INTEGRAL ABUTMENT DATA FROM THREE STEEL GIRDER BRIDGES

SCOTT CIVJAN

BROOKE QUINN

SERGIO BRENA

(no photo available)

CHAD ALLEN

BIOGRAPHY

Scott Civjan is an Associate Professor at the University of Massachusetts at Amherst (UMass). He has been instrumenting and researching bridges for 15 years. The UMass Bridge Monitoring Program has collected in-situ field data from 9 bridges. His research background includes experimental testing, field instrumentation, analytical modeling and implementation projects.

Brooke Quinn is a Ph.D. Candidate in Structural Engineering and Mechanics at UMass Amherst. Her research is focused on integral abutment bridges.

Sergio Brena is an Associate Professor at UMass Amherst and member of the UMass Bridge Monitoring Program.

Chad Allen is a Geotechnical Engineer with the Vermont Agency of Transportation. As part of the Research and Materials group he was involved in research projects related to Integral Abutment Bridges and the Technical Representative for this project.

SUMMARY

Long-term data during approximately 4 years of monitoring are presented for three Integral Abutment Bridges (IABs) of increasing complexity. A straight-girder non-skew bridge, a straight girder bridge with a 15 degree skew angle, and a two-span curved girder (11.25 degree curvature) bridge were instrumented to monitor strains, pile and abutment rotations, backfill pressures, and global deformations. Three-dimensional finite element models were created for and used for comparison to field data. Data are presented to illustrate the variability of seasonal response as well as the stabilization of the bridges in latter seasons that could be considered to represent the steady state response. Differences between field behavior and finite element model assumptions are addressed.

All three bridges had acceptable performance which supports the use of steel girder integral abutment bridges for moderate span structures. Data suggests that existing restrictions many states impose on steel girder IABs of moderate curvature should be relaxed, which would broaden the application of these efficient and cost effective bridges.

Integral Abutment Bridge Data from Three Steel Girder Bridges

Three steel girder integral abutment bridges (IABs) of increasing complexity were instrumented during construction and monitored since November, 2009. The bridges were designed by the Vermont Agency of Transportation who sponsored the four year monitoring research program. The University of Massachusetts Amherst developed the instrumentation program, installed the gages and conducted the research program including data collection and evaluation. In this paper the long term data of bridge response to thermal effects is described.

