



Transportation Research Division



Technical Report 15-01

Bridge-in-a-Backpack™

Task 4: Monitoring and Load Rating

Final Report – Task 4, January 2015

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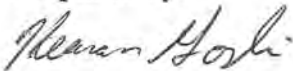
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BRIDGE-IN-A-BACKPACK™

Five bridges were constructed utilizing UMaine's hybrid composite arch technology as part of the Maine Department of Transportation's Composites Initiative. Three were constructed in 2010, one in 2011 and the final one in 2012. The designs for each of the first four were completed in 2010 and the final design of the Grey's Brook Bridge in Ellsworth, Maine (2012 construction) was completed in 2011 using lessons learned from the construction of the 2010 bridges. Highlights of the construction process, lessons learned, inspection observations, and load testing are presented in this report for the first four bridges (2010 and 2011 seasons).

Task 4.1 Construction report

UMaine provides a summary of design and construction details for the bridges included as a means to learn from these design and construction decisions. UMaine had varying levels of involvement on each site. The Royal River Bridge had very little involvement in comparison to the Perkins Bridge where UMaine was on site for a considerable amount of time. UMaine was onsite for the Jenkins Bridge a moderate amount of time. This reflected the level of instrumentation and data collection for Task 4.2 where the Perkins Bridge required heavy instrumentation and data collection.

The Royal River Bridge was the first bridge constructed as part of this effort in 2010. This bridge has an arch span of 38ft and rise of 9ft-6in. It is pile-supported with a minimum cover at the crown of approximately 4ft including wearing surface. A combination of cast-in-place leveling slabs and modular precast concrete retaining wall was used for the headwalls.

The Perkins Bridge in Belfast, Maine is a 47'-7" span buried composite arch bridge. This bridge carries traffic over the Little River and was constructed primarily in the late summer and fall of 2010. Final paving and landscaping were completed in the spring of 2011. Sixteen (16) hybrid concrete filled FRP tubes were cast into the foundations that were cast against bedrock. Lightweight FRP corrugated decking spans between the arches and supported the concrete deck as shown in Figure 1, Figure 2, and Figure 3. A combination of cast-in-place leveling slabs and modular precast concrete retaining wall was used for the headwalls.

The Jenkins Bridge was constructed in the summer and fall of 2010. This bridge is a pile supported 28ft 6inch span bridge with a skew of 19 degrees. Unlike the Royal River Bridge or the Perkins Bridge, the Jenkins Bridge in Bradley had a hybrid FRP headwall. The headwall is a concrete filled pultruded FRP panel that is self-reacting with a steel frame and tie rods. The base of the steel frame behind each wall is supported at the foundation as seen in Figure 4.



Figure 1: Concrete placement at base of headwall (Perkins Bridge)



Figure 2: Rebar mat allowed for walking during construction



Figure 3: Details of rebar at base of headwall



Figure 4: Headwall frame construction at Jenkins Bridge

Lessons Learned

Concrete filled FRP tubular arch bridges have seen increased use. Building on the Neal Bridge construction modifications were made to the design to increase efficiency and make it easier to construct. Neal Bridge opportunities for improvement were collected during construction and documented with a meeting post construction.

Following the construction of the Royal River Bridge, Perkins Bridge, and Jenkins Bridge a similar meeting was held that included UMaine, the Maine DoT, and contractors involved with all the bridges. A conscious effort was made to document areas where constructability could be improved, costs could be reduced due to conservatism in design or unnecessary redundancy, and improvements to durability could be made. Notes were made during construction site visits. The meetings focused on recommendations for improvement in structural portions of the designs where small or large changes could be made. These areas included the abutments, FRP arches, arch fill concrete, composite decking, concrete deck slab, and headwalls. Also noted were any general design recommendations or constructability recommendations. A compilation of lessons learned is presented for these three bridges constructed in 2010 (Auburn, Belfast, and Bradley). Complete meeting notes are provided in Appendix A.

Task 4.2 Load testing of Perkins Bridge and Jenkins Bridge

The Jenkins Bridge in Bradley, Maine and the Perkins Bridge in Belfast, Maine were live load tested to investigate the construction and live load effects on the bridges. Live load testing of the Perkins Bridge and the Jenkins Bridge took place in the fall of 2010 with an additional live load test of the Perkins Bridge in the summer of 2013. The results of the final tests are described here. Recommendations resulting from this testing and monitoring are included in section 4.5.

4.2.1 Perkins Bridge Construction Monitoring and Live Load Testing

The Perkins Bridge was instrumented to monitor construction load effects and live load effects on the bridge. Students and staff from the UMaine Composites Center were on site to collect data through the construction of the Perkins Bridge superstructure.

Construction Monitoring

Data was collected during significant steps in the construction process including the filling of hollow tubular FRP arches with self-consolidating concrete, deck placement, and backfilling. The data included strain in the FRP tubes, vertical and horizontal deflections of discrete points in selected arches and soil pressures behind the decking. Greater details of the instrumentation and data are available in Walton 2011. Data was collected for all sensors throughout the construction and some data were not usable due to excessive noise, unreadable data levels, or other data collection problems. Clear data is presented for hollow tube fill as well as the soil pressure data up to Elevation 52 feet. The final grade at the bridge centerline is approximately 57.96 feet. The bridge can be seen during backfilling in Figure 5.



Figure 5: Perkins Bridge prior to backfilling of the superstructure

Strain gages and string potentiometers were attached to selected arches under the bridge and are described in detail in Walton (2011). Total pressure cells (TPC's) were used to measure soil pressure along the centerline arch at five elevations (43ft, 46ft, 49ft, 52ft and 55ft). A diagram and photograph of a typical TPC sensor and installation buried in soil can be seen in Figure 6 and Figure 7 respectively.

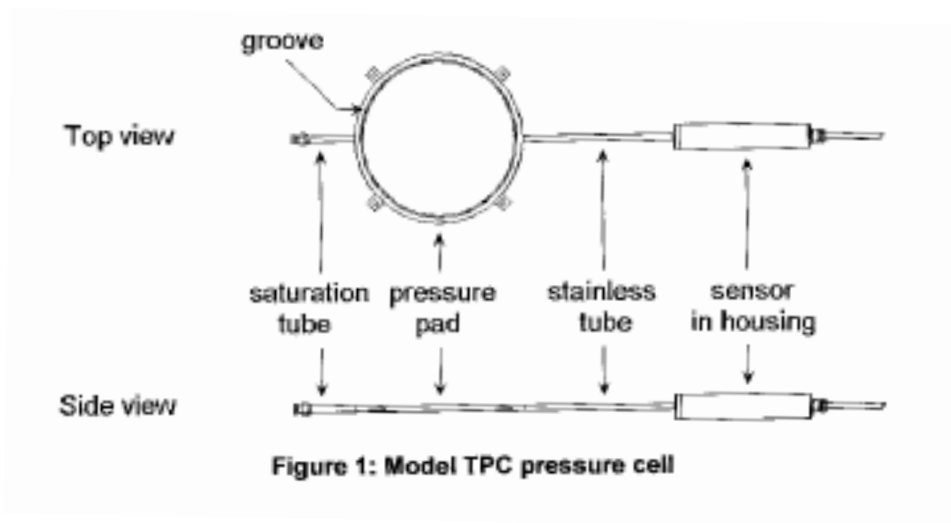


Figure 6: TPC sensor diagram



Figure 7: TPC in place prior to backfilling

The location of each of the TPCs can be seen in Figure 8. Sensors were placed to measure radial pressures applied to the arches.

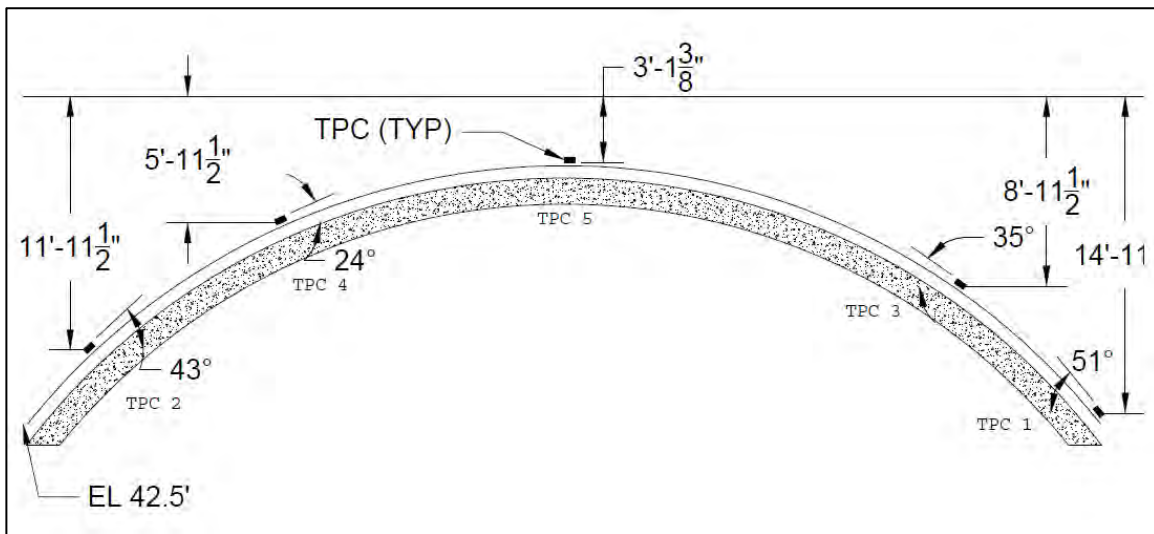


Figure 8: TPC Locations along centerline arch

Results of Construction Monitoring

Two data sets were collected that were reliable and significant. These included the strain and deflection data during the SCC placement filling the FRP tubes and the TPC data during backfilling.

Strain and deflection data during the tube filling with SCC is summarized here. A detailed reporting and analysis is provided in Walton 2011. In general, strain at the apex of the arches tracked well at the end points, once the arches were filled with SCC, with predictions as seen in Figure 9. Variations between predictions and measurements during filling may be due to uneven loading of the arches with SCC or inconsistent filling. Strains at the footing are similar but harder to measure given the proximity of the gages to the footing. Effects of the gages location may account for some of the difference in behavior as compared to the predictions as seen in Figure 10.

Deflections of the arch crown and shoulders can be seen in Figure 11 and Figure 12 respectively. Trends agreed with predictions, but final values for deflections were not as close at apex strains. Jumps in the deflection data may be able to be attributed to bouncing of personnel on the bridge during construction trying to “consolidate” the SCC in the tubes and allow trapped air to escape.

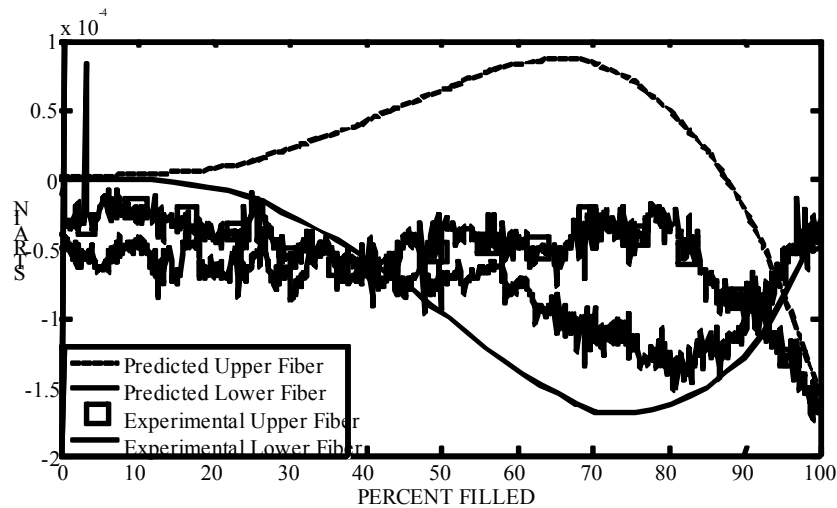


Figure 9: Apex Strains during Field Fill (Walton 2011)

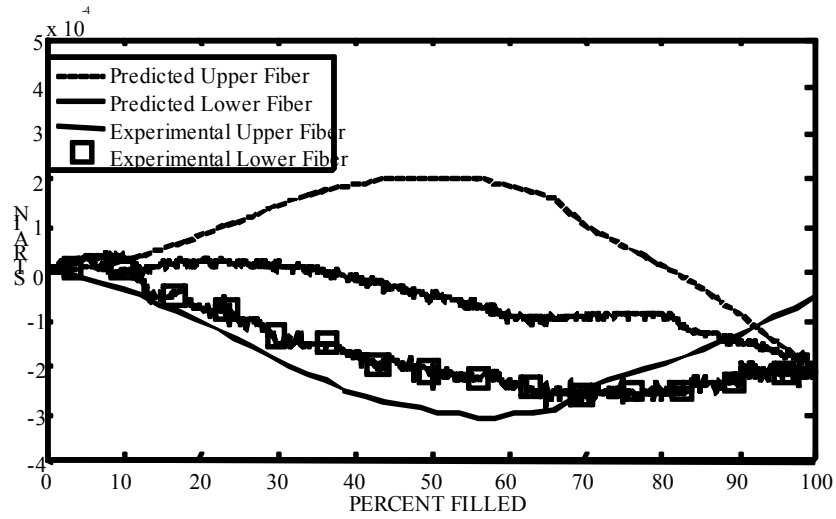


Figure 10: Footing Strains during Field Fill (Walton 2011)

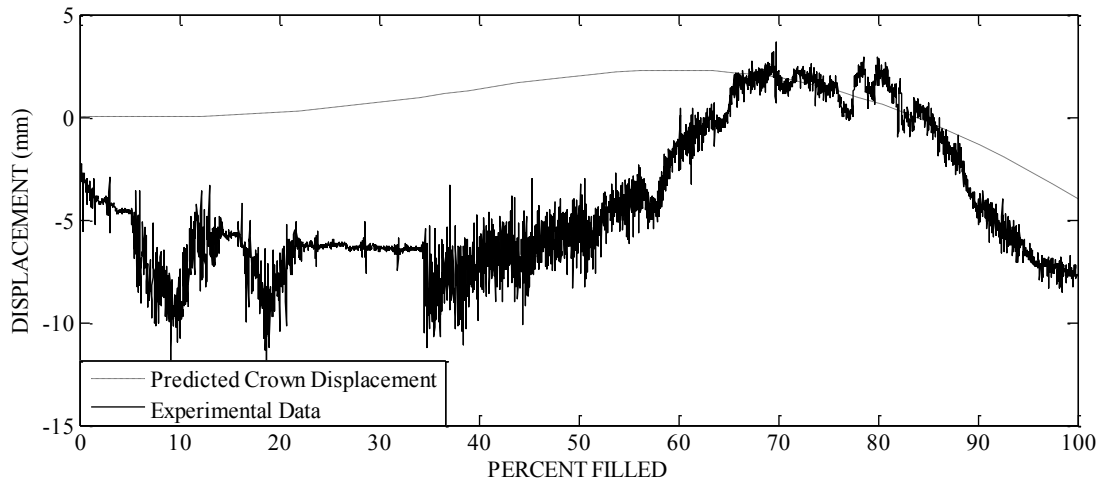


Figure 11: Apex Displacement, Arch 4 (Walton 2011)

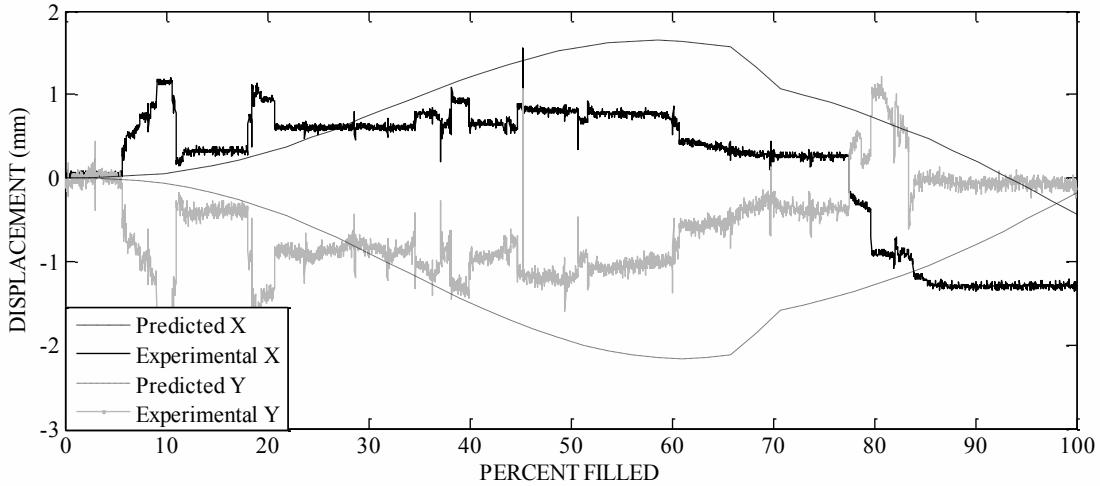


Figure 12: Shoulder Displacement, Arch 4 (Walton 2011)

Attempts were made to collect data during concrete deck placement but data values were very small and within the noise of the signal. Soil pressure data and strain data were collected during backfilling. Usable data included TPC data up to EL 52ft. Data collection was a problem for the remaining 5 ft of cover. Predictions for each of the sensors are given in Table 1 and a plot of the soil pressure data versus time is given in Figure 13. There is no data presented between November 8th and 10th so the straight line should not be interpolated in that region.

