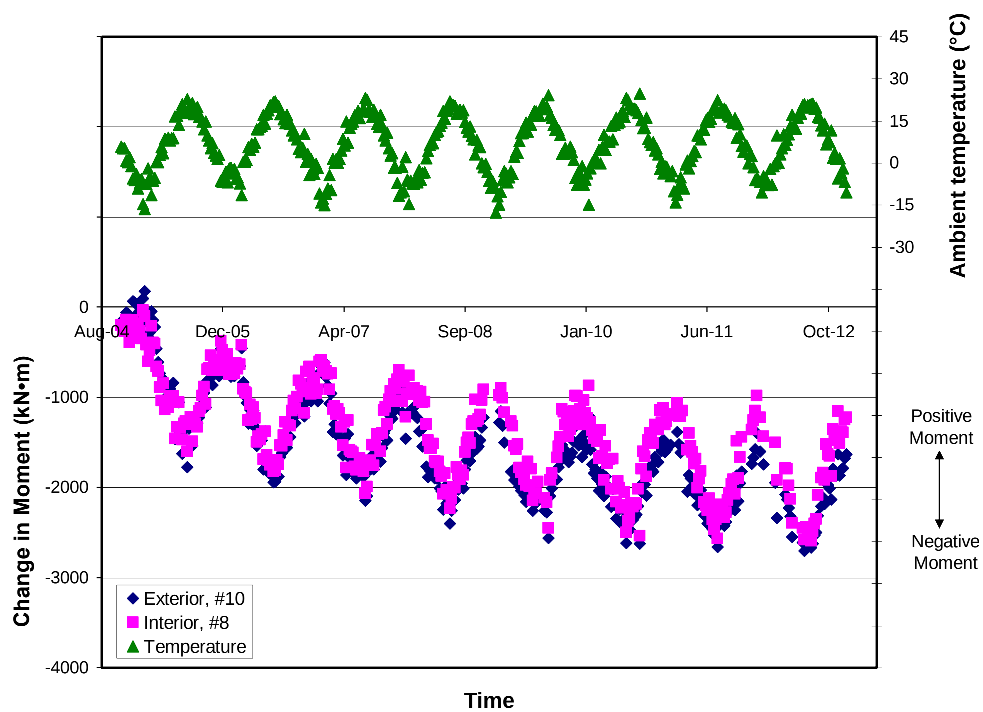

Figure 6: Moment response of the exterior and interior composite sections near the west abutment, specifically locations #10 and 8; the ambient temperature profile is also shown.