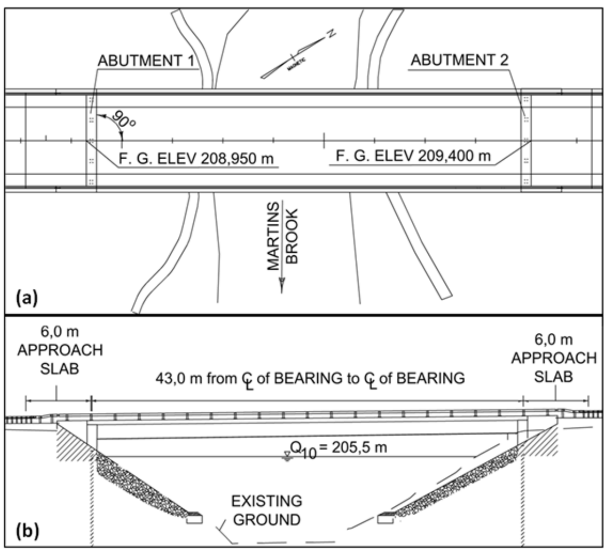


Figure 1 Middlesex Bridge

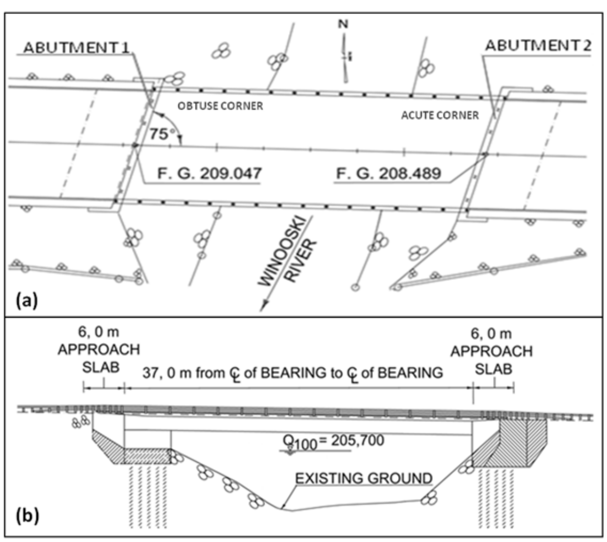


Figure 2 East Montpelier Bridge

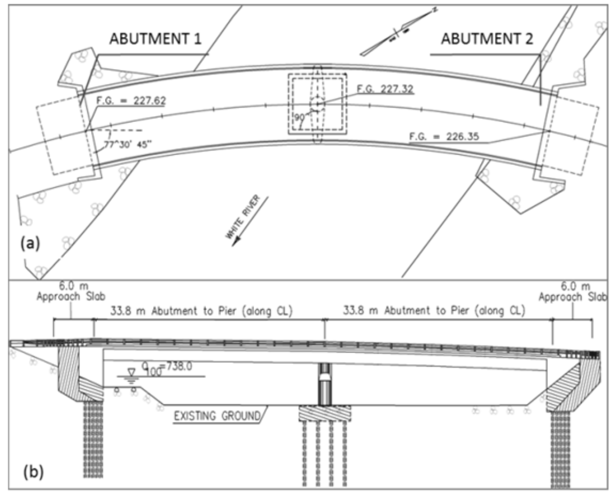


Figure 3 Stockbridge Bridge

The monitored bridges include a straight girder non-skew bridge of 140 ft span and 33.5 ft width (Figure 1), a straight girder bridge with skew of 15 degrees of 120 ft span and 46.6 ft width (Figure 2), and a two-span curved girder (11.25 degree curvature) bridge of 222 ft span and 37.0 ft width (Figure 3). All piles are oriented with their weak axis bending corresponding to deformation perpendicular to the abutments. It is noted that the straight bridge represents a standard IAB design for VTrans, the skew in the second structure is within design limits (but states often limit skew of these structures to 20 or 30 degrees) and the curved bridge is not allowed by VTrans but was constructed specifically to evaluate behavior of a curved IAB. Stockbridge is also notable as having geofoam material placed behind the abutment with the intent of minimizing backfill pressures.

These bridges were instrumented with 83, 89 and 131 gages, respectively. Instrumentation was concentrated at the abutments and details on the instrumentation program have been reported elsewhere[1]. Gages were provided to evaluate strains (in piles, girders and pier), pile and abutment rotations, backfill pressures and overall bridge displacements. Each gage included an internal thermistor. Three dimensional (3-D) finite element (FE) models were developed in SAP-2000 that include non-linear geometric, material and soil effects. Many parametric FE models have been evaluated as part of the project; space does not permit a description of the modeling assumptions. However, details of the FE modeling have been previously reported[2, 3]. Comparisons to these models are shown in several plots to provide reference points of "expected" behavior versus the field data.

Overall Bridge Movements

All structures experienced a temperature fluctuation of approximately 100 °F. Using the simple thermal expansion equation:

$$\delta = \alpha L \Delta T \qquad (1)$$

where L = bridge span length; α = coefficient of thermal expansion of superstructure= 0.0000117/°C (0.0000065/°F); and ΔT = temperature change. Therefore, total displacement (combination of both abutment movements) over the year is expected to be on the order of 1.1, 1.0 and 1.7 in for the Middlesex, East Montpelier and Stockbridge Bridges, respectively. The actual ambient temperature fluctuations of each bridge (averaged from shaded gages on the bottom beam flanges) are shown in Figure 4.