Table 1: Predicted TPC Pressures with Backfill at EL 52ft

11/12/2010	Depth (ft)	Angle (deg)	Radial to Arch Pressure (psf)	Radial to Arch Pressure (kPa)
TPC 1	8.96	51	1292	62.00
TPC 2	6.00	43	915	43.91
TPC 3	3.00	35	473	22.72
TPC 4	0.00	24	0	0.00
TPC 5	0.00	0	0	0.00

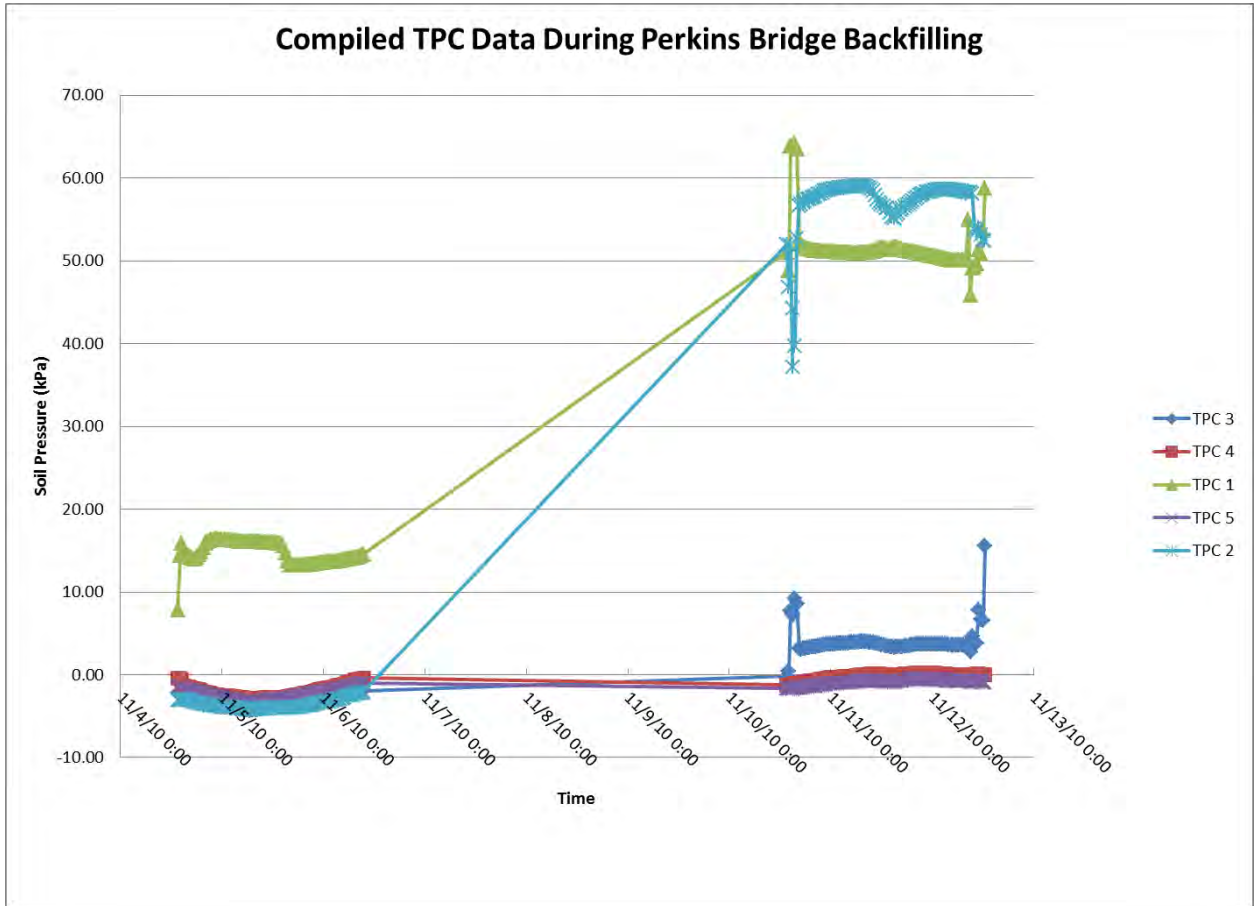


Figure 13: Available soil pressure data from backfilling

Peak soil pressures agreed well with the predicted values for radial soil pressures for backfilling to EL 52ft. Many variables that were not captured in the predictions contribute to variations from measurements to predictions. These include non-uniform backfilling elevations, non-uniform compaction and densities around sensors, inaccurate elevations of sensors and backfill, and differences in the angle of the sensors. Predictions were made with an assumed at-rest coefficient of 0.47 and compacted soil density of 145pcf (Maine DOT 2008).

In general, the predictions at various stages of construction agreed well with data collected. Measured values were small in many cases indicating conservatism in the design.

Live load testing

The Perkins Bridge in Belfast was instrumented and tested to investigate the live load response of the structure. Diagnostic load testing was conducted on August 20th, 2013. Previous testing with earlier equipment did not give good results. The BDI wireless structural testing system was used for this final test to measure strain. Soil pressure data

was also collected throughout the test. Loaded dump trucks were used to load the structure with rolling trucks and at static positions specified from previous tests.

Instrumentation

Strain and soil pressure data were collected with the newly acquired BDI wireless structural testing system and total pressure cells (TPCs) respectively. Twenty four strain gages were installed at various positions on various arches as seen in Figure 14. Each position on the arch had three gages installed at seen in Figure 15.

Gage Locations on Arch's

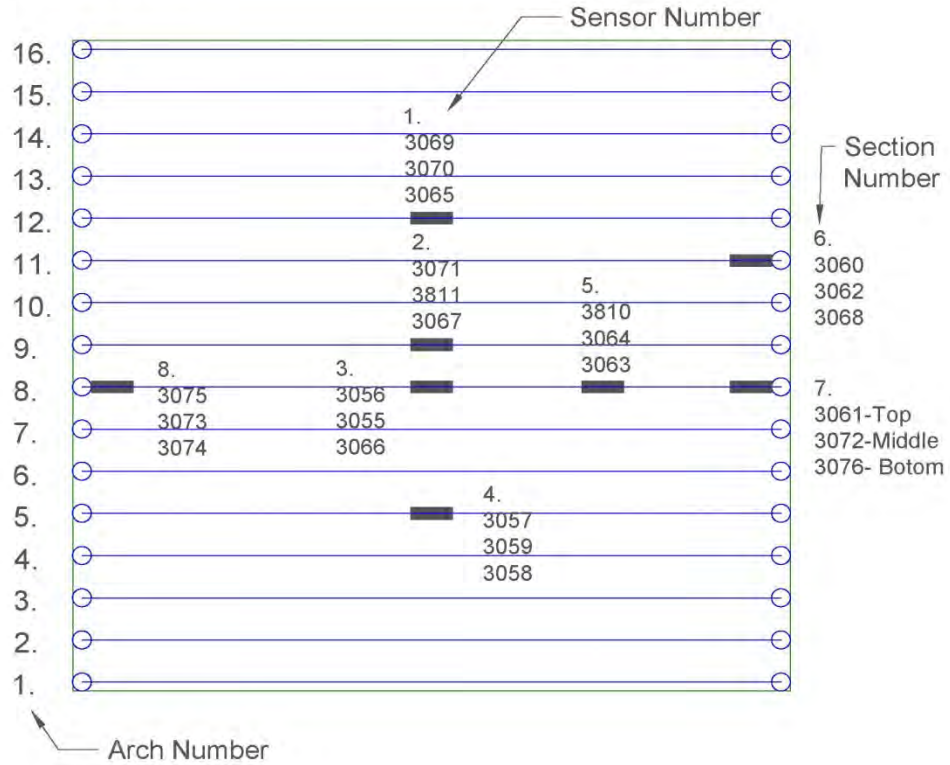


Figure 14: Gage Locations on Arch's (numbered starting from upstream)

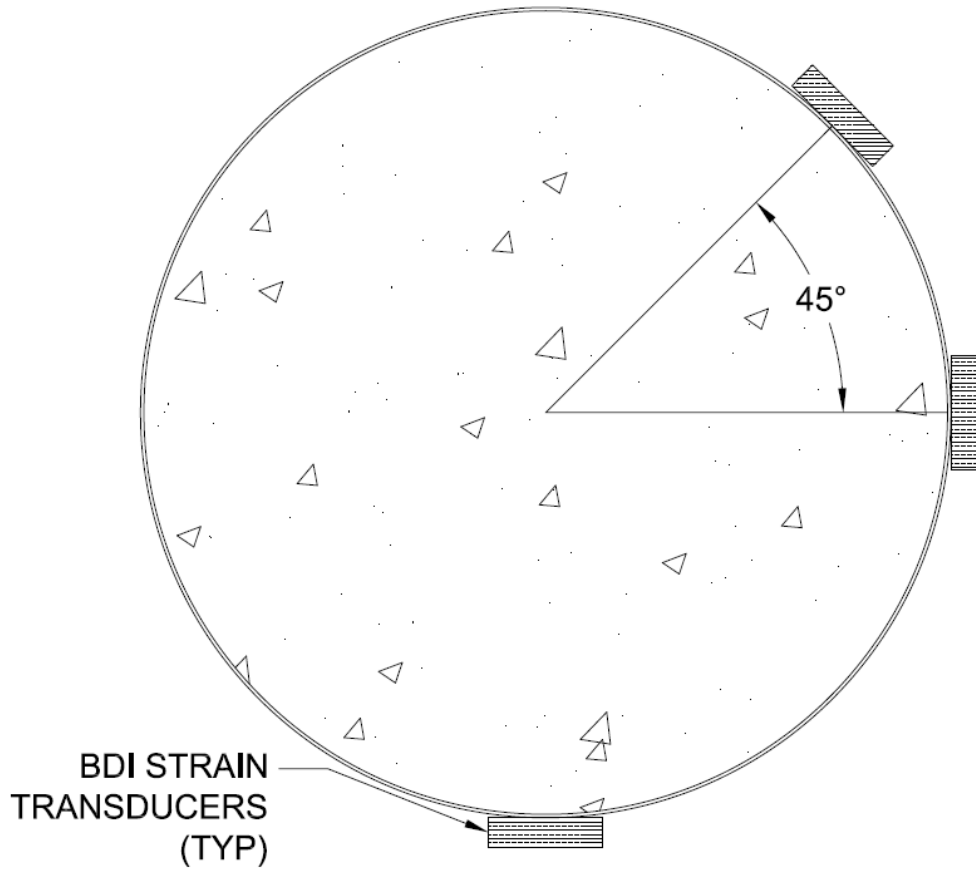


Figure 15 - Strain gage location in cross section

Live Load

Two loaded tandem rear axle dump trucks were used to load the bridge in a series of longitudinal and static tests. Their weights and dimensions can be seen in Figure 16 and Figure 17.

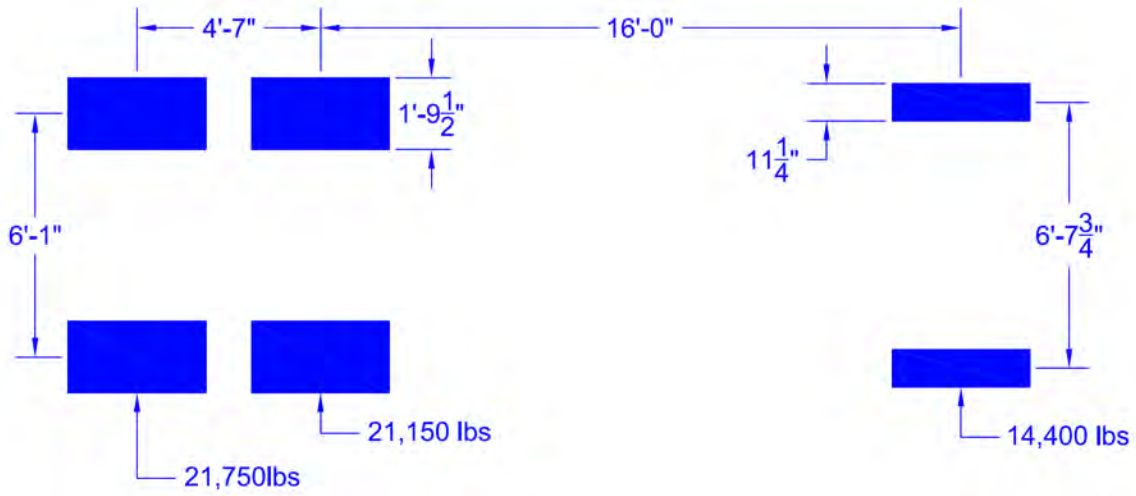


Figure 16: Truck 1 (T01-214) dimensions and axle weights

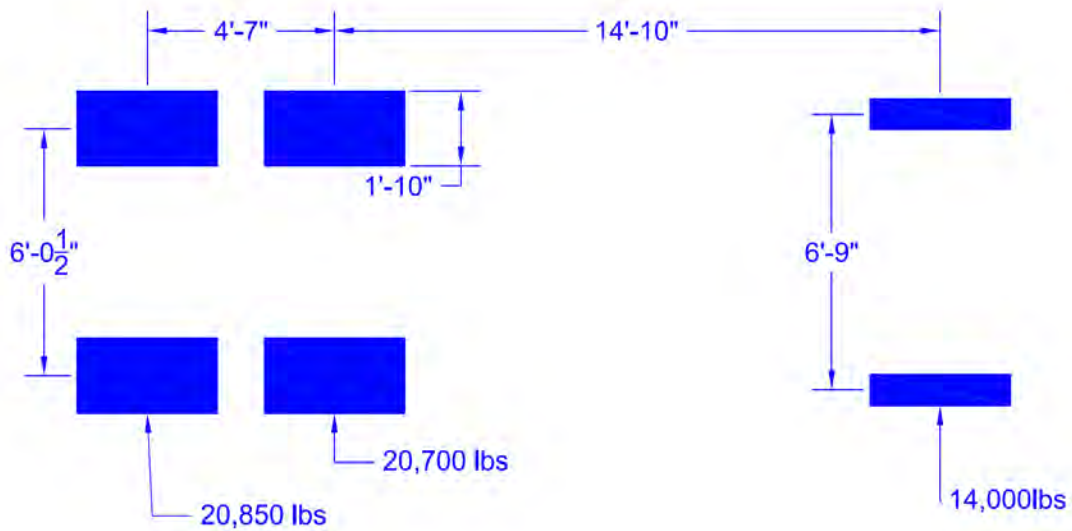


Figure 17: Truck 2 (T01-117) dimensions and axle weights

There were six rolling and five static truck positions. Five of the rolling positions (Y1 through Y5) used a single truck at different transverse positions on the bridge and their locations are shown in Figure 18. In Y6, two trucks were each placed in a lane 2 feet off centerline and driven at the same speed across the bridge. The static positions can be seen in Appendix B.

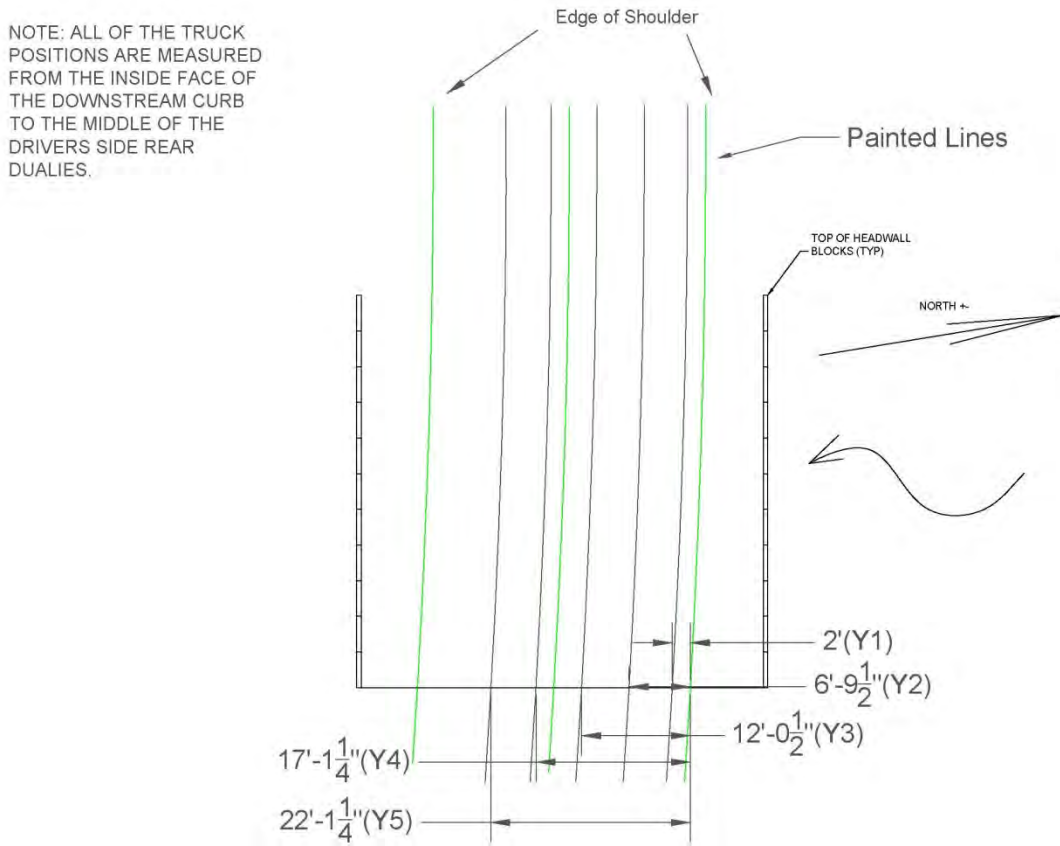


Figure 18: Truck Positions Y1 to Y5

Results

Peak strain values for each of the truck positions are given in Table 2. A peak strain value of 18.0 microstrain was recorded with both lanes loaded with one truck in each at midspan. Plots for peak strain are shown in Figure 19, Figure 20, and Figure 21.

Table 2: Perkins Bridge Maximum and Minimum Strain at each Truck Position

Truck Position	Cross Section	Truck Position	Max Strain (micro strain)	Cross Section	Min Strain (micro-strain)
Y1	1	N/A	12.0	1	-12.2
Y2	2	N/A	11.6	5	-9.0
Y3	3	N/A	12.2	2	-11.4
Y4	4	N/A	13.7	5	-6.5
Y5	4	N/A	14.9	4	-7.5
Y6	2	N/A	17.6	2	-16.1
Abutment 1	4	Static/End of bridge	2.2	8	-5.3
Y7	4	Static/1/4 point	4.4	8	-9.6
Y8	2	Static/midspan	18.0	2	-17.3
Y9	4	Static/3/4 point	17.1	5	-15.0
Abutment 2	5	Static/end of bridge	11.0	5	-14.0