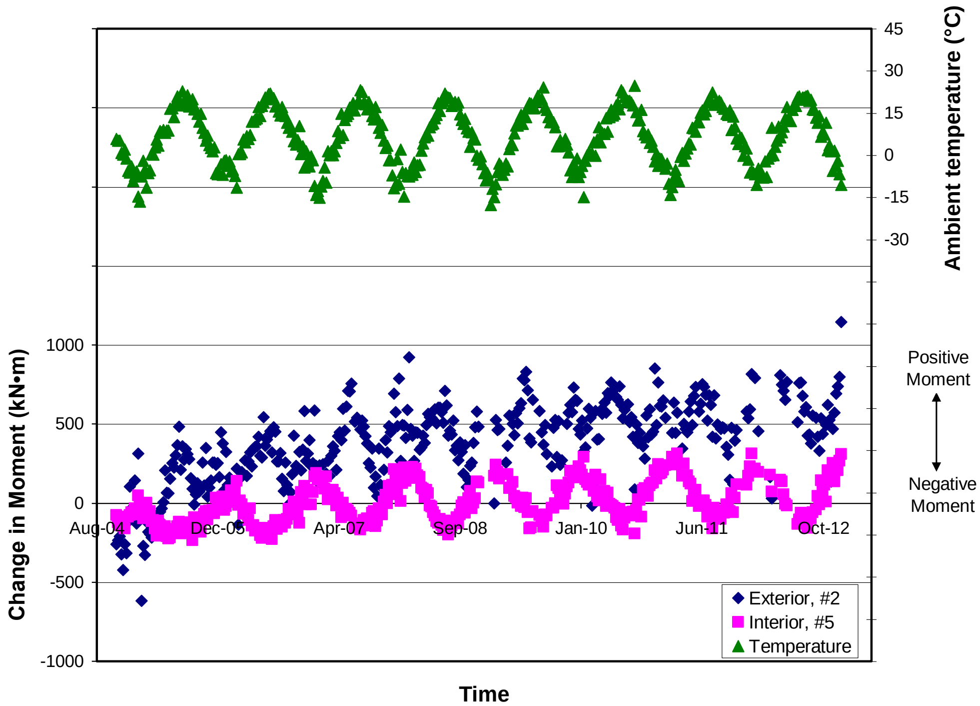

Figure 7: Moment response of the exterior and interior composite sections near the midspan of the east span, specifically locations #2 and 5; the ambient temperature profile is also shown.

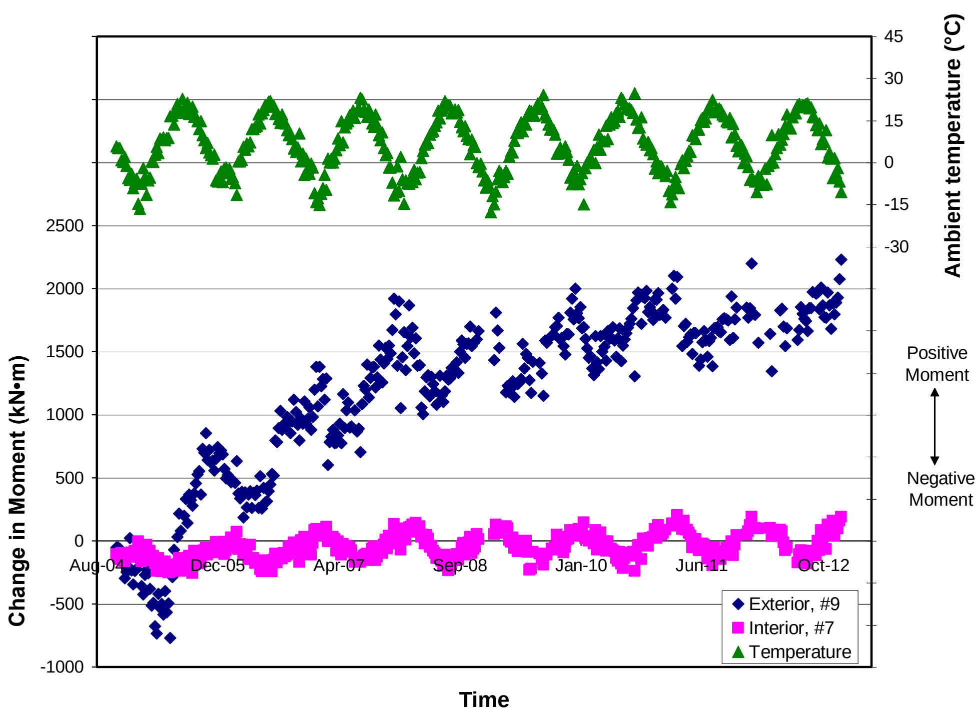

Figure 8: Moment response of the exterior and interior composite sections near the midspan of the west span, specifically locations #9 and 7; the ambient temperature profile is also shown.