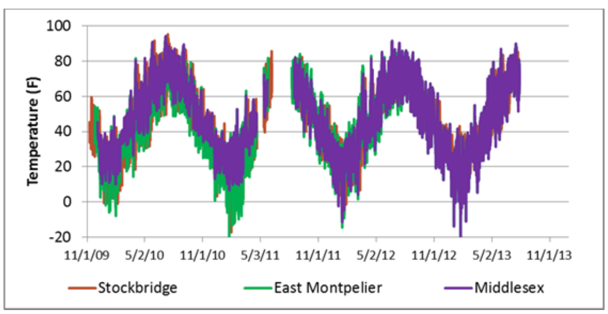
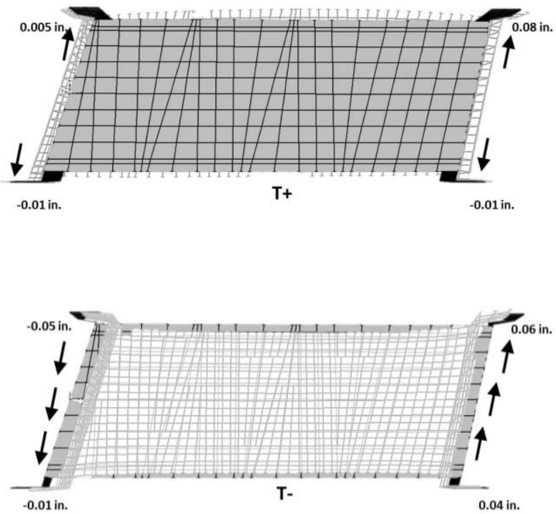


Figure 4 Ambient bridge temperature

From FE modeling, it is expected that the straight bridge would exhibit longitudinal movement of the structure, while the skewed structure would show some twist under thermal expansion and contraction and the curved bridge would twist about the center pier as shown in Figure 5. Actual longitudinal and transverse displacements observed in the bridges at the top of the abutments are shown in Figure 6 and Figure 6. The seasonal expansion and contraction of these bridges is clearly seen in the data, but it is noted that the field data exhibits permanent drift of the abutments. The bridges exhibit a lengthening with time, which is likely due to deck cracks being filled with debris over time and not fully closing.

(a)



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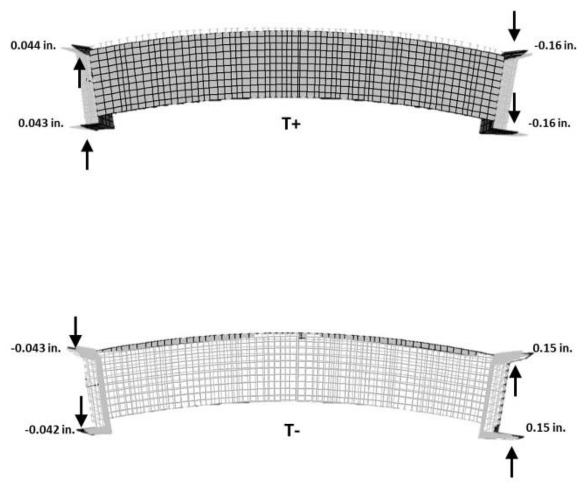
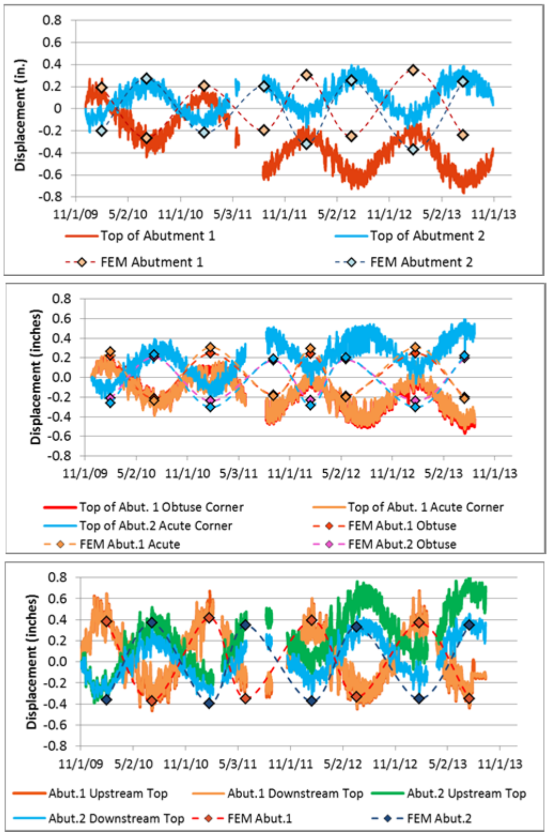