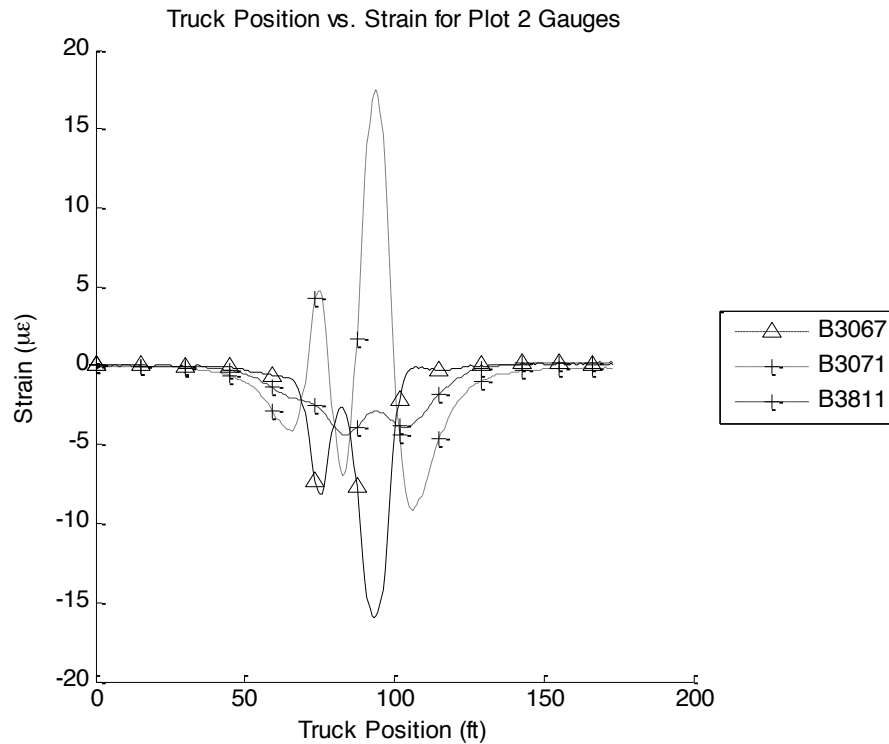


Figure 19: Y6 Cross-Section 2 Strain Values

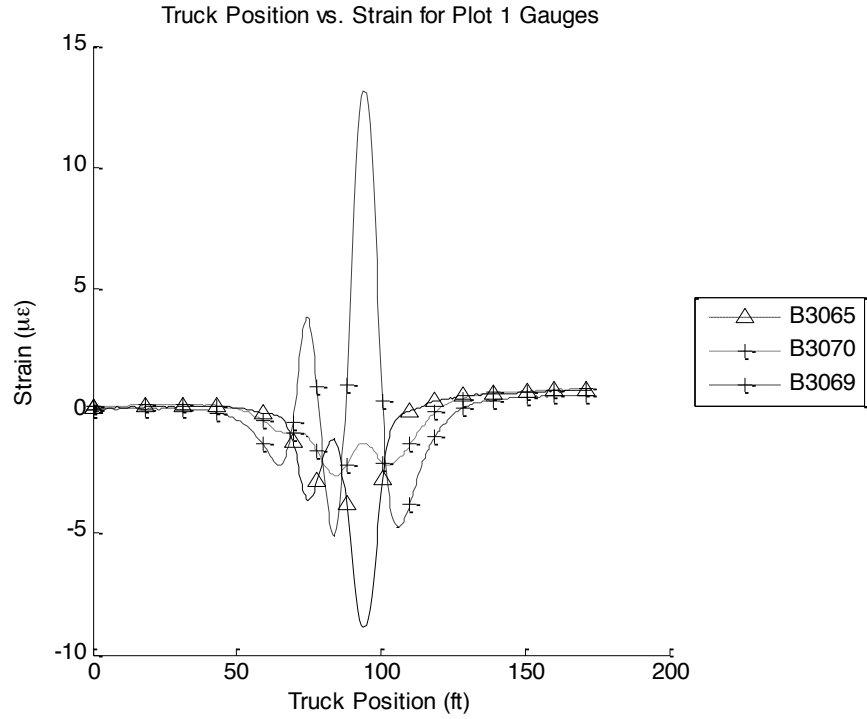


Figure 20: Y1 Section 1 Strain Values

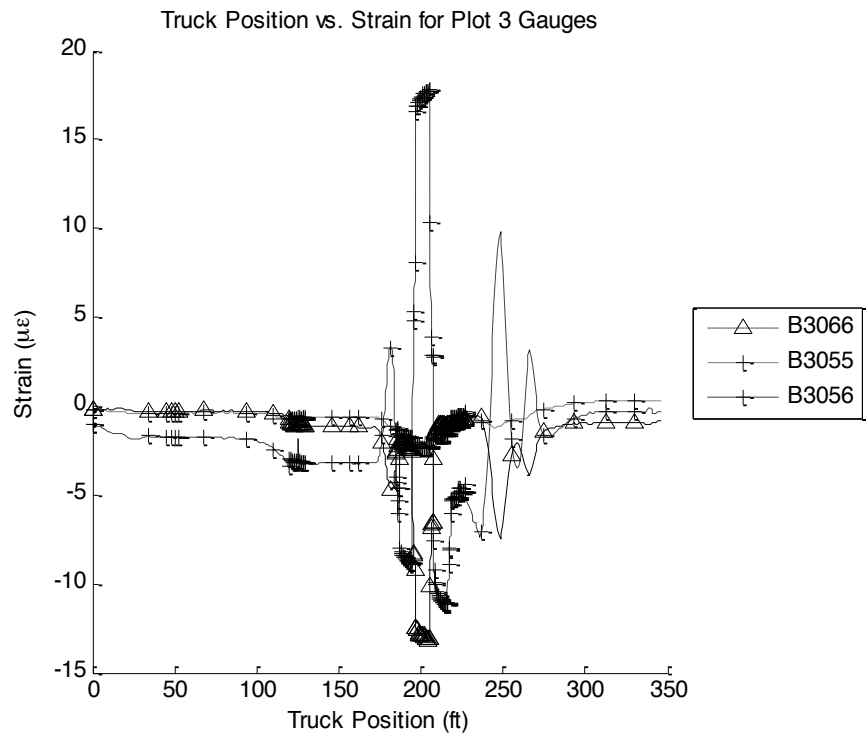


Figure 21: Y7-Y9 Section 3 Strain Values

Additional strain graphs for the remaining sensors and load cases are shown in Appendix C. There were problems with the data collection for the TPC data, which was not collected correctly during the test.

4.2.2 Jenkins Bridge Construction Monitoring and Load Testing

The Jenkins Bridge in Bradley was instrumented with the objective of looking at the pile-supported foundation thrusts from the arches. Significant thrust reactions were designed into the foundation piles at a considerable cost. Measurements were taken during construction and during a live loading to investigate arch flexure and thrust into the foundation.

Instrumentation

Three sets of instruments were installed on the structure. They included foil resistance strain gages to measure bending strains in the FRP tubes, vibrating wire (VW) total pressure cells (TPCs) to measure soil pressures behind the foundation during construction and during loading, and PK nails on the exposed foundation to measure any foundation movements. A steel surveyors tape was used to take measurements between PK nails. Figure 22 shows the PK nail measurement during bridge loading. PK nail data from before and after backfilling is presented with the live load test data.



Figure 22: PK Nail measurements during loading

TPCs were mounted to the back of the foundation to measure soil pressures at the rear vertical face of the foundation. Grout was used to ensure flush mounting of the TPCs to the vertical concrete face. Stone was used for backfilling this elevation of the foundation so sand was packed around the TPC with landscaping cloth to ensure as uniform a pressure as possible on the oil filled pan. However, during construction all of the TPCs were bent over to allow for the membrane covering of the concrete deck to be placed. It is unknown if the bending of the thin oil filled tubes affected the measurements. No kinks were visible or other indications of damage. Data is thus presented for reference only and may not be reliable. TPC data was only collected during the load testing of the bridge.

Diagnostic Load Testing

Diagnostic load testing of the Jenkins Bridge was conducted on November 30th, 2010. Data was collected to capture the performance of the bridge under 15 different static load cases. Two double rear axle dump trucks were used to load the bridge in two different static test configurations.

Live Load

Two loaded tandem rear axle dump trucks provided by the Maine DOT were used as the live load for this testing. Individual average axle weights for the trucks are given in Figure 23.

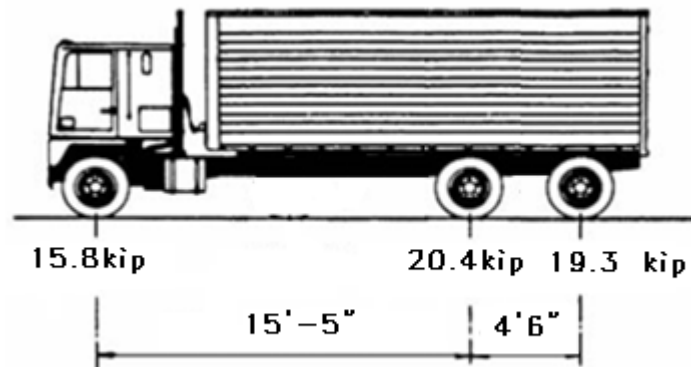


Figure 23: Average Axle Weights of Test Trucks

Longitudinal Truck Positions

Five longitudinal static truck positions were used along the length of the bridge with two transverse lane positions for a single truck followed by loading with one truck in each lane. This totals 15 truck positions and is shown in Table 3. Longitudinal truck positions were determined using locations for predicted peak thrust load and moment in the arches at the footing. The scaled influence line from those calculations is shown in Figure 24 and Figure 25.

Table 3: Truck Positions During Testing

Transverse Location of Driver's Side Wheel Line or Position of Truck			
Longitudinal Position of Front Axle from CL	Lane A-Single	Centerline- Single (B)	Two Trucks CL of Lanes (feet from CL) (C)
-40 inches	CL of South Lane	Trk centered over CL	2' and 7'-5"
165 inches	CL of South Lane	Trk centered over CL	2' and 7'-5"
309 inches	CL of South Lane	Trk centered over CL	2' and 7'-5"
Centerline	CL of South Lane	Trk centered over CL	2' and 7'-5"
190 inches	CL of South Lane	Trk centered over CL	2' and 7'-5"

Influence Line for Thrust from Single Arch

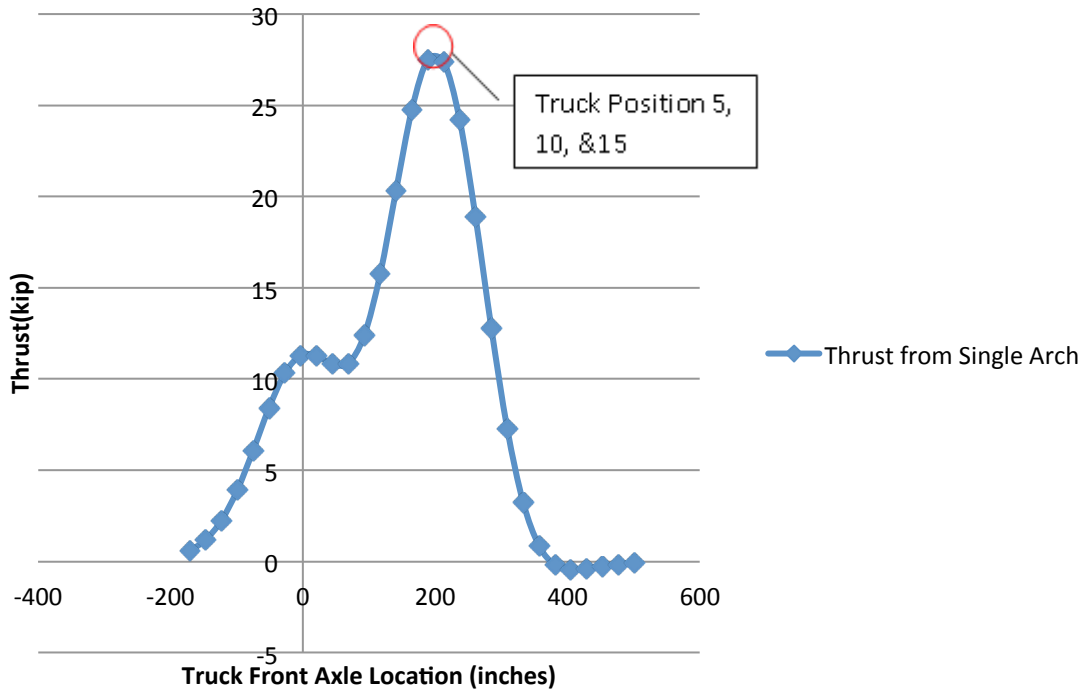


Figure 24: Influence line for thrust from single arch

Approximate lane positions are also shown graphically in Figure 26. Single truckload cases are shown A and B transverse lane positions. The trucks were side by side in the

third transverse lane positions as shown in figures label C Test Vehicle in Figure 26. Predicted live load strains are shown in Table 4.

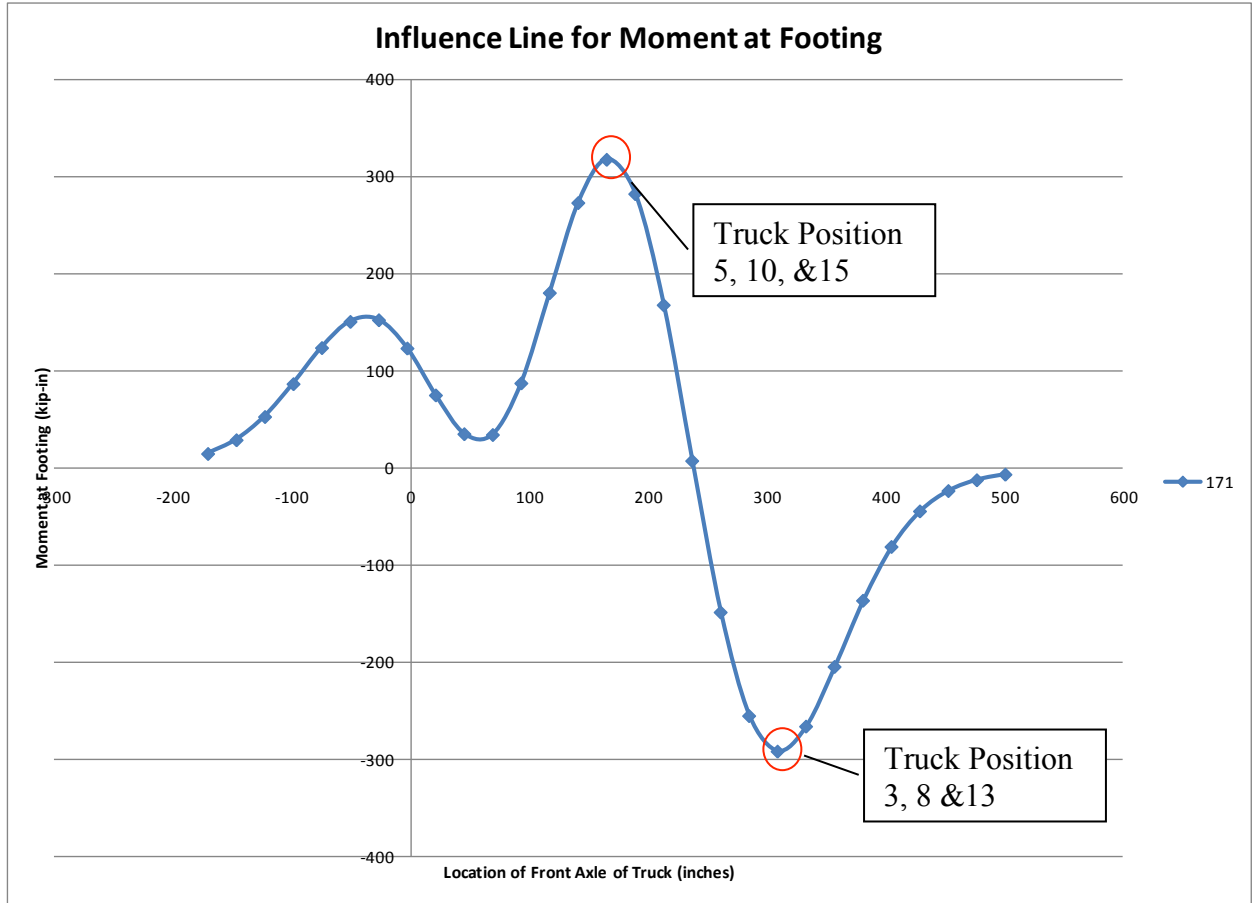


Figure 25: Influence Line for Moment at Footing of Arch

Table 4: Predicted Strain for Live Load Testing

Jenkins Bridge (Bradley, Maine)

Single Truck Loading Positions	1,6	2,7	3,8	4,9	5,10
Truck Positions (inches from end of bridge)	-40	165	309	0	190
Flexural Tensile Strain (in/in)	4.14E-05	1.66E-05	7.50E-06	4.52E-05	1.18E-05
Axial Strain (in/in)	3.50E-05	1.06E-05	9.92E-06	3.05E-05	1.02E-05
Total Tensile Strain (in/in)	7.64E-05	2.72E-05	1.74E-05	7.58E-05	2.20E-05
Total Compressive Strain (in/in)	9.02E-05	1.37E-04	1.44E-04	1.66E-04	1.40E-04

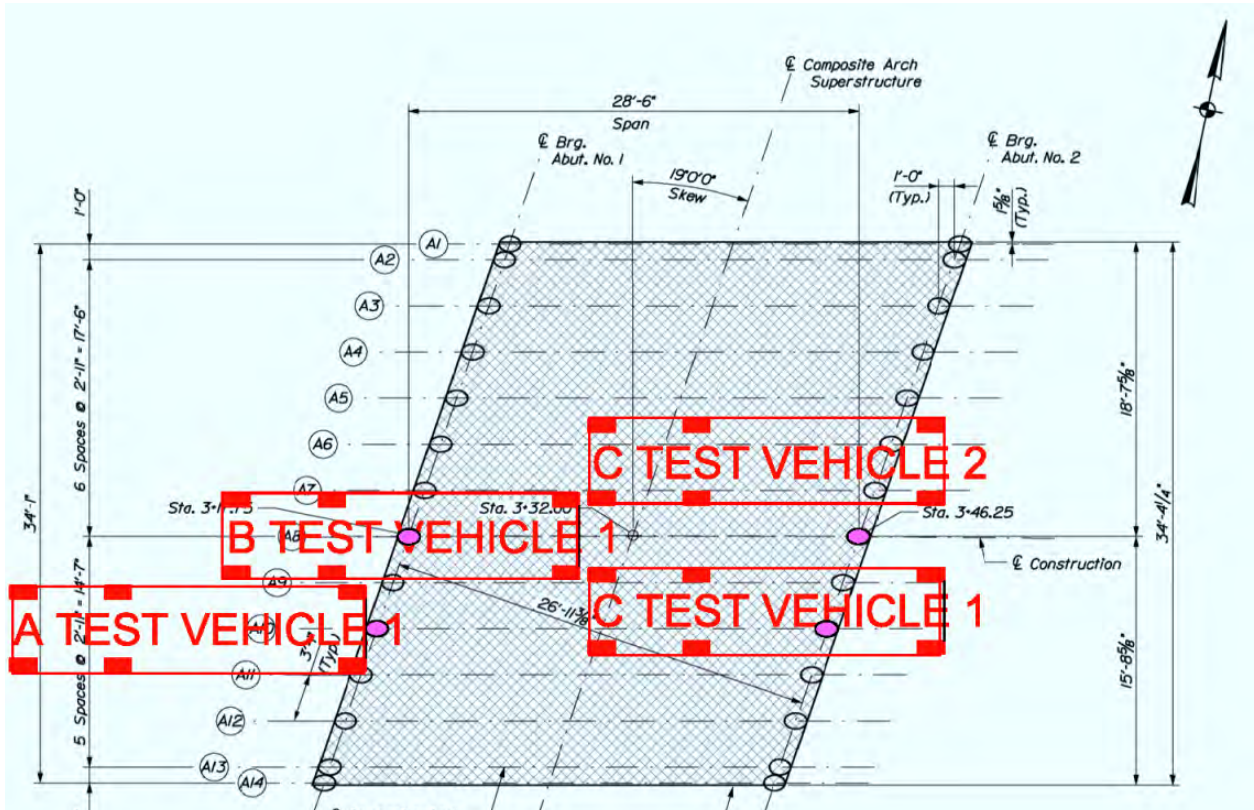


Figure 26: Truck lane positions (3) during testing

Results

PK nail movement and TPC soil pressure data were the emphasis for the instrumentation for this bridge. A summary of PK nail measurements is presented in Table 5. Soil pressure data is presented in Figure 27. Strain information is also presented in Figure 28.

In general, some outward movement of the foundations was measured (peak of 0.025ft relative spreading of both footings) along with some inward movements of other sections of similar magnitude (.03ft). Peak soil pressure measurements with two lanes loaded was 400 psf.