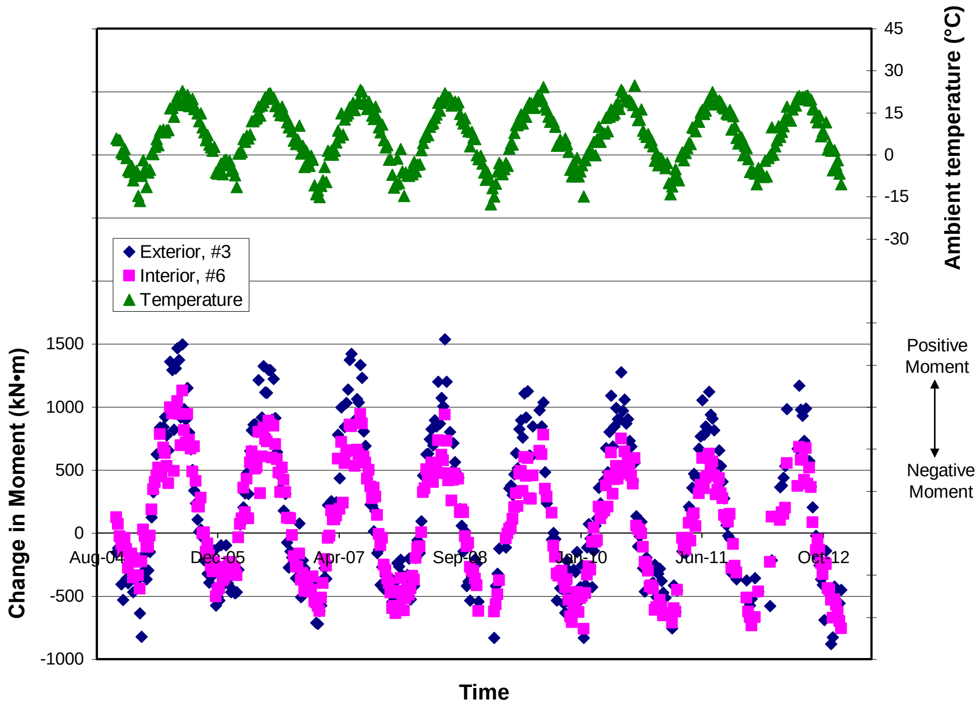

Figure 9: Moment response of the exterior and interior composite sections near the pier, specifically locations #3 and 6; the ambient temperature profile is also shown.

For the most part, the moment variations in the superstructure respond to thermal fluctuations as expected with the variations near the abutment and pier ends consistent with the anticipated behaviour outlined in Figure 4. The midspan moment variations are comparatively smaller in magnitude signifying that these instrumentation locations are near the area where the moment changes sign from positive to negative or vice versa.

The moment variations at the pier and at the midspan locations on the interior girders are relatively unchanged over time; however, those at the abutment appear to decrease (Figures 5 and 6). The apparent decrease at the abutment locations may in fact be residual moment in the superstructure resulting from abutment movements, i.e., permanent translation or rotation. The moment variations measured at the midspan locations on the exterior girders (#2 and #9) are unusual in that those at location #2 fluctuate more frequently and those at location # 9 appear to increase over time. There is no clear explanation for this behaviour at location #2 beyond the possibility of a malfunctioning strain gauge (s).

4. CONCLUSIONS

The primary focus of this paper was to report long term (8 years) factual data from strain gauges installed in the superstructure of an integral abutment bridge near Fredericton, N.B. Ten locations throughout the superstructure were instrumented. From the field data, it was determined that the superstructure responds to thermal fluctuations as expected with midspan moment variations being smaller in magnitude when compared to those observed at the abutment and pier ends. Furthermore, the magnitude of moment variations measured on the exterior girders was larger when compared to similar locations on the interior girders; this is consistent with the exterior girders receiving more sun exposure. In general, the magnitude of moment variation at each of the installation locations was unchanged throughout the study period; however, it appears that residual moments exist in the superstructure near the abutments. Detailed interpretation of the strain gauge data, in combination with data from sensors monitoring abutment movement, is required and is planned for presentation in a separate paper.

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REFERENCES

Barker, K..J. and Carder, D.R. 2001. Performance of an Integral Bridge Over the M1-A1 Link Road at Branham Crossroads. *Transportation Research Laboratory Report 521*, Transportation Research Laboratory, Wokingham, Berkshire, UK.