Figure 5 Expected global deformations (a) East Montpelier (b) Stockbridge

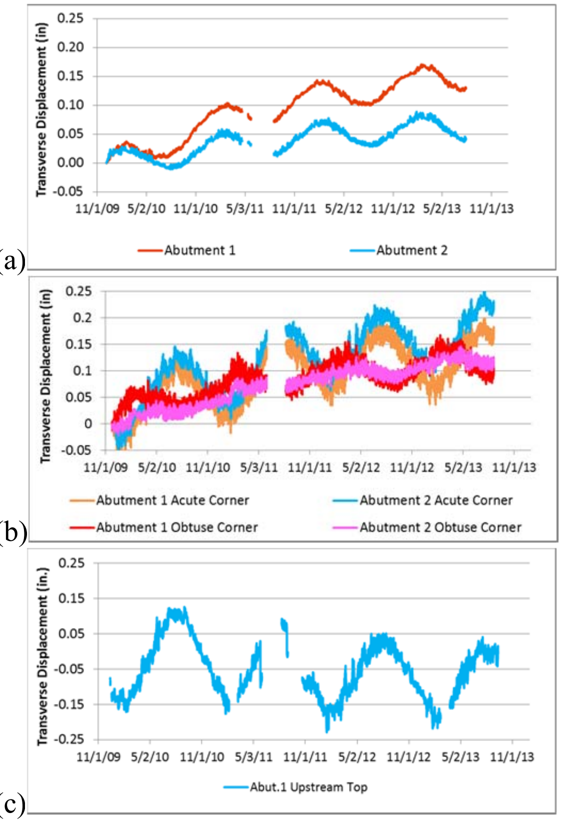


(a) (b) (c)

Figure 6 Top of abutment longitudinal displacement (field data and FEM) (a) Middlesex (b) East Montpelier (c) Stockbridge

Transverse displacements of the skew structure agree with FE results where most movement is concentrated at the acute corners, though it is noted that both the straight and skew bridge exhibit a lateral movement each year that is not recovered. The skew angle does not significantly affect the transverse displacements, as they were comparable to those observed in the straight bridge which theoretically would not experience transverse displacements.

It is noted that FE modeling of the bridges is reasonably accurate for longitudinal displacements in the first year, but cannot account for the drift. The more complicated curved structure exhibits much more stable measurements over time and is matched well by FE models.

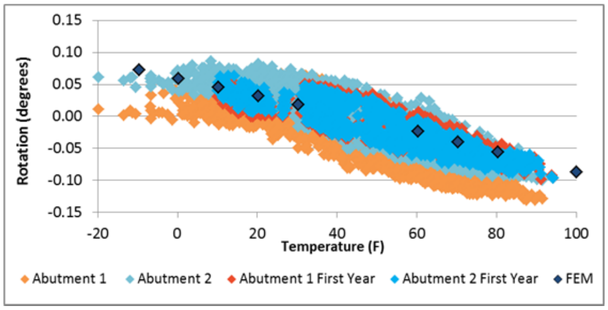


(a) (b) (c)

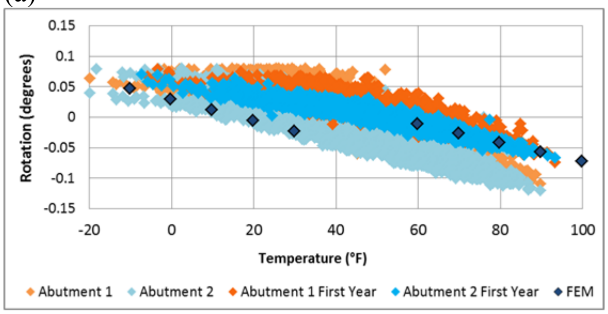
Figure 7 Top of abutment transverse displacement (field data and FEM) (a) Middlesex (b) East Montpelier (c) Stockbridge

Due to the thermal load on the structures, the abutment rotates as well as translates to account for superstructure expansion and contraction. Field data and FE results for the abutment rotation are shown in Figure 8. This rotation has two effects on steel components of the bridge. First, the longitudinal displacements at that bottom of the abutment are much lower than at the top of the abutment, thereby significantly reducing the lateral deformation required of the piles. Deformation of the substructure (abutment and piles) under maximum yearly temperatures is shown in Figure 9and Figure 10. Several years of data are shown, as will be discussed later, along with FE results (dashed lines). Second, the effect of the end restraint is to provide a moment at the end of the superstructure (which is constant along the length of the single span structures, variable in the two-span bridge) and possibly axial load in the superstructure. These effects were not significant, with a maximum girder strain reading due to thermal effects of 219.7 με, 192.4 με, and 202.1

$\mu\varepsilon$ in Middlesex, East Montpelier and Stockbridge, respectively.