Table 5: Corrected P.K. Nail Measurements Before and During Load Testing of Jenkins Bridge

Event	Downstream		Upstream	
	Top (ft)	Bottom (ft)	Top (ft)	Bottom (ft)
Backfill to back of footing	26.150	27.180	26.135	27.130
Dead Load Only	26.150	27.140	26.150	27.130
Single Truck Over A8- 190" From CL	26.155	27.160	26.140	27.130
Single Truck Over A10- 190" From CL	26.170	27.150	26.150	27.130
Two Trucks 2ft From CL (A8)- 190"	26.160	27.150	26.160	27.120

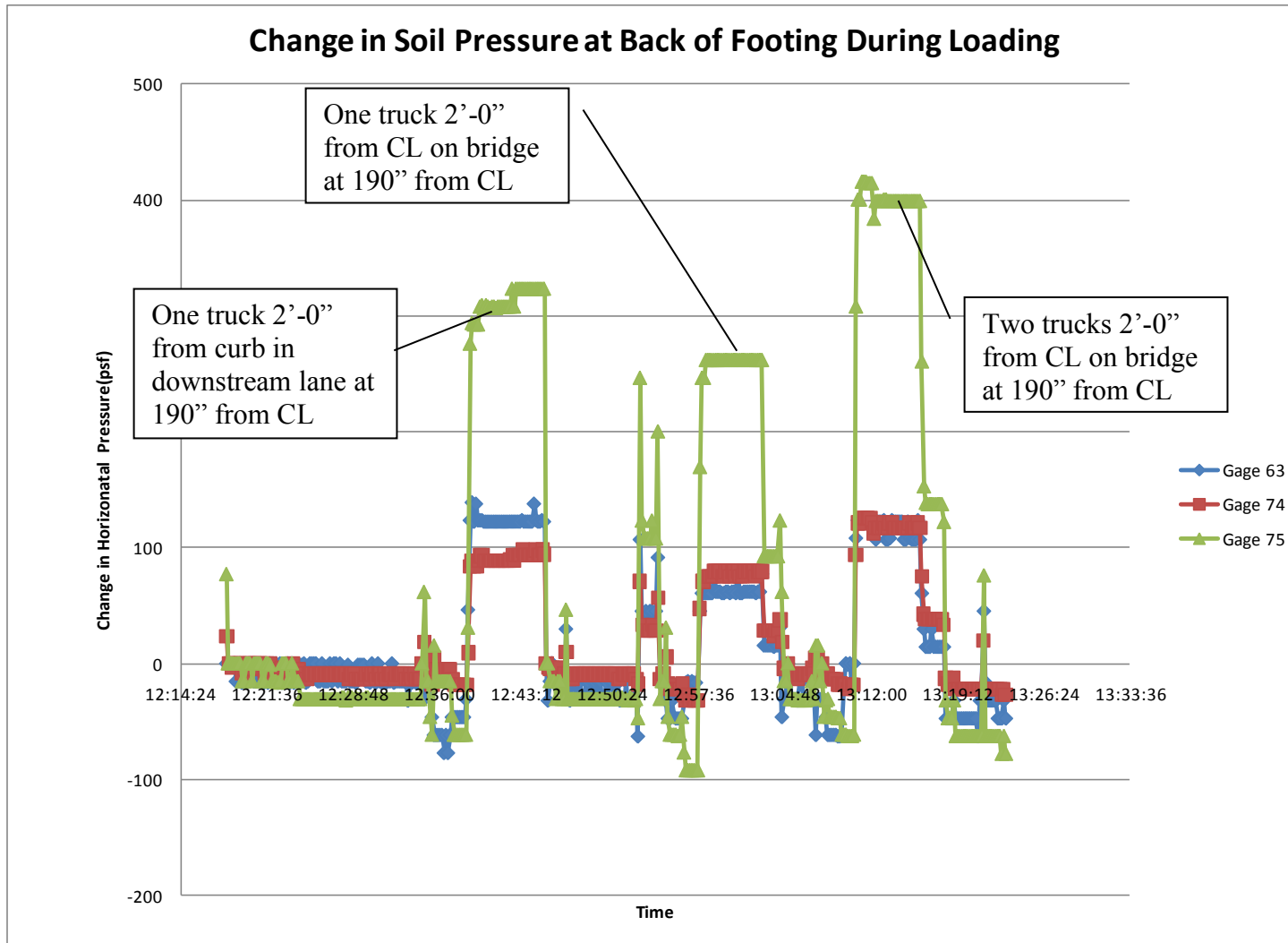


Figure 27: Recorded change in soil pressure (thrust) from TPCs behind foundation

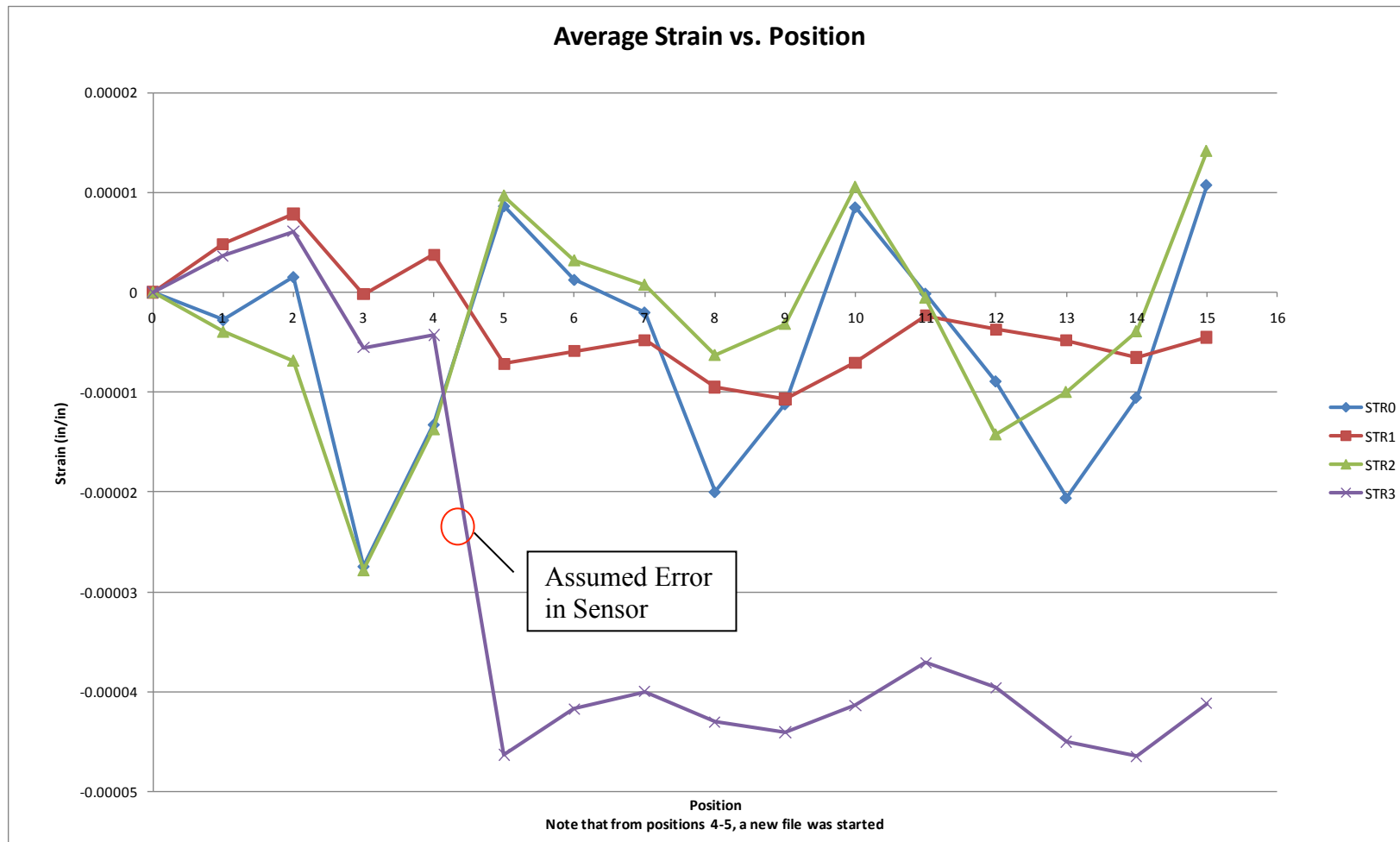


Figure 28: Plot of strain values during bridge loading

Summary

Construction monitoring and live load testing of the Perkins and Jenkins bridges was completed and provided information in the performance of the hybrid composite arch structures. There were several problems with instrumentation and data collection but useful information was gathered in regards to foundation forces of pile supported arch bridges, arch forces during concrete filling, backfilling pressures onto the deck and arch strains due to live load.

Data provided from this testing showed that existing analysis techniques give good results for arch forces during concrete filling. Additional conclusions can be found in Walton 2011.

Live load testing showed the design of these structures to be conservative with regard to live load effects. Arch measured strains were an order of magnitude smaller than predicted strains (Table 4 & Figure 28).

Task 4.3 Inspections of 2010 bridges

Inspections were completed on July 1, 2011 of the Royal River Bridge and Perkins Bridge by Bill Davids and Keenan Goslin. The inspections were conducted to provide an independent view on construction, design and expected durability of these three bridges.

Royal River Bridge

A visual inspection was conducted looking mainly at the FRP tubular arches, decking and headwalls. An overall view of the underside of the bridge is given in Figure 29.



Figure 29: Underside of bridge

A very noticeable detail is the holes used to attach the formwork for the headwall concrete detail. It is understood by UMaine that these holes were allowed to be drilled and then plugged after formwork removal as seen in Figure 30. These holes can be expected to negatively impact strength and durability of the hybrid FRP member to an unknown degree but if other details are available it is recommended not to use this method in the future.



Figure 30: Plugged hole in tube

Concrete spillage around the filling hole has been observed in most of the bridges constructed to date to varying degrees (Figure 31). Though mostly cosmetic it is expected minor steps can be taken during placement of the deck to mostly eliminate this and keep SCC in the tube or on the deck.



Figure 31: Concrete leakage near crown

The detail of the headwall attachment and closure around the exterior arch required concrete placement up to the end of the corrugated FRP. In this case orange expanding foam was used to seal the underside of the corrugations. Alternative details or colors would help the aesthetics of the bridge as seen in Figure 32.



Figure 32: Colorful foam backer

The Royal River is an attractive and innovative bridge design. However, additional design and construction refinements can make the hybrid composite arch bridge technology more cost competitive and visually appealing.

Perkins Bridge

A similar visual inspection was completed at the Perkins Bridge in Belfast, Maine. With the exception of the diameter of the FRP tubes, the Perkins Bridge superstructure was designed very similarly to the Royal River Bridge.

High water events were noticed during construction and have occurred at this site since construction was completed. A puncture was noticed in one of the tubes as seen in Figure 33. This was in the first upstream tube. It was marked and mentioned to AIT engineers. Debris was also seen in the heavy rip rap, higher than expected as seen in Figure 34.

The Perkins Bridge had a similar headwall design as the Royal River Bridge. Here the discoloration between the CIP and precast concrete was more evident (Figure 35). This may fade to match in years to come, but is an aesthetic consideration.



Figure 33: Hole in tube



Figure 34: Debris in heavy rip rap indicating high water mark above base of tubes



Figure 35: Concrete discoloration between T-Wall and CIP concrete

The largest potential impact from the inspection was found in the 7th arch from the upstream headwall where severe fiber angle deformities in the exterior layer of braided carbon fiber tows were found. This was located approximately 3-4ft up from the foundation. This is expected to be a low stress region for this FRP layer so there is potentially minimal impact for this case, but fiber angle consistency should be inspected with future bridges. It was not possible to get a good photo of this.

Jenkins Bridge

The Jenkins Bridge was inspected in 2014 by Keenan Goslin. A visual inspection was conducted looking again at the arches, headwalls and decking details. This bridge used

different headwall and foundation details than the Perkins Bridge and Royal River Bridge.

The exterior FRP tube has an additional tube bonded to it to support the base of the headwall and is the only critical point for this inspection. Shortly after construction of the bridge, high water and debris caused some damage as seen in Figure 36. This hollow tube is susceptible to damage, but was also easily repaired as seen in Figure 37. There was no apparent damage to the FRP tubes, decking or debris seen between the tubes (Figure 38).

The self reacting FRP headwall is attractive and was inspected for deflections. None were seen visually (Figure 39). Some large vegetation was seen on the bridge span near the headwall that could be addressed in the future as seen in Figure 40.



Figure 36- Damaged outside arch (2011)



Figure 37 - Close up View of Repair on Upstream Tube (2014)



Figure 38 – Underside of Jenkins Bridge (2014)



Figure 39 - Headwall of Jenkins Bridge (2014)



Figure 40 - Vegetation Behind Headwall of Jenkins Bridge

The Jenkins Bridge appeared to be in good condition. Some design recommendations were mentioned in the Lessons Learned meeting and are recorded in Appendix A.

Task 4.4 Inspections of 2011 Bridges

An inspection of the Farm Access Underpass Bridge in Caribou, Maine was conducted after the FRP deck placement, and during concrete filling of the arch tubes by Bill Davids of UMaine and Dan Bannon of AIT. Visual inspection of the arches indicated some local irregularities in tube braid angle at non-critical locations, an issue that can be rectified with improved infusion techniques. There were significant issues during filling due to poor performance of the SCC. Specifically, the mix was not uniform with some pockets of dry cement observed during pumping, and the mix began to lose spread (increase viscosity) quickly after the truck's arrival, preventing arches from being filled. Representatives from the admixture supplier were on-site during the pour, and these problems were rectified during a later pour with no negative effect on project schedule.

Task 4.5 Guidance and recommendations

Construction monitoring, live load testing and inspections of the hybrid composite arch bridges has led to some recommendations. Additional comments and recommendations by contractors, the MaineDOT and others are included in Appendix A.

Design changes can be made in the future to enhance the competitiveness and constructability of the hybrid composite arch bridges. Testing has shown conservatism in live load distribution to the structure. Enhanced analysis and/or testing can lead to greater arch spacing and more efficient deck design. This would include the contribution of the concrete deck, other concrete placements, and soil-structure interaction that was

accounted for in the design of the bridges addressed in this report. Large concrete placements in Belfast added considerable stiffness to exterior arches that was not accounted for. This also added to construction time. Headwall solutions that do not require these large concrete placements are recommended.

An alternative to the decking used is recommended. This will allow for greater arch spacing and allow for greater competition potentially for alternative products.

Greater quality control is recommended for the arches. The deviation in fiber angle in the Perkins Bridge is a red flag, as are tolerances mentioned by contractors trying to level the arches before attaching the rigid decking.

The headwall design can be improved. It was expected that each of the three 2010 bridges have three different designs to allow for more improvements. However, a rapidly constructed headwall that better matches the geometry of the arches and is easily connected to the deck would eliminate the large concrete placements performed in Auburn and Belfast and complex reinforcement details at the exterior arches. Finally, the hollow exterior tube used at the Jenkins bridge is susceptible to damage and should not be used in future construction.

These projects allowed for the demonstration of several design and construction methods that still provided a long lasting, attractive bridge. The testing conducted showed that current analysis methods for dead loads do a good of predicting response, but live load effects are conservatively predicted by current analysis methods. Reductions through enhanced analysis or testing could reduce the cost of future projects.

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APPENDIX A – LESSONS LEARNED



Date: Friday, November 19, 2010

Location: University of Maine – AEW Center

Subject Bridge(s): Meeting (1) – Royal River Bridge, Auburn, ME and Jenkins Bridge, Bradley, ME
Meeting (2) – Perkins Bridge, Belfast, ME

Attendees:

Maine DOT	Nate Benoit, David Sherlock, Dale Peabody, Robbin Lanpher (1), Glenn Philbrook (1), Rich Gebert (2)
AEWC	Larry Parent, Keenan Goslin, Josh Clapp, Bill Davids (2)
AIT	Dan Bannon, Jon Kenerson, Bob Schmitt
Kleinfelder SEA	Keith Wood, Matt Steele, Bob Blunt
Wyman & Simpson (1)	Doug Hermann, Brian MacFawn, Kim Suhr, Robert Herbert
Stetson & Watson (2)	John Stetson

Purpose: A meeting was held to identify solutions to some of the field issues encountered during construction of the composite arch bridges in Auburn, Bradley, and Belfast, and to make recommendations for future improvement of the composite bridge system. This document provides a summary of the recommendations given during the two meetings.

Summary of Recommendations:

1. Abutments

- a. Avoid driving piles, found on ledge when possible, provide a spread footing option for sites where scour is not an issue
- b. Avoid deep excavations for abutments
- c. Minimize concrete placements, single placement for abutments if possible
- d. Avoid vertical construction joints, use a single horizontal construction joint for arch embedment
- e. Locate first construction joint above mean water level to get out of water as soon as possible
- f. In skewed bridges, abutment and reinforcement need to be skewed to alignment of arches
- g. Avoid overhang castings (as in Bradley)
- h. Rebar cage design needs to consider constructability: bar spacing needs to accommodate arches, bars need to be supported in multiple planes to not require additional bars for construction, tolerance on bar installation needs to be matched to construction tolerance on arch installation

2. Arches

- a. Address arch tolerance issues, construction tolerance needs to match arch fabrication tolerance, arch tolerance of $\pm\frac{1}{2}$ " should be adequate for constructability
- b. Beveled end detail (Belfast) provides for easier construction than locator rod (Auburn & Bradley)
- c. Remove flow media prior to shipping arches to avoid any obstructions to concrete flow
- d. Possible idea of modularizing – 3 arches come to site attached with decking or spacers
- e. Limited access to middle of arches when over water makes alignment and inspection difficult, provide a means for alignment from ends

3. Arch Fill Concrete

- a. Top fill method preferred to end fill, 2.5" hole and fill box worked well
- b. Concrete spec is very rigid, relax where possible

- c. Investigate segregation, not observed on site, but suppliers noted that dynamic segregation may be a concern when filling tubes
 - d. Mix is very sensitive, need to determine why mixes that are proportioned the same (within small margin of error) vary widely in performance
 - e. More retarder must be used for SCC than a conventional mix to get the specified working time
- 4. Composite Decking**
- a. Investigate other suppliers – current panels have a long lead time and no competitive bidding, one option is AIT stocking and supplying panels
 - b. Some screws stripped, locally thickening of tubes was recommended
 - c. Do not recommend using rivets, more time for installation and would require direct contact between arch and panel to install
 - d. Eliminate rivets on adjacent panel seams if possible
- 5. Concrete Deck Slab**
- a. Bar spacing or mesh size should consider people having to walk on the arch slope
 - b. Placement of deck reinforcement difficult, need a better way to maintain spacing off corrugated panels and required cover (in Belfast, this required 3000 rebar chairs, in Bradley, the mesh was spaced with ¾” pex pipe which worked well)
 - c. Eliminate waterproofing membrane, unnecessary step increases cost and time (in some cases up to 14 days can be required between concrete placement and membrane application)
 - d. If membrane is used, come up with a better detail at deck to headwall connection to prevent water damage on wall
- 6. Headwalls**
- a. Recommendations for improved wall design:
 - i. Cast T-walls matched to geometry of arch to eliminate steps, use in combination with a cast in place deck and coping
 - ii. Widen structure and eliminate wall completely
 - iii. Widen structure somewhat to minimize wall height, then use a CIP wall 12” tall at the crown and increasing in height towards each end
 - iv. CIP wall with a stay in place FRP panel form (fill voids in panels used in Bradley)
 - b. Design a better solution for anchoring base of wall – CIP coping, curved steel waler, other
 - c. Need to consider serviceability conditions for FRP panel walls, deflection of panels in Bradley noticeable, deflection of waler beam in vertical direction (weak axis) was significant
- 7. General Design Recommendations**
- a. Site selection is critical for an arch bridge
 - b. Cost is most important issue, system needs to be competitive
 - c. On a project where there are many design changes, having the right team involved and working cooperatively are very important
 - d. Accurate geotechnical, hydraulic, and survey information are critical
- 8. General Constructability Recommendations**
- a. When bidding, construction procedure is based on mean water elevation (Q1.1), if this is not accurate it can have a large effect on constructability.
 - b. Riprap placement in front of abutments is difficult with forms for embedment pour still on footings.