Civjan, S.A., Breña, S.F., Butler, D.A., and Crovo, D.S. 2004. Field Monitoring of Integral Abutment Bridge in Massachusetts. *Transportation Research Record*, 1892: 160–169.

Clayton, C.R.I., Xu, M., and Bloodworth, A. 2006. A Laboratory Study of the Development of Earth Pressure Behind Integral Bridge Abutments. *Géotechnique*, 56(8): 561–571.

Elgaaly, M., Sandford, T.C., and Colby, C. 1992. Testing an Integral Steel Frame Bridge. *Transportation Research Record*, 1371: 75–82.

Fennema, J.L., Laman, J.A., and Linzell, D.G. 2005. Predicted and Measured Response of an Integral Abutment Bridge. *Journal of Bridge Engineering*, *ASCE*, 10(6): 666–677.

Hassiotis, S., Khodair, Y., and Wallace, L.F. 2005. Data from Full-Scale Testing of Integral Abutment Bridge. *Transportation Research Board 84th Annual Meeting*, Transportation Research Board, Washington, D.C., United States of America, pp. 1–15.

Huang, J., Shield, C.K., and French, C. 2005. Time-Dependent Behaviour of a Concrete Integral Abutment Bridge. *6th International Bridge Engineering Conference on Reliability, Security, and Sustainability in Bridge Engineering*, Transportation Research Board, Boston, Massachusetts, United States of America, pp. 299–309.

Huntley, S.A. and Valsangkar, A.J. 2008. Soil Deformations Behind an Integral Abutment Bridge. *Second International British Geotechnical Association Conference on Foundations*, IHS BRE Press, Dundee, Scotland, United Kingdom, pp. 1037–1048.

Huntley, S.A. and Valsangkar, A.J. 2013. Field Monitoring of Earth Pressures on Integral Bridge Abutments. *Canadian Geotechnical Journal*, 50(8): 841–857.

Huntley, S.A. and Valsangkar, A.J. 2014. Behaviour of H-piles Supporting an Integral Abutment Bridge. *Canadian Geotechnical Journal*, 51(7): 713-734.

Husain, I. and Bagnariol, D. 1996. Integral Abutment Bridges. *Structural Office Report SO-96-01*, Structural Office, Ministry of Transportation, Ontario, Canada.

Kim, W. and Laman, J.A. 2012. Seven-Year Field Monitoring of Four Integral Abutment Bridges. *Journal of Performance of Constructed Facilities*, *ASCE*, 26(1): 54-64.

Kunin, J. and Alampalli, S. 2000. Integral Abutment Bridges: Current Practice in United States and Canada. *Journal of Performance of Constructed Facilities*, *ASCE*, 14(3): 104–111.

Lawver, A., French, C., and Shield, C.K. 2000. Field Performance of Integral Abutment Bridge. *Transportation Research Record*, 1740: 108–117.

Nikravan, N. 2013. *Structural Design Issues for Integral Abutment Bridges*. Ph.D. thesis, Department of Civil Engineering, Ryerson University, Toronto, Ontario, Canada.

Ooi, P.S.K. and Lin, X. 2006. Field Behavior of an Instrumented Integral Abutment Bridge in Hawaii. *Transportation Research Board 85th Annual Meeting*, Transportation Research Board, Washington, D.C., United States of America, pp. 1–26.

Russell, H.G. and Gerken, L.J. 1994. Jointless Bridges – The Knowns and the Unknowns. *Concrete International*, 16(4): 44–48.

Sandford, T.C. and Elgaaly, M. 1993. Skew Effects on Backfill Pressures at Frame Bridge Abutments. *Transportation Research Record*, 1415: 1–11.

Shoukry, S.N., William, G.W., Riad, M.Y., and McBride, K.C. 2006. Field Monitoring and 3D FE Modeling of an Integral Abutment Bridge in West Virginia *Transportation Research Board 85th Annual Meeting*, Transportation Research Board, Washington, D.C., United States of America, pp. 1–15.