(a)



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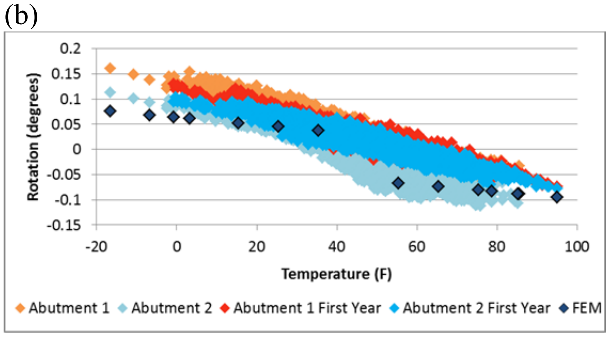


(c)

Figure 8 Abutment rotations with temperature (a) Middlesex (b) East Montpelier (c) Stockbridge

The substructure responses shown in Figure 9 and Figure 10 merit further discussion; Subsequent years show a movement of the Middlesex and East Montpelier bridges towards that backfill, while the Stockbridge Bridge has more consistent behavior. These figures also clearly show the abutment rotation and resulting deformation of the piles. Initial FE results did not produce the expected top of abutment deformations, though they matched reasonably to the data in Figure 6. The substructure response is very complicated, with maximum deformations and pile moments not necessarily occurring at times of maximum temperatures. Details of findings on this topic have been reported elsewhere[4]. However, in the current evaluations the temperature variation in the FE has been modified to match the field deformations at the top of the abutment in FE Matched results. It is seen that while the FE Matched results capture the overall substructure behavior for the Stockbridge bridge, the Middlesex and East Montpelier bridges still show that there is less soil restraint in the field than would be predicted by the model even when the top of abutment displacement is matched. Both of these results are reported in the Pile Moment section results.

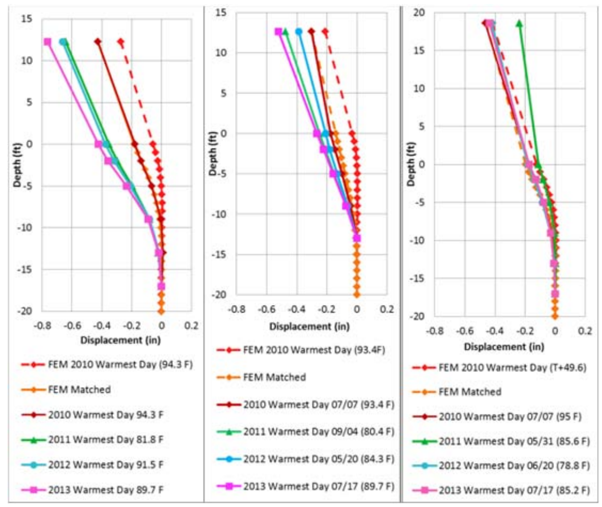


Figure 9 Substructure deformations under maximum yearly temperatures (bridge expansion).

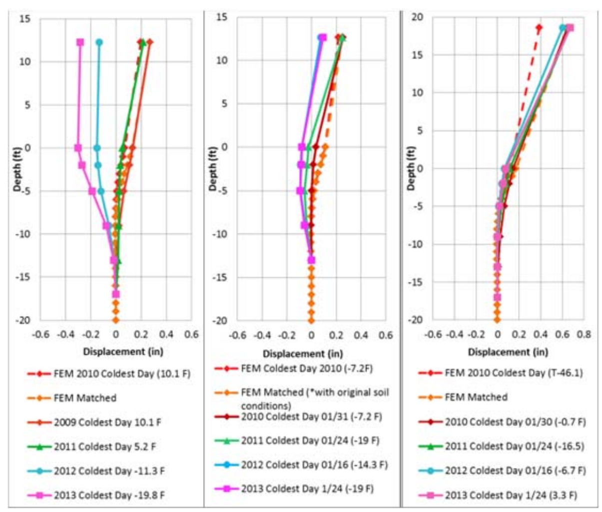


Figure 10 Substructure deformations under minimum yearly temperatures (bridge contraction).