APPENDIX B—TRUCK POSITIONS FOR THE 2013 PERKINS BRIDGE LIVE LOAD TEST

Truck Position- Start of Abutment

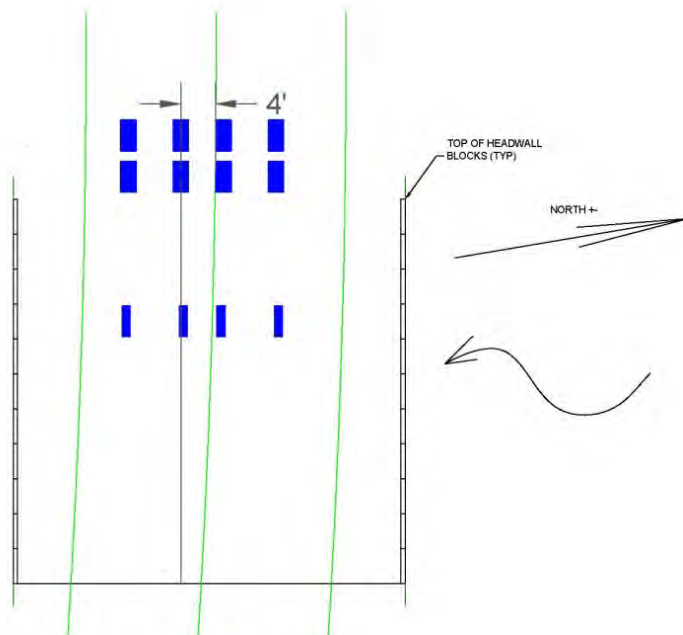


Figure 41

Y8 Static Truck Position- Quarter Point

NOTE: TRUCKS TRAVELED IN STRAIGHT LINE FROM THE START OF THE ABUTMENT

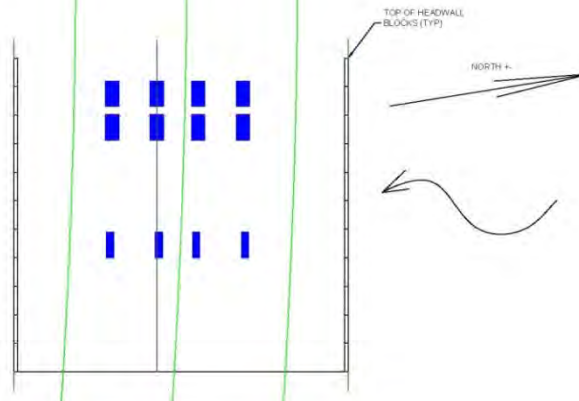


Figure 42

Y8 Static Truck Position- Midspan

NOTE: TRUCKS TRAVELED IN STRAIGHT LINE FROM THE START OF THE ABUTMENT

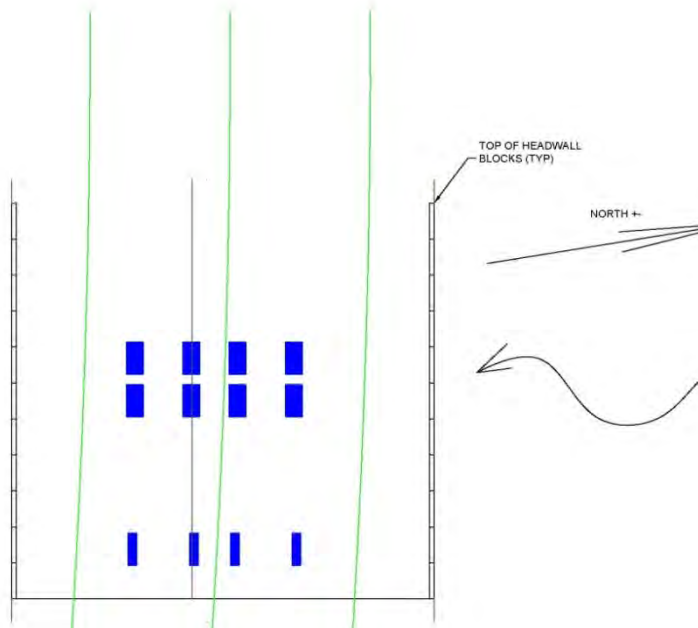


Figure 43

Y9 Static Truck Position- 3/4 Span

NOTE: TRUCKS
TRAVELED IN
STRAIGHT LINE FROM
THE START OF THE
ABUTMENT

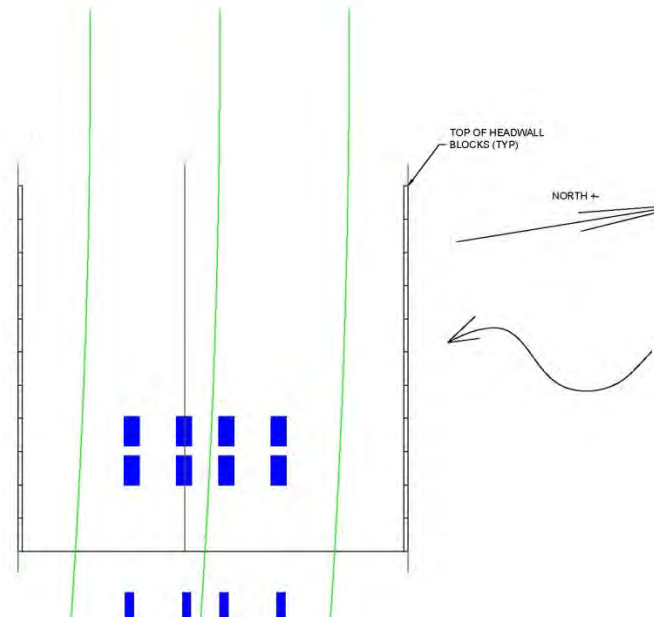


Figure 44

Static Truck Position- East Abutment

NOTE: TRUCKS
TRAVELED IN STRAIGHT
LINE FROM THE START
OF THE ABUTMENT

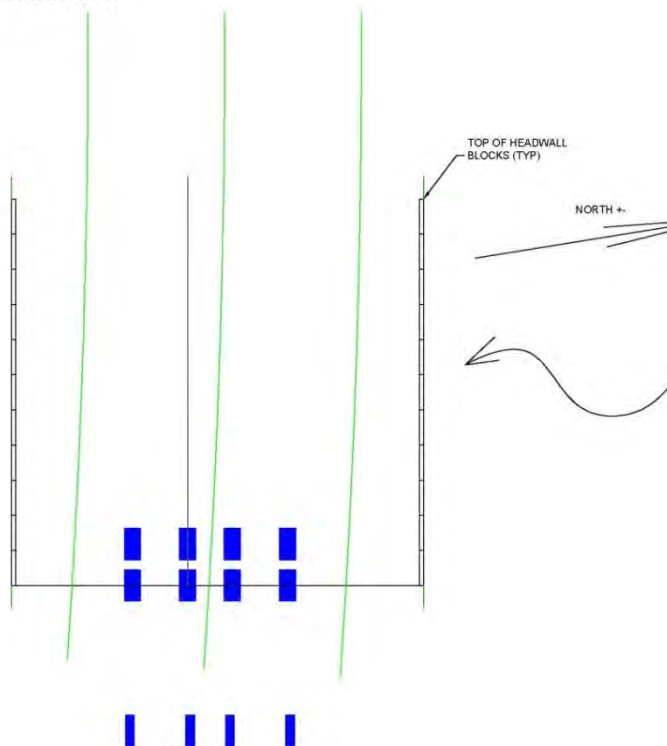


Figure 45

APPENDIX C – STRAIN GRAPHS PERKINS BRIDGE (2013)