Pile Moments

The deformations previously discussed result in moments in the piles, which were oriented with their weak axis resisting longitudinal movements of the bridge superstructure. Stockbridge results are not presented, as only 3 gages were available per gage location on the piles, so extracting the biaxial moments was not possible. The top instrumented location (20 inches below the bottom of the abutment) reported higher pile moment values than other instrumented locations below this location on the pile, and these values are shown in the following plots.

Weak axis moments at the top instrumented location are shown for all instrumented piles in Figure 11. In the first year the Middlesex weak axis moments cycle as expected from the FE models and first year deflected shapes (Figure 9 and Figure 10). However, as the contraction deflected shape shifts each year, at minimum temperature the piles do not fully contract which results in a change in sign of the maximum moment. The East Montpelier bridge weak axis pile moments have shown minimal seasonal fluctuations at the gage location.

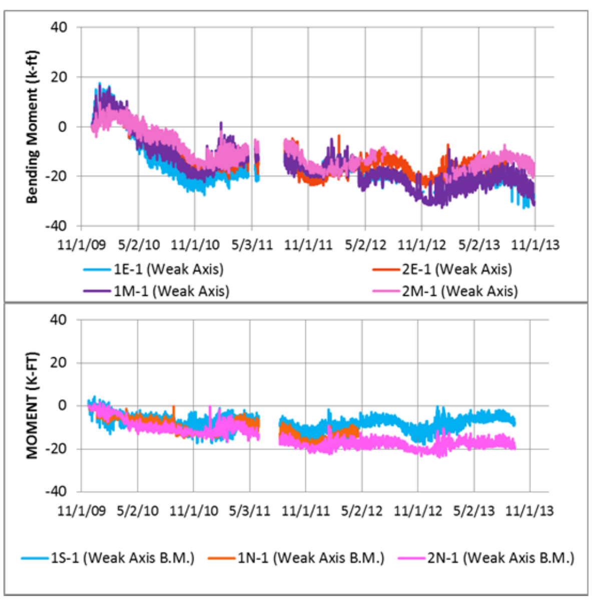


Figure 11 Pile weak axis (top) Middlesex (bottom) East Montpelier

Strong axis moments at the top instrumented location are shown for all instrumented piles in Figure 12. As would be expected, the straight Middlesex bridge strong axis moments (transverse to bridge) are much smaller than the weak axis values and show some seasonal variation in line with the transverse displacements shown in Figure 7. The East Montpelier bridge strong axis pile moments are much higher in value than the weak axis values at the instrumented location, indicating the resistance to transverse movements and bridge twisting indicated in Figure 5 and Figure 7.

FE modeling has shed some light on the moments reported in the data at this instrumented location. As seen in Figure 13 for the FE model using original soil conditions from soil boring logs (“FE Nominal”), there is a significant increase in moment at the pile/abutment interface from the instrumented location. When the FE Matched model is considered (to match the actual displacement at the top of the abutment as well as adjusted soil conditions to match field data substructure displacement) the increase in moment is greater and the maximum moment increased up to -30.1 kip-ft.

Pile moments are summarized in **Error! Reference source not found.** and Table 2. Piles at the Middlesex and East Montpelier bridges are HP 12X84 (weak axis M_y=144 kip-ft, strong axis M_y=442 kip-ft). Piles at the Stockbridge bridge are HP14X117 (weak axis M_y=248 kip-ft, strong axis M_y=716 kip-ft). It is therefore noted that pile yielding was not expected, nor indicated by field data, due to thermal load effects. Maximum strains reported from field data during long term monitoring have been 219.8 με, 216.8 με, and 157.7 με for the Middlesex, East Montpelier and Stockbridge bridges, respectively. These are all well below the nominal yield strain of 1725 με.

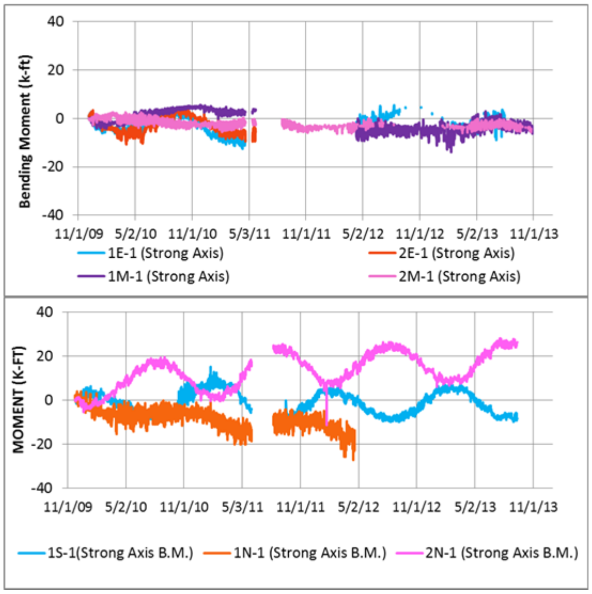


Figure 12 Strong axis moments (top) Middlesex (bottom) East Montpelier

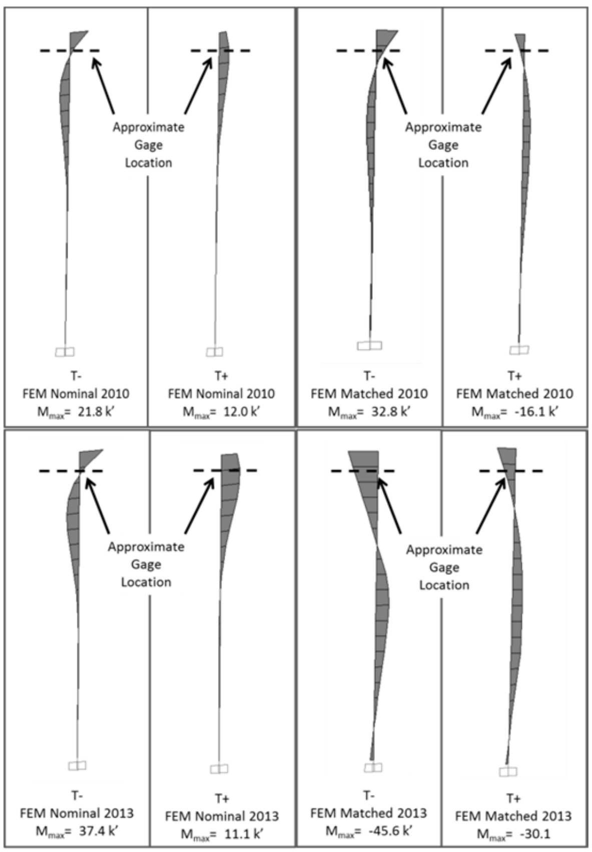


Figure 13 Pile weak axis moments for Middlesex Bridge FE Nominal and FE Matched for 2010 (top) and 2013 (bottom)

Table 1 Weak axis pile bending moments

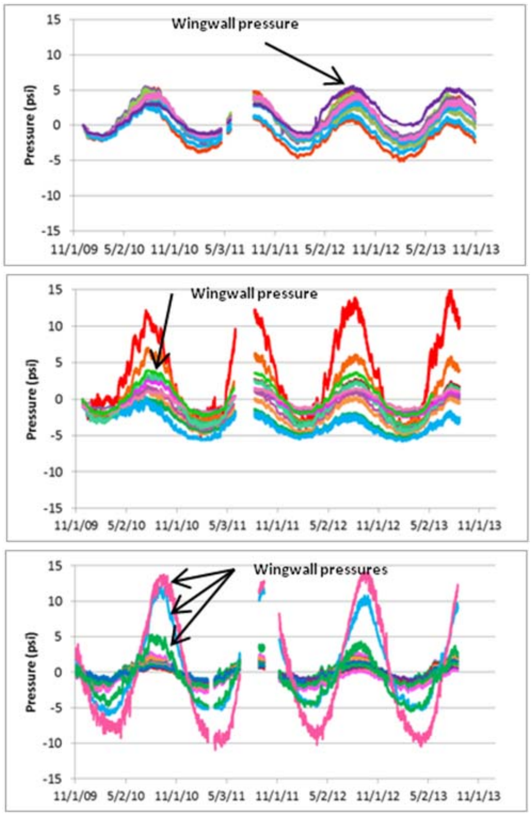
Weak Axis Bending Moment (kip-ft)				
		MI	EM	ST
Field Moment at Peak Temp. First Year	T+	-2.3	-6.4	N/A
	T-	12.6	-13.0	N/A
Max. Field Moment	T+	-18.9	-20.3	N/A
	T-	17.5	-24.1	N/A
FE Max. Moment	T+	12.0	9.5	-49.3
	T-	37.4	63.4	45.8
FE Matched Max Moment	T+	-30.1	-7.5	-51.8
	T-	-45.6	71.9	60.1