Y-1 Strain Graphs

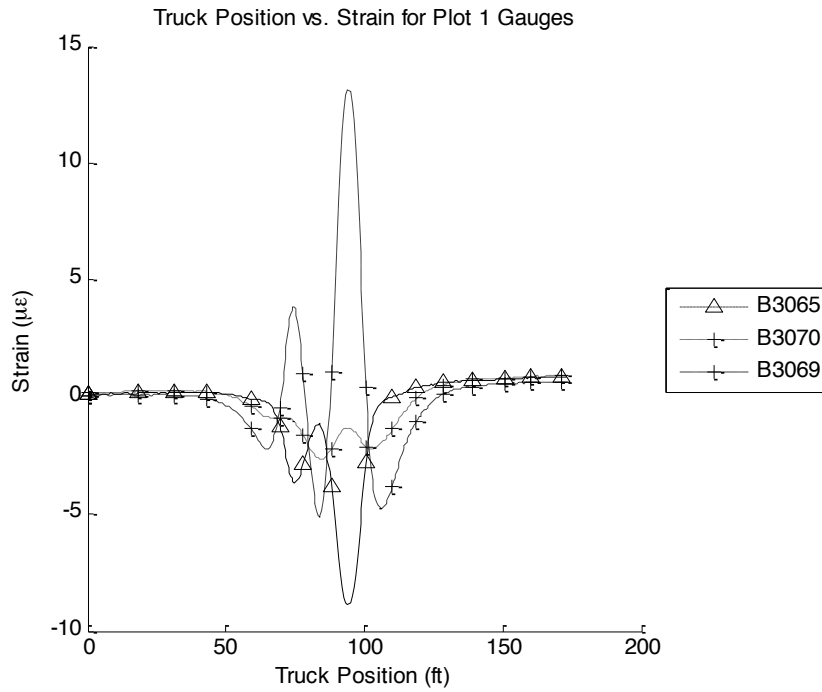


Figure 46: Section 1 Gages

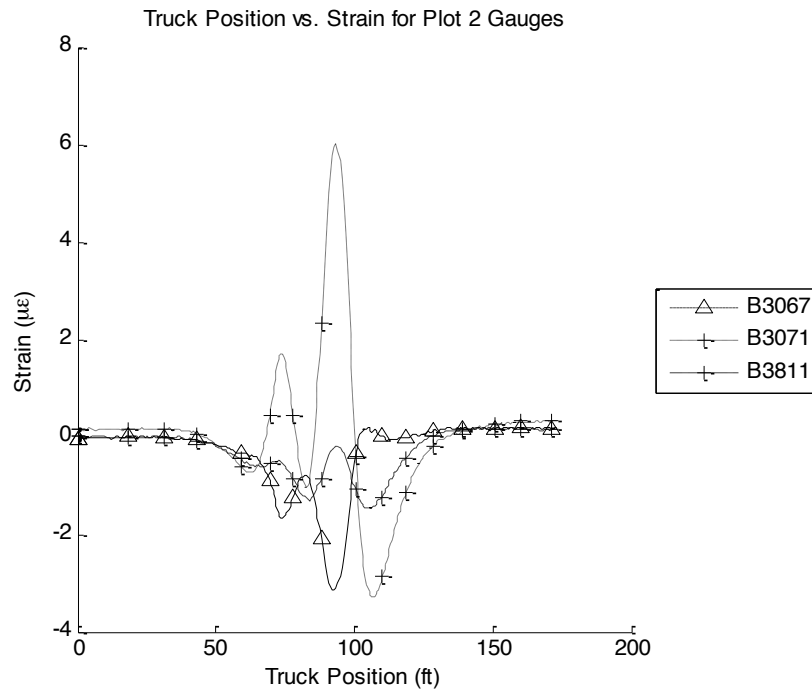


Figure 47: Section 2 Gages

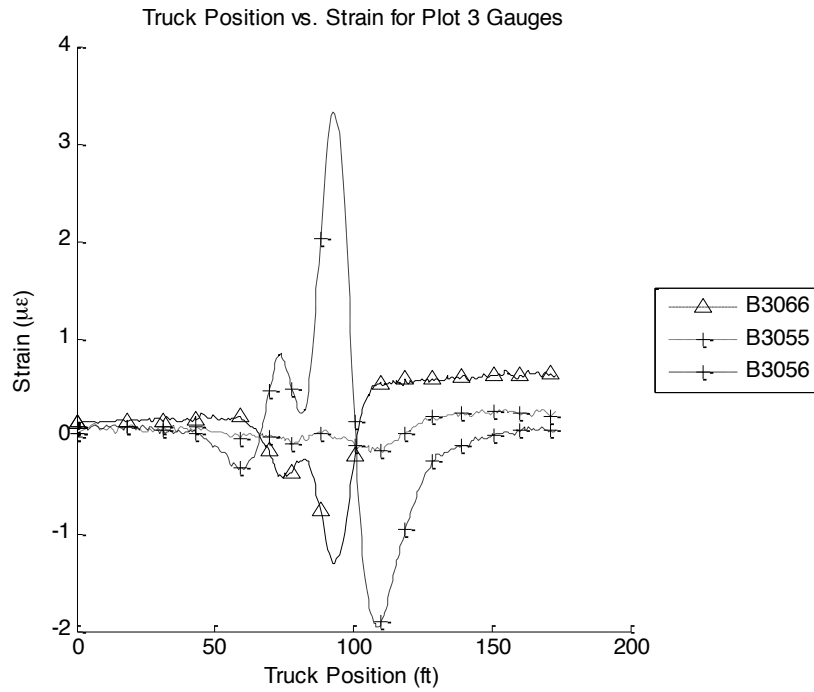


Figure 48: Section 3 Gages

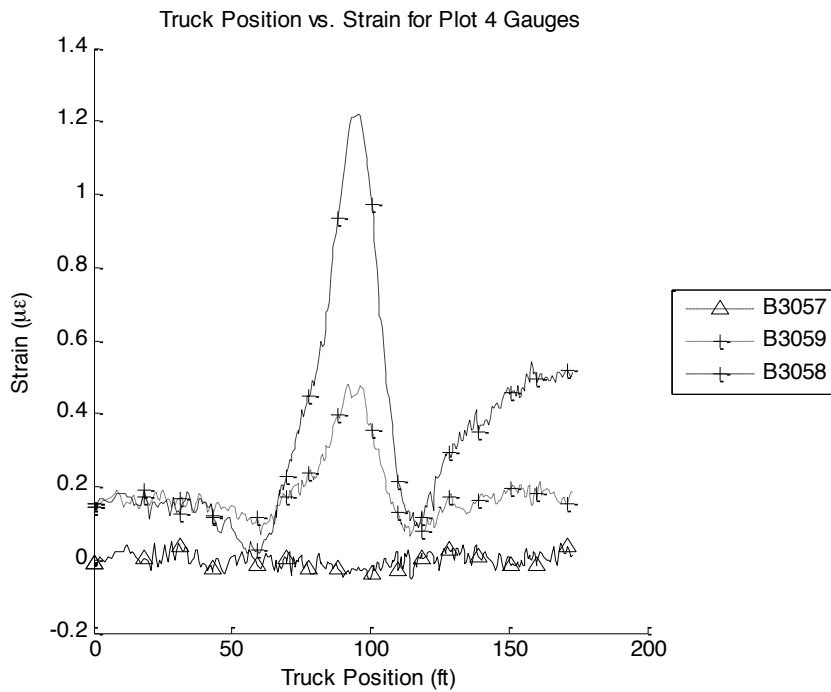


Figure 49: Section 4 Gages

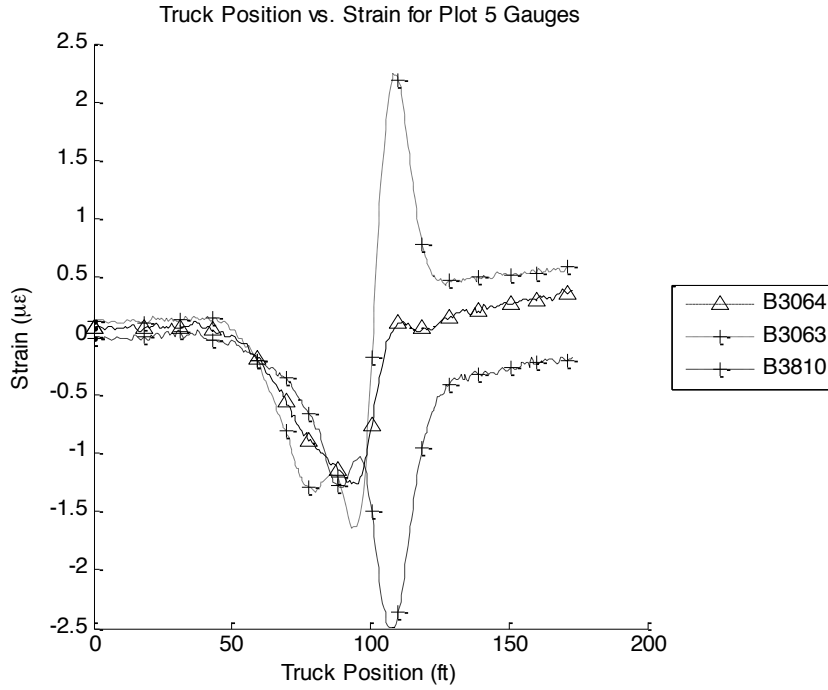


Figure 50: Section 5 Gages

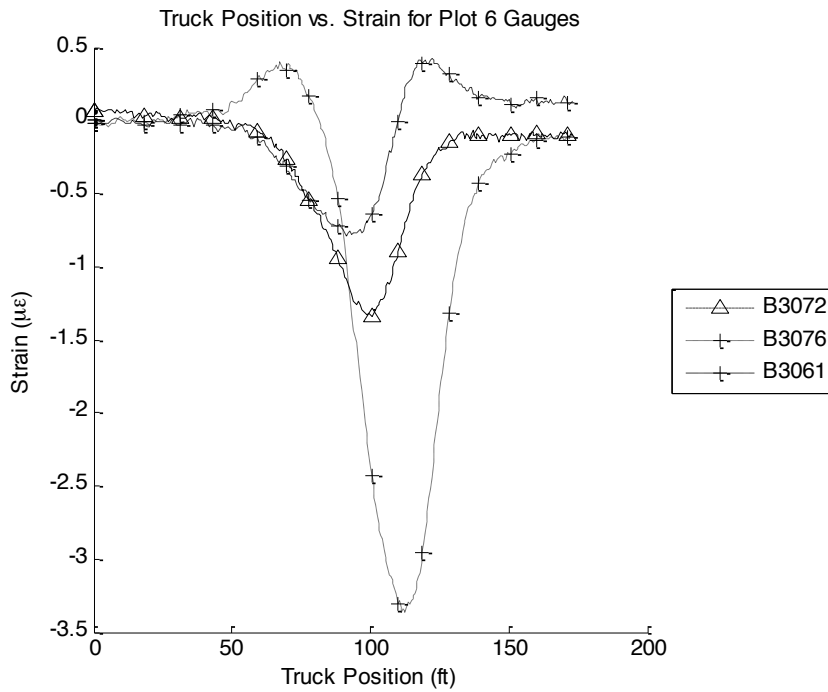


Figure 51: Section 6 Gages

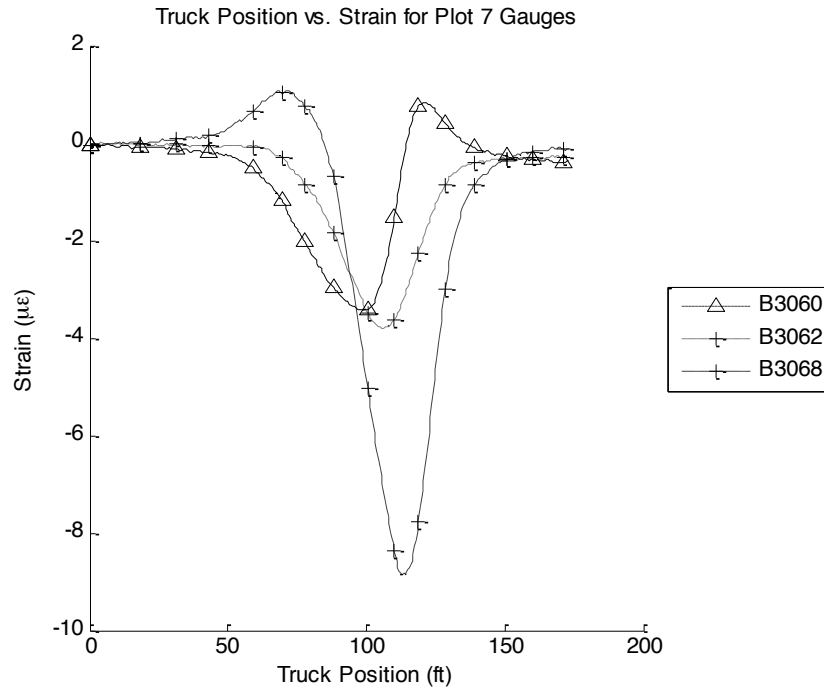


Figure 52: Section 7 Gages

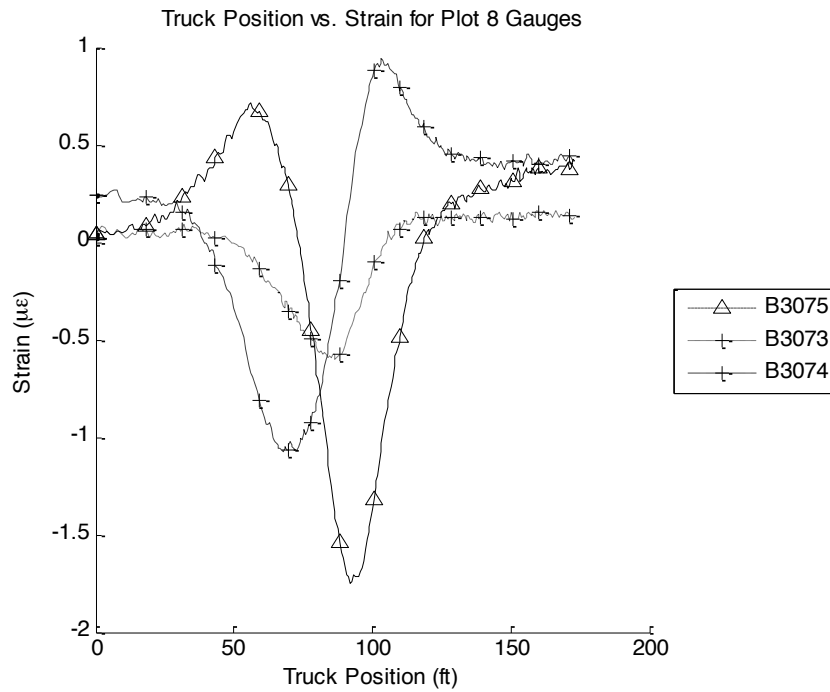


Figure 53: Section 8 Gages

Y2 Strain Graphs

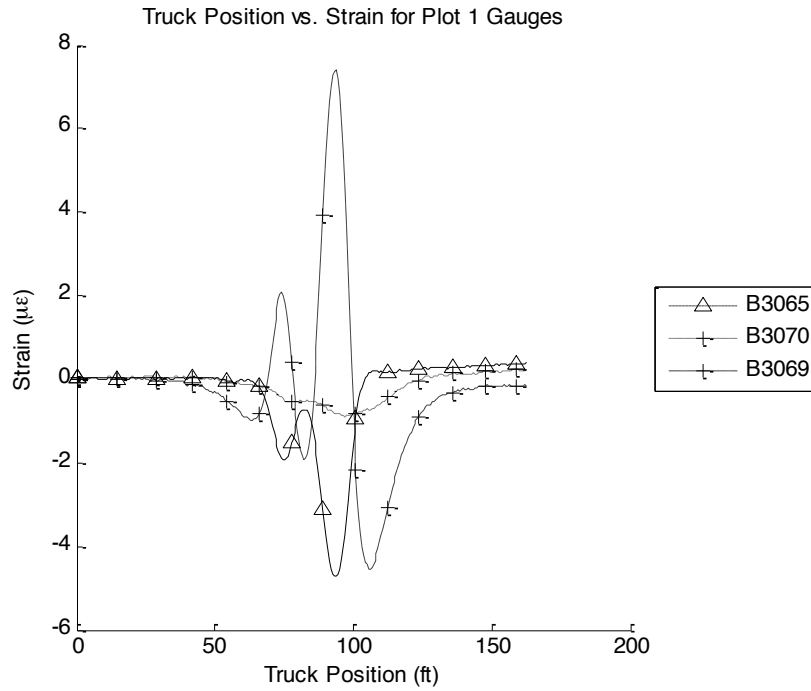


Figure 54: Section 1 Gages

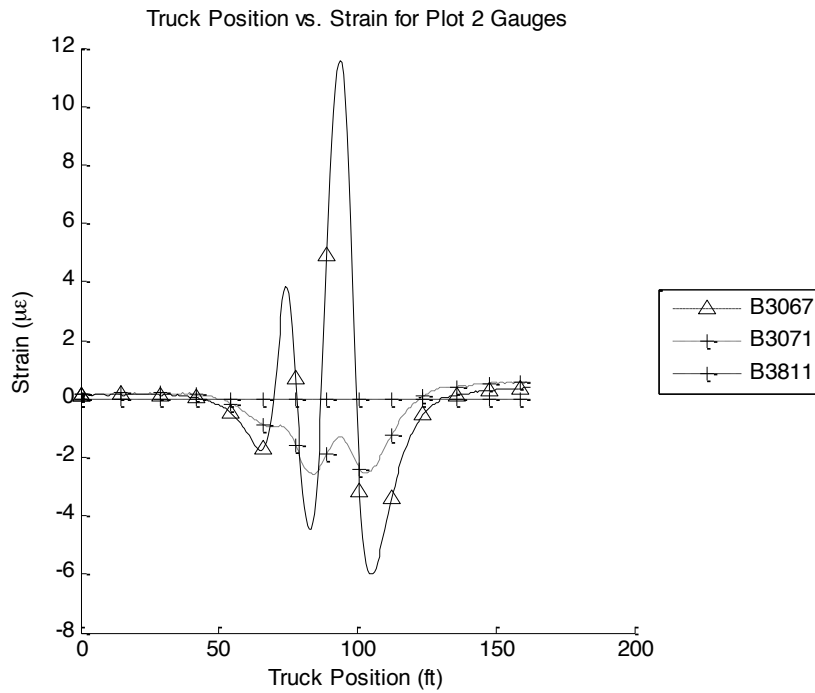


Figure 55: Section 2 Gages

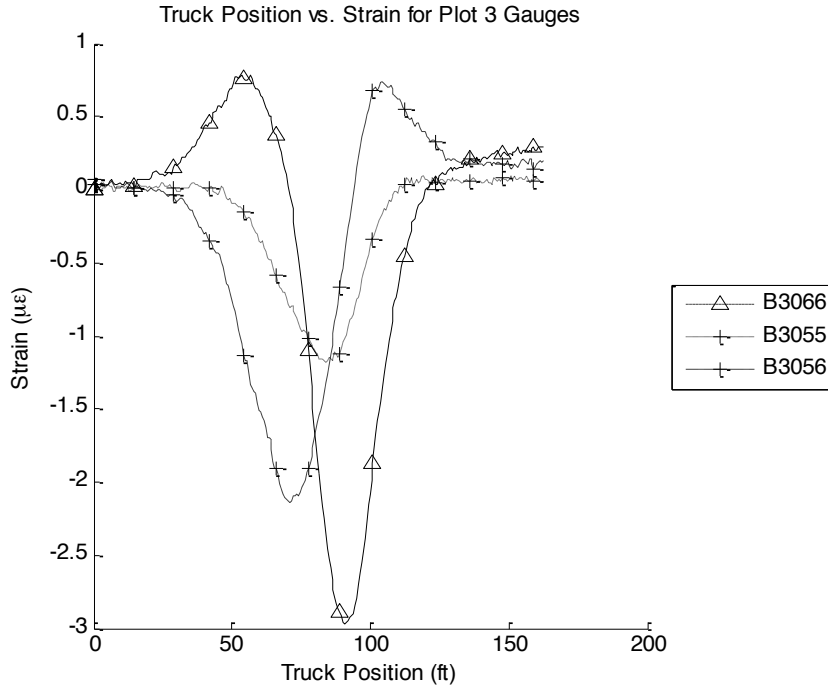


Figure 56: Section 3 Gages

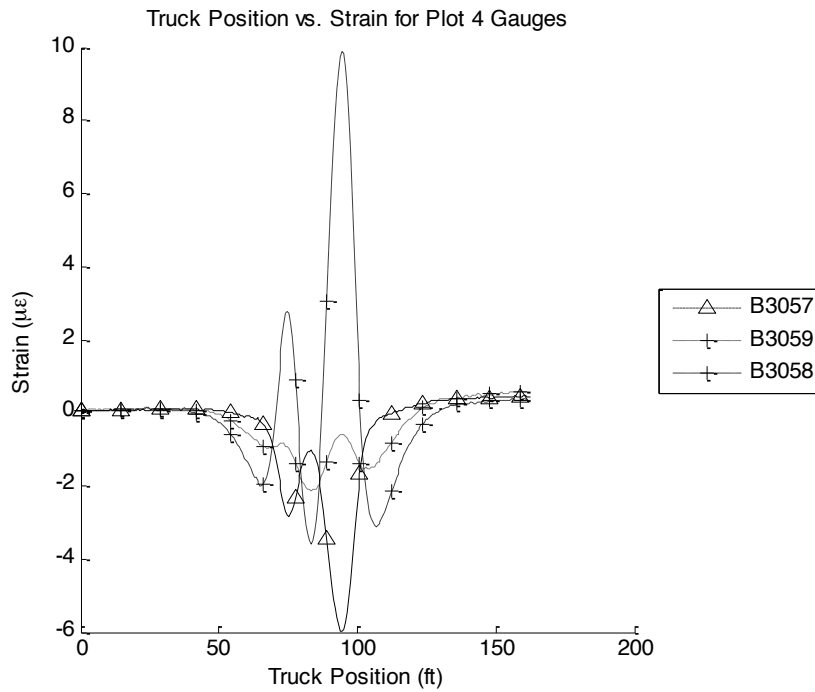


Figure 57: Section 4 Gages

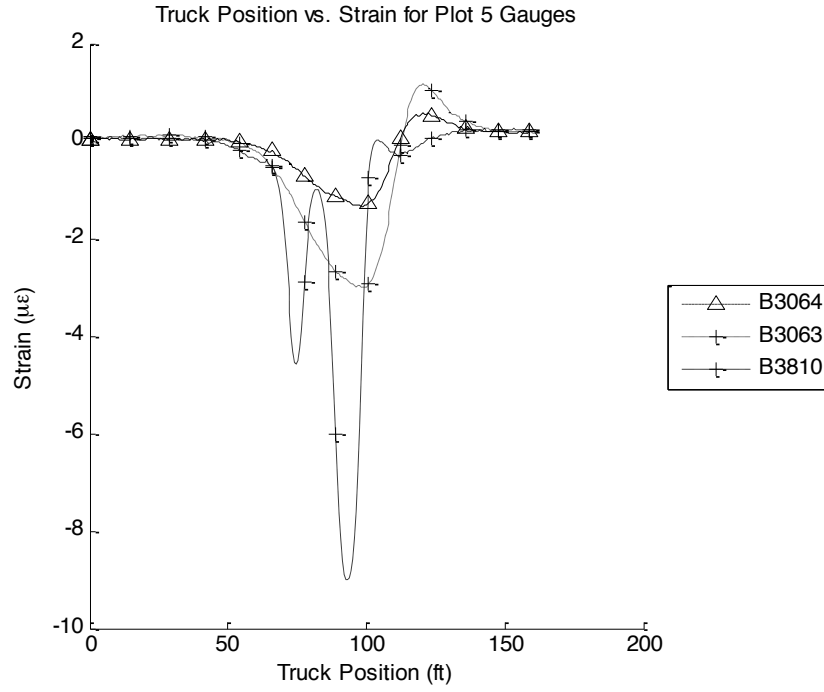


Figure 58: Section 5 Gages

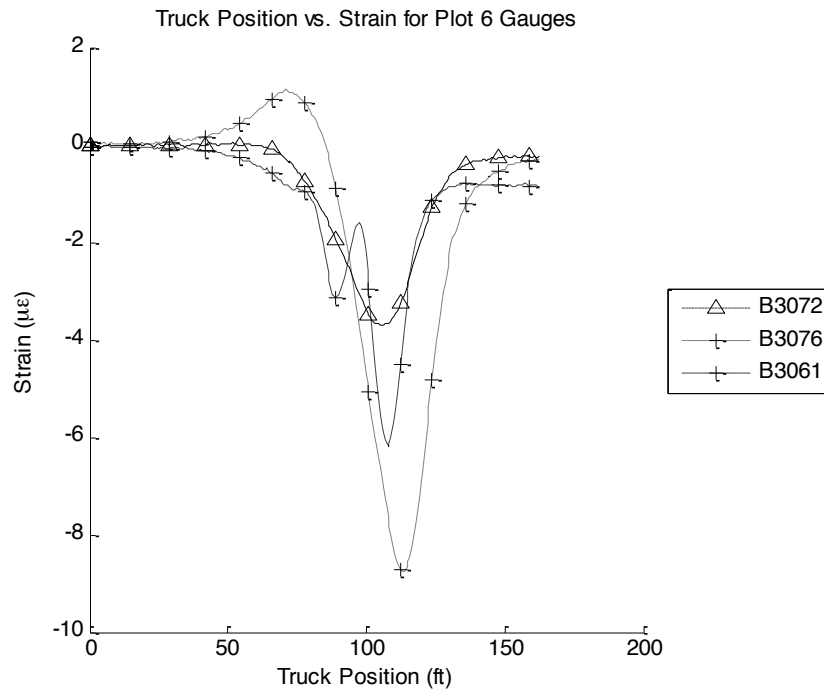


Figure 59: Section 6 Gages

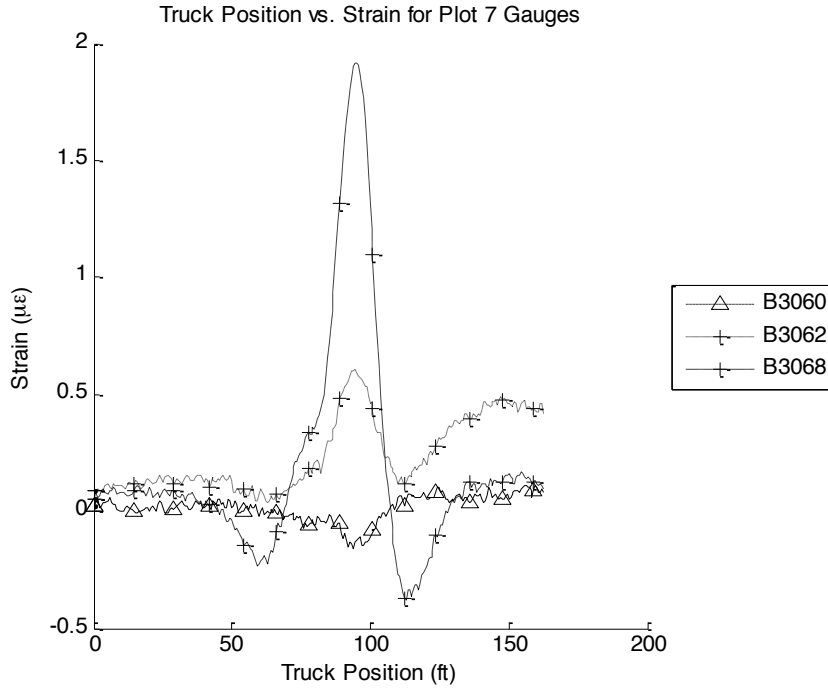


Figure 60: Section 7 Gauges

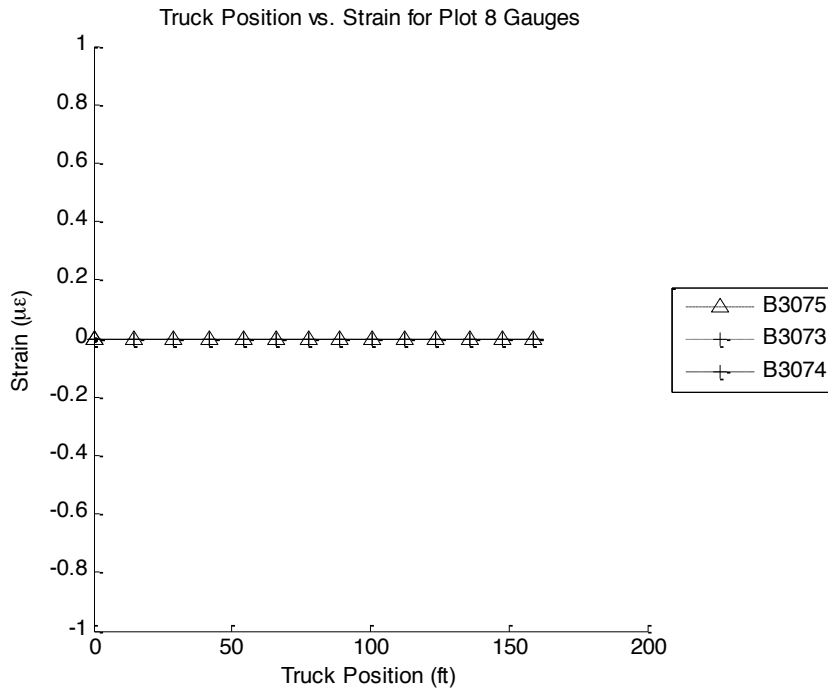


Figure 61: Section 8 Gauges

Y3 Strain Graphs:

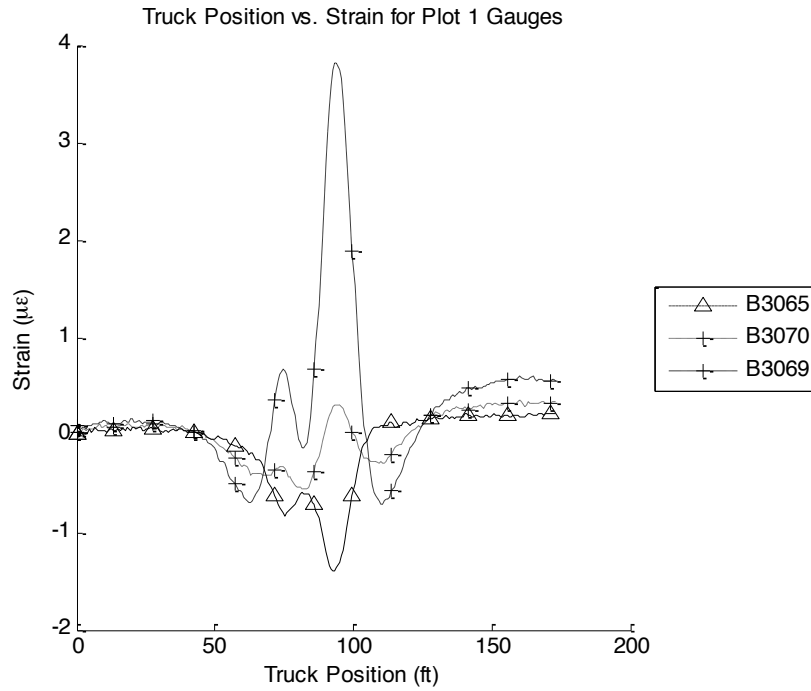


Figure 62: Section 1 Gages

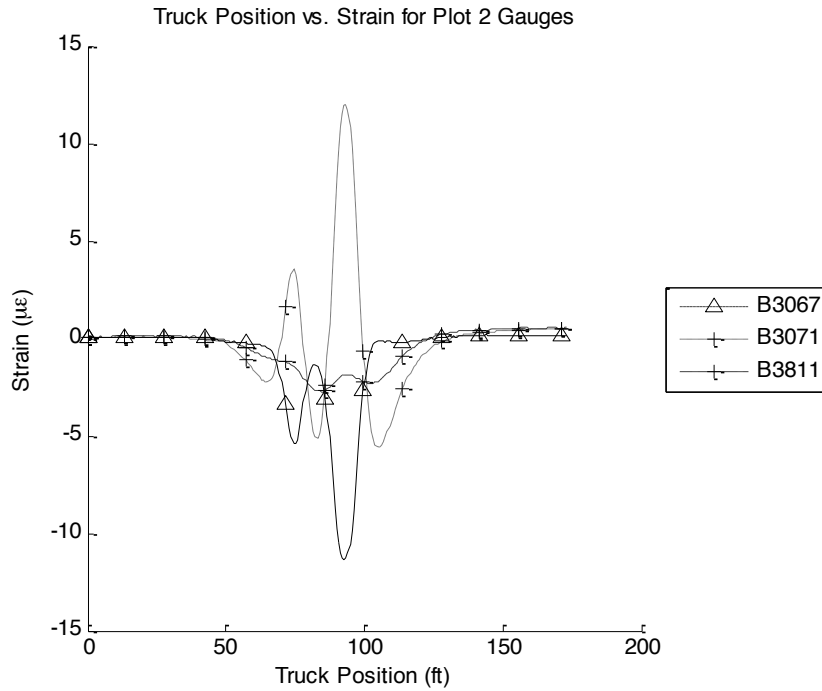


Figure 63: Section 2 Gages

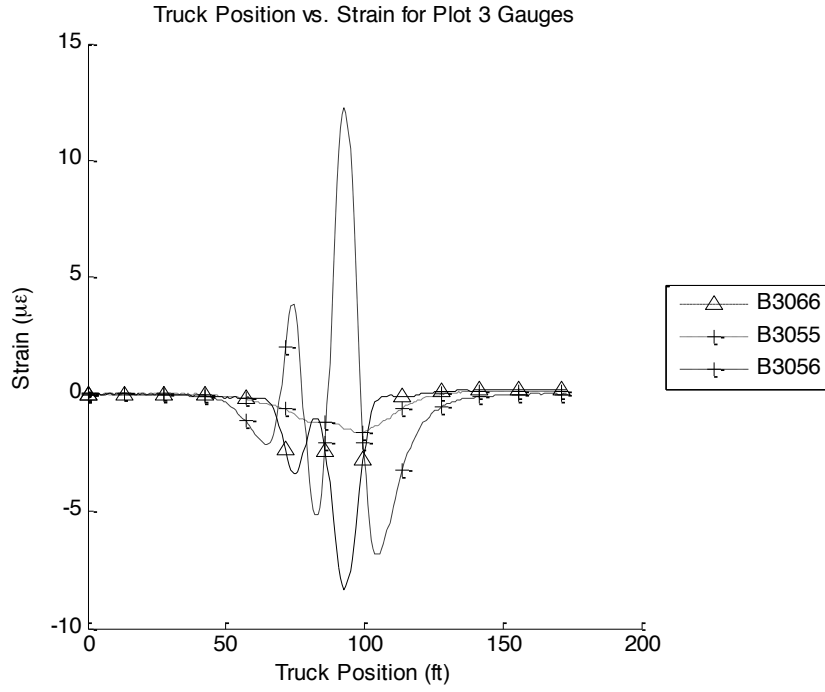


Figure 64: Section 3 Gages

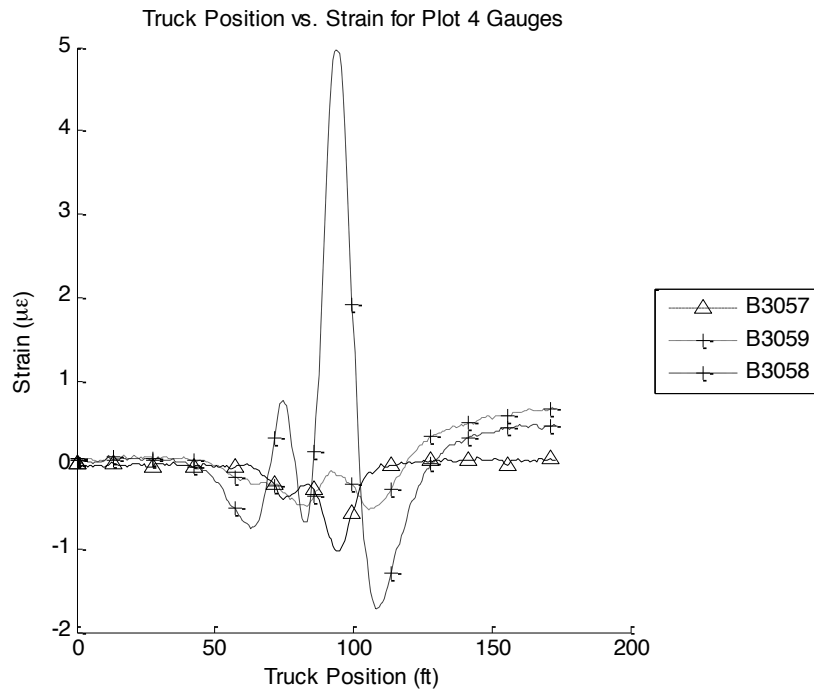


Figure 65: Section 4 Gages

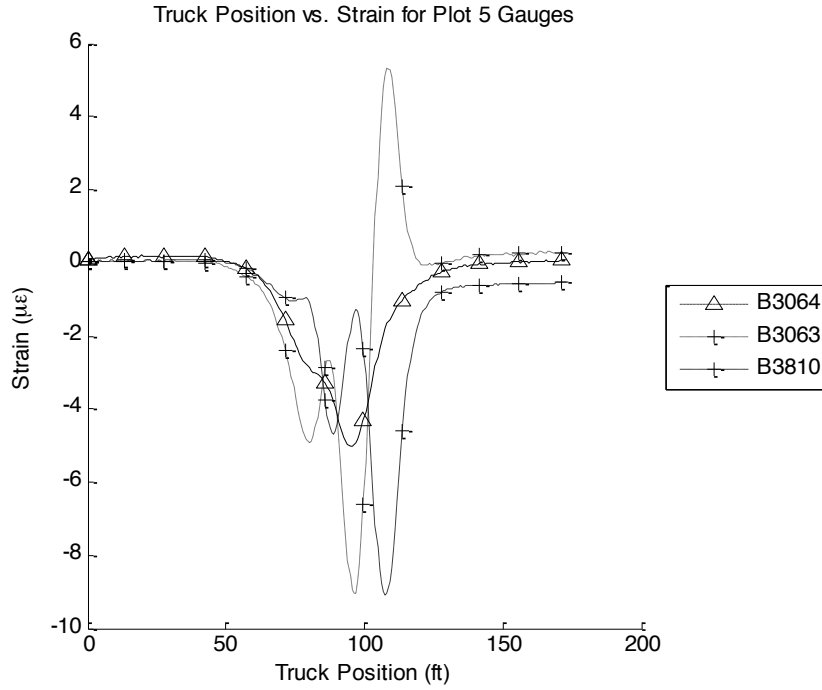


Figure 66: Section 5 Gages

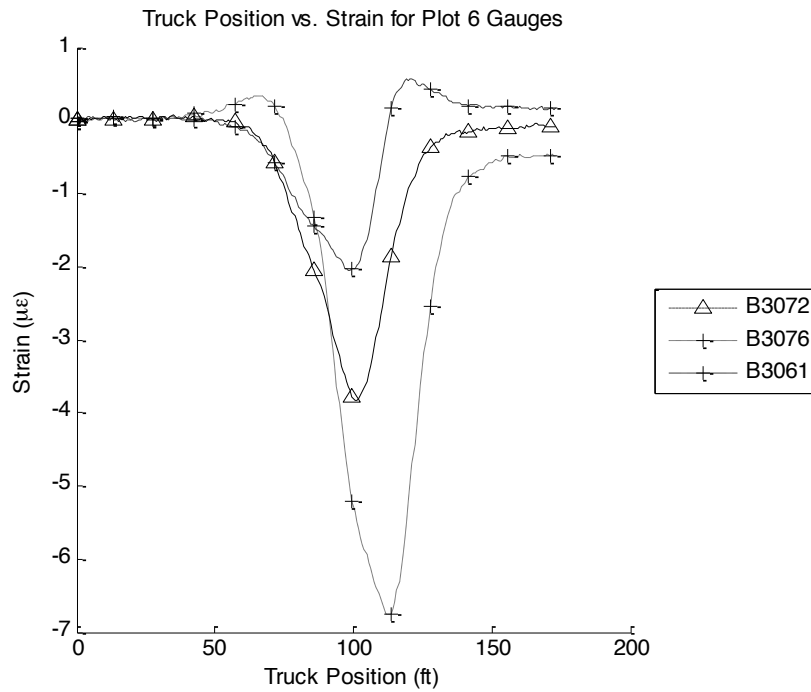


Figure 67: Section 6 Gages

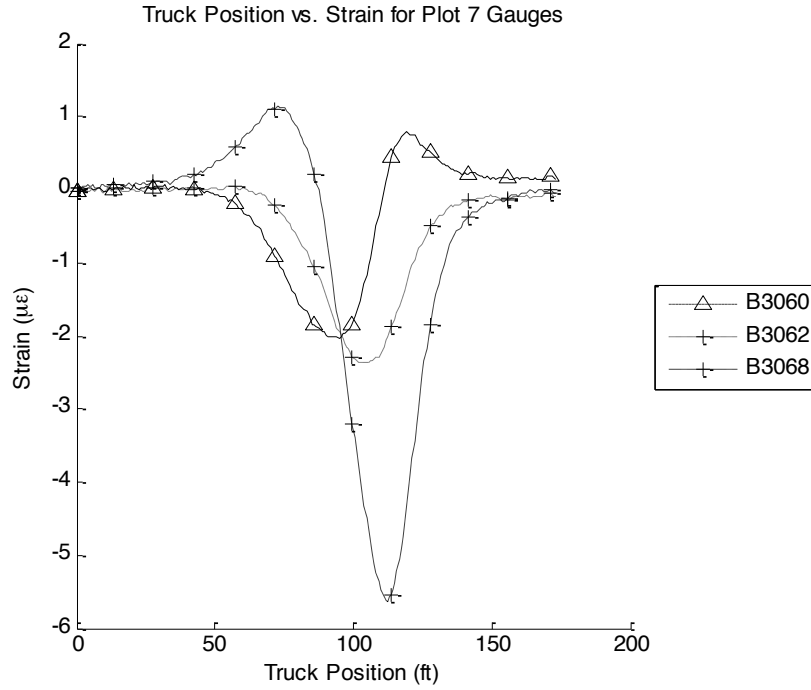


Figure 68: Section 7 Gauges

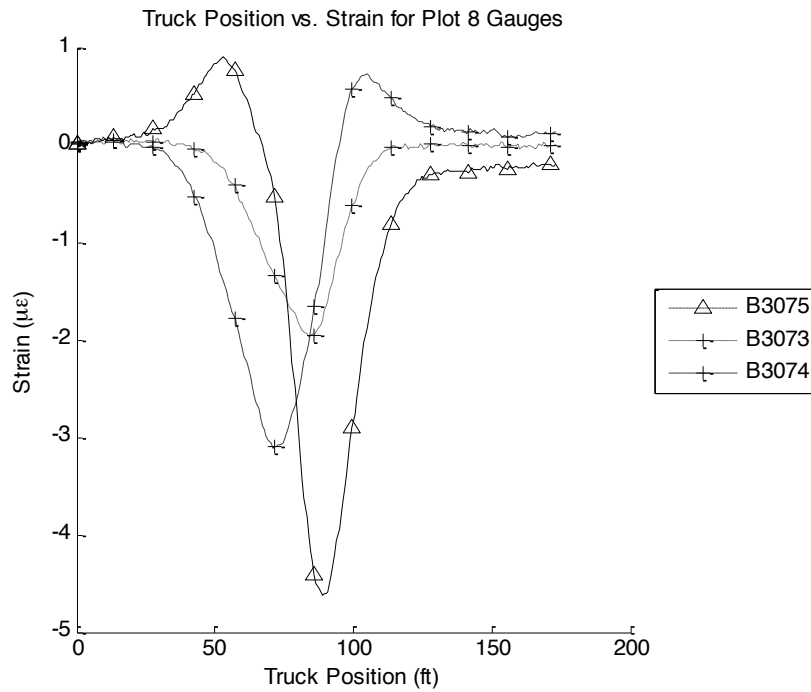


Figure 69: Section 8 Gauges

Y4 Strain Graphs

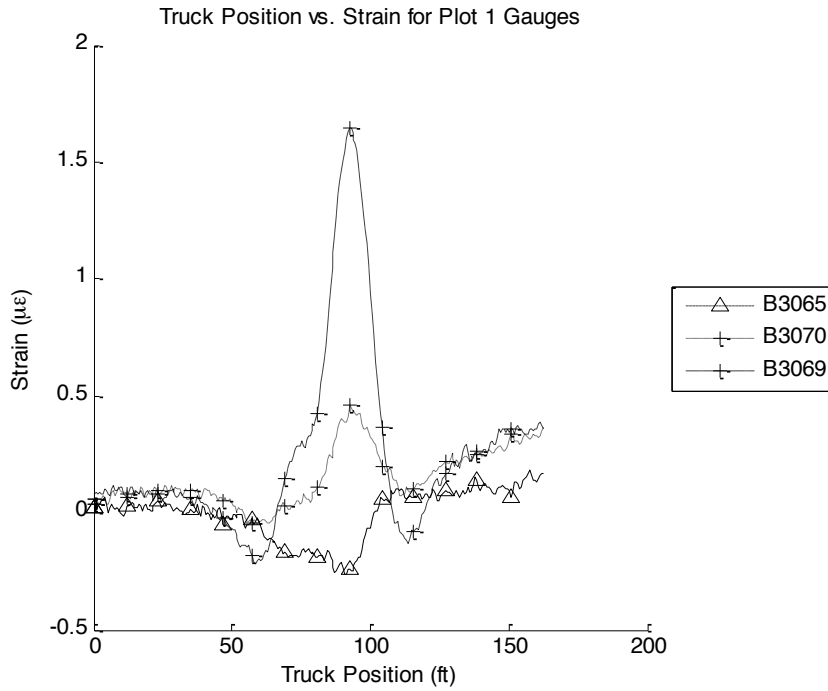


Figure 70: Section 1 Gages

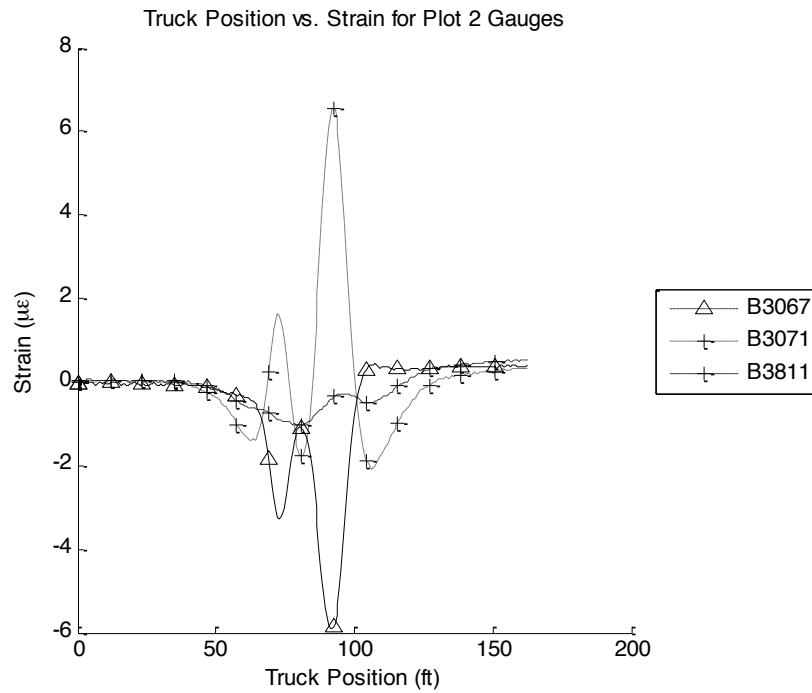


Figure 71: Section 2 Gages

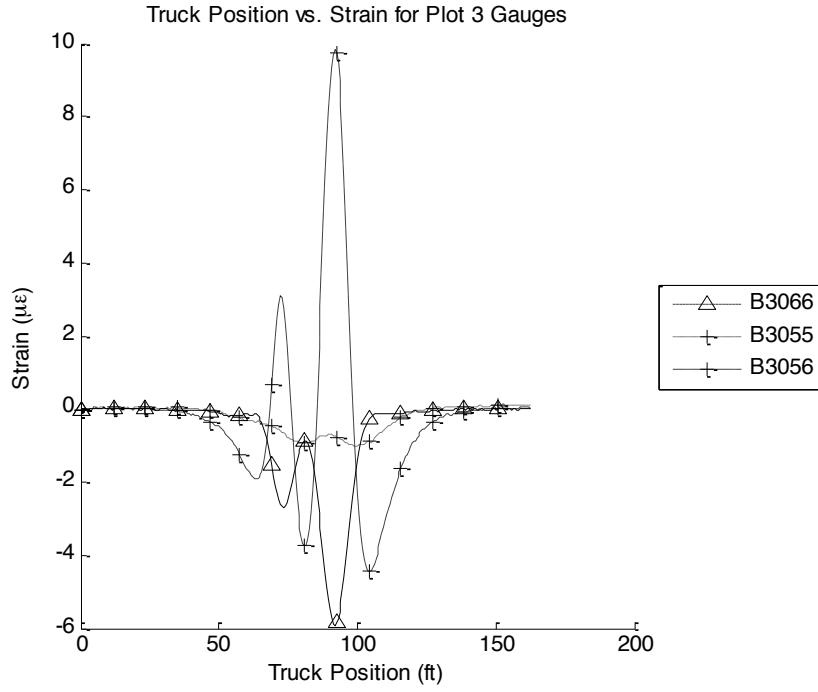


Figure 72: Section 3 Gages

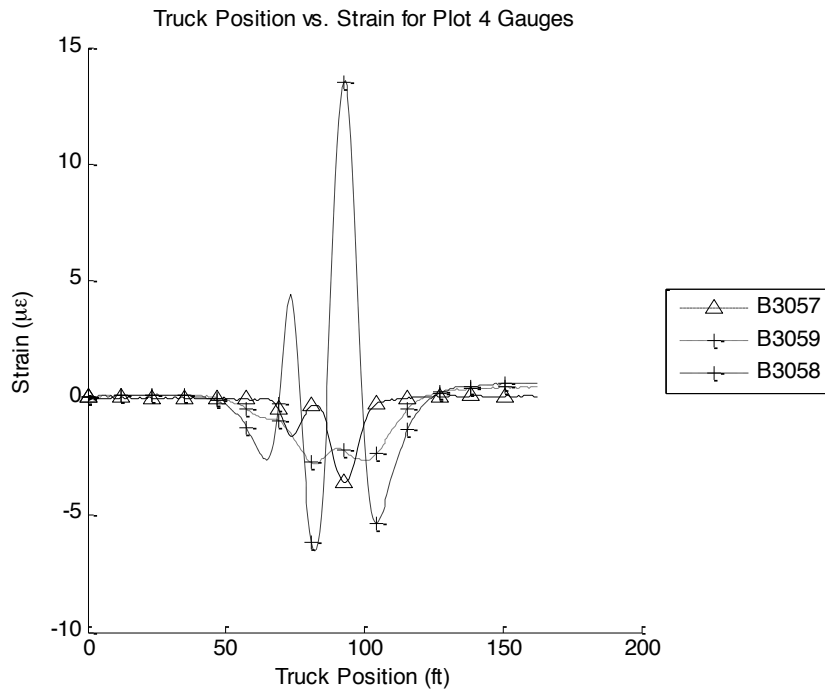


Figure 73: Section 4 Gages

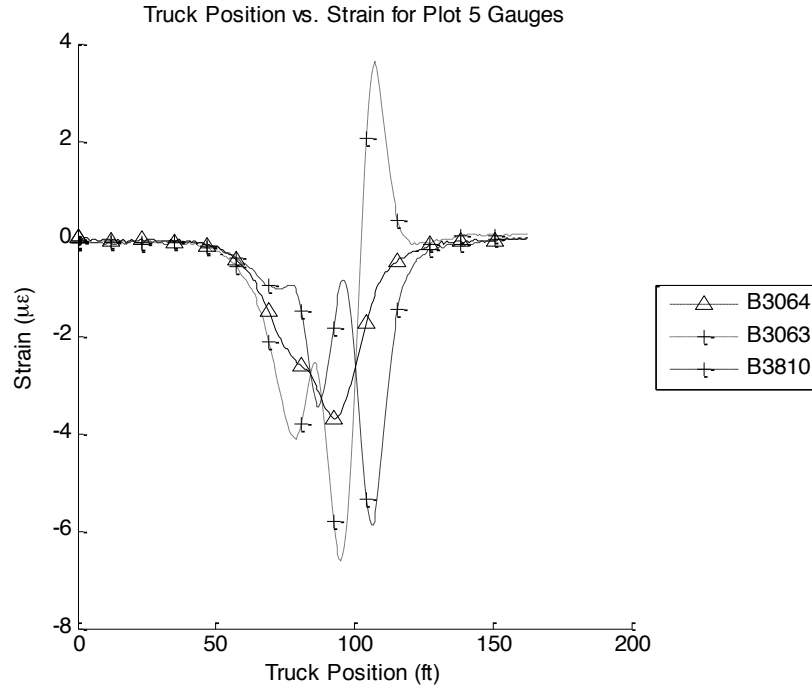


Figure 74: Section 5 Gages

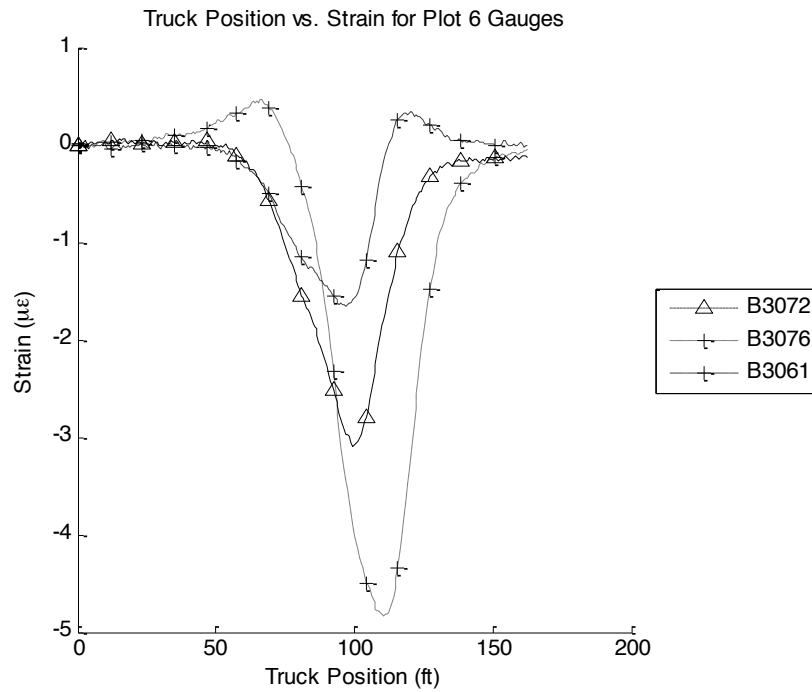


Figure 75: Section 6 Gages

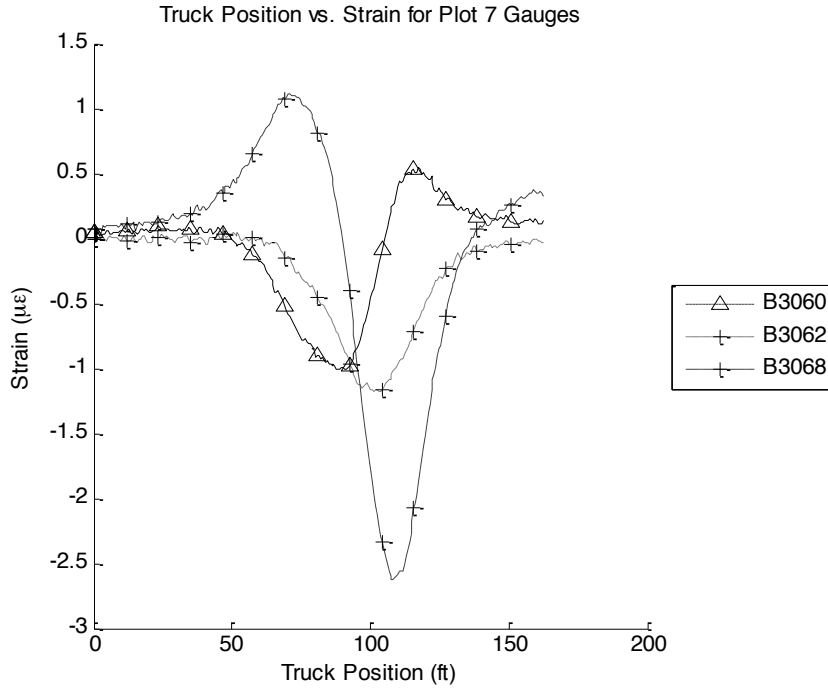


Figure 76: Section 7 Gages

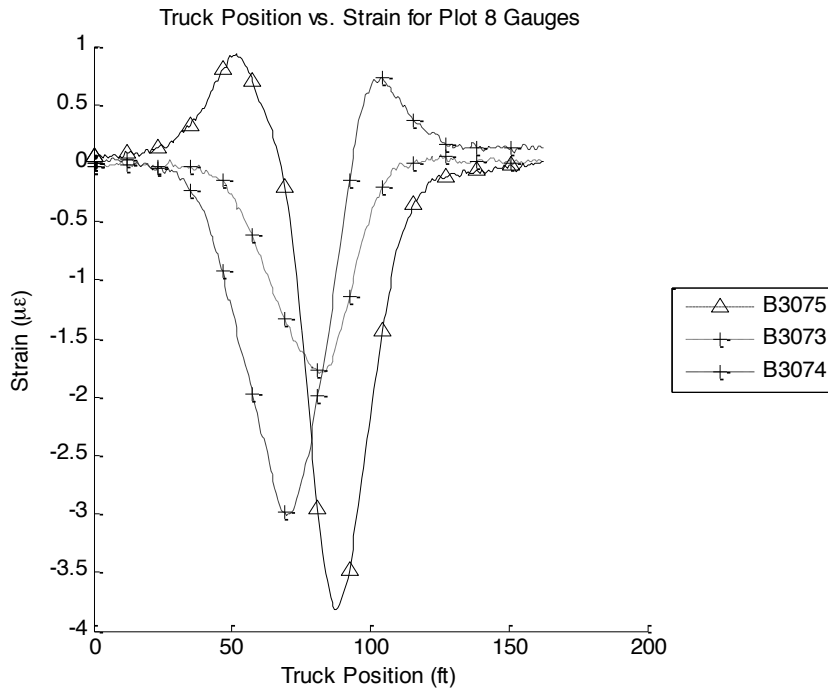


Figure 77: Section 8 Gages

Y5 Strain Graphs:

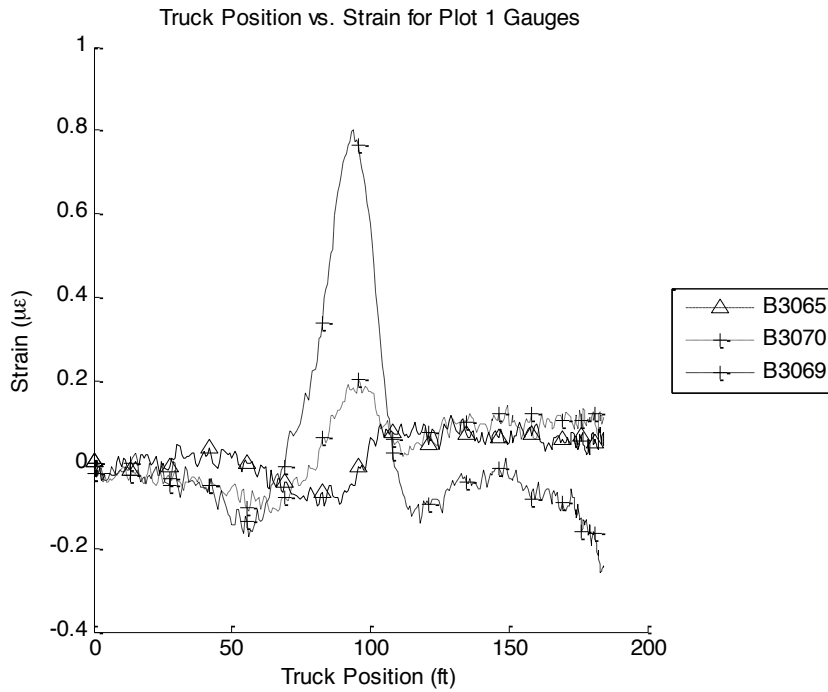


Figure 78: Section 1 Gages

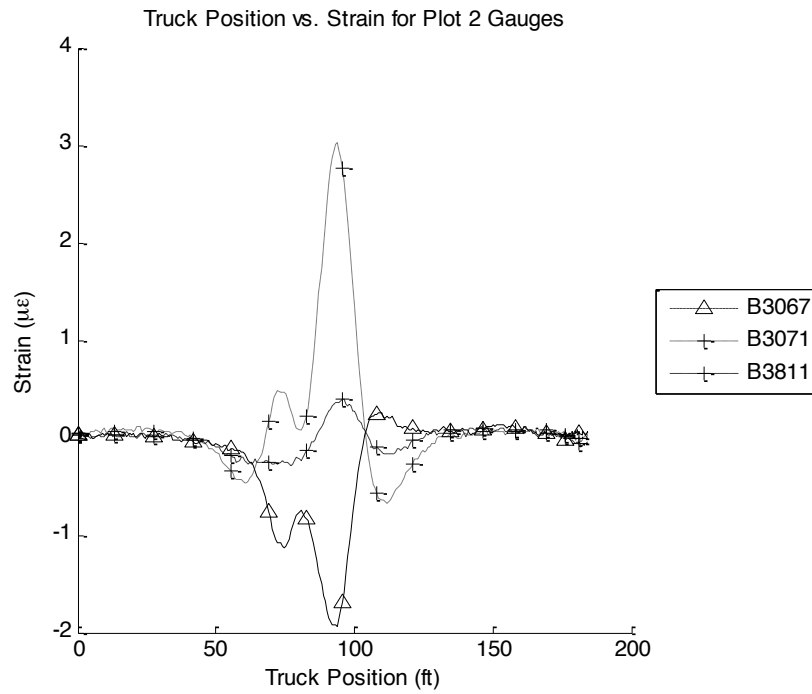


Figure 79: Section 2 Gages

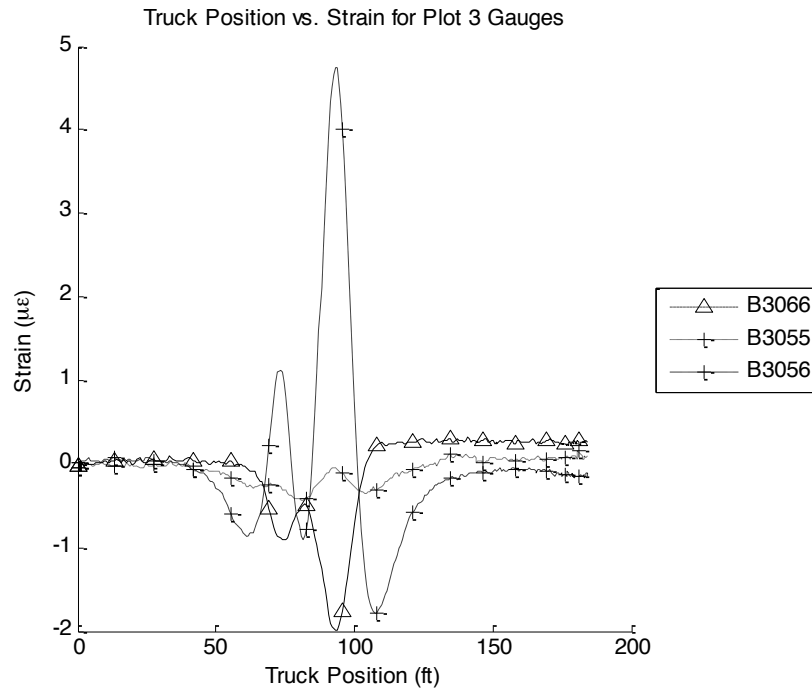


Figure 80: Section 3 Gages

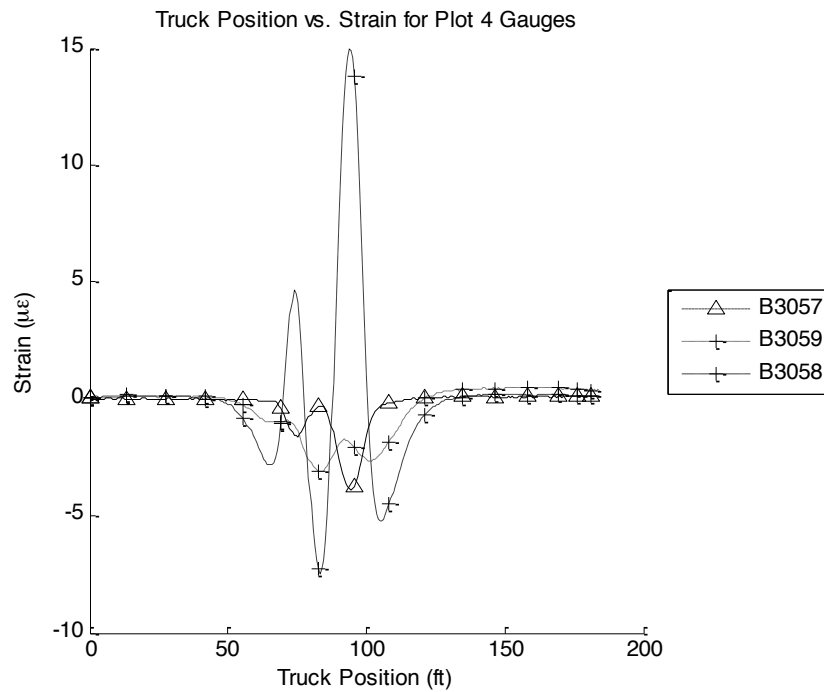


Figure 81: Section 4 Gages

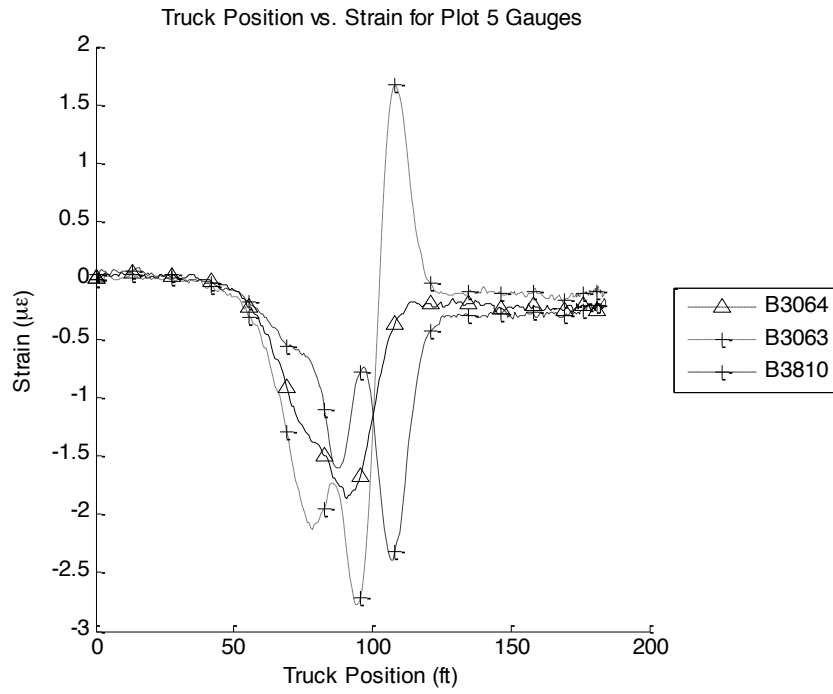


Figure 82: Section 5 Gages