Table 2 Strong axis pile moments

Strong Axis Bending Moment (kip-ft)				
		MI	EM	ST
Field Moment at Peak Temp. First Year	T+	-5.1	18.6	N/A
	T-	-2.3	-8.7	N/A
Max. Field Moment	T+	4.9	27.9	N/A
	T-	-13.1	-25.3	N/A
FE Max. Moment	T+	3	51.9	-6.1
	T-	-1.8	-36.6	5.7
FE Matched Max Moment	T+	4.9	38.7	-14.2
	T-	-2.7	-46.7	17.6

Earth Pressures

Earth pressure cells were placed behind the abutment and wingwalls at several locations across the abutment in the bottom half of the abutment in Middlesex and East Montpelier and in the bottom 2/3 of the abutment in Stockbridge, as well as individual locations on the wingwalls. This data is shown in Figure 14.



(a)
(b)
(c)

Figure 14 Earth pressures over time (a) Middlesex (b) East Montpelier (c) Stockbridge

From this data several important factors can be observed. Despite the fact that deflection has increased into the backfill in subsequent years (see Figure 6), the maximum soil pressures have seen minimal increases. This indicates that soil ratcheting effects (increasing soil pressure under cycling at a constant wall displacement), which have led many DOTs to design their abutments for full passive pressures, are not occurring in these moderate span structures. The Middlesex pressures are very consistent throughout the abutment, whereas the skewed East Montpelier bridge had highly variable earth pressures across the abutment, with one location (at the acute corner near the wingwall) having significantly higher pressures than all other instrumented locations. Stockbridge, in contrast, had much lower abutment pressures despite its much longer span. This shows the effectiveness of the geofoam material in minimizing the abutment pressures. However, wingwall pressures, which resist twisting of the bridge and where no geofoam was provided, are much higher. At the skewed East Montpelier Bridge, wingwall pressures were higher than abutment pressures with the exception of the obtuse corner of Abutment 1. At both of these bridges a crack has formed at the abutment/wingwall interface, though the wingwall pressures did not dissipate after this occurred.

Summary and Conclusions

Four years of data from long-term monitoring of three IABs in Vermont have been presented. Data were presented illustrating differences in seasonal response between the initial and latter years. Finite Element results are also included and compared to actual behavior.

In the first year displacements at the top of the abutments of all three structures are similar to those expected of a non-skewed straight girder bridge. Piles restrained transverse deformations of the skewed and curved bridges, generating strong axis bending moments in the piles. In latter years, a shift in the abutment and pile deformations was apparent which was not predicted from analysis. Abutment deformations included a combination of translational and rotational response of each abutment, with abutment rotation decreasing weak axis moments induced in the piles. A lag in the response of the pile deformation was also noted, resulting in maximum pile moments that did not align with times of extreme temperatures. Combined stresses due to axial, weak axis bending and strong axis bending have not provided any indication of pile yielding. In the two single-span structures of moderate length, soil ratcheting was not observed. Geofoam material was installed behind the abutment of the curved bridge and was effective at minimizing soil pressures and resulted in the most consistent seasonal response of the three bridges.

Field data and FE results indicate that the skew and curved bridge designs are performing very well. Pile strains have shown no signs of yielding, backfill pressures are not increasing and deck and approaches are in good condition. Variations in response are not significantly different from the straight bridge. In fact, the repeatability of response in the curved structure was much more predictable and consistent than the other two structures.

The acceptable performance of these three bridges supports the use of steel IABs as a structure of choice for moderate span structures. The existing restrictions many states impose on steel girder integral abutment bridges of moderate curvature should be relaxed, broadening the application of these efficient and cost effective bridges. Pile and abutment designs in these bridges are areas where improvements in efficiency should be pursued.

Acknowledgements

The research reported was prepared in cooperation with the Vermont Agency of Transportation. Conclusions and recommendations contained herein are based upon the research data obtained and the expertise of the researchers, and are not necessarily to be construed as Agency views or policy. The Vermont Agency of Transportation assumes no liability for its contents or the use thereof.

References

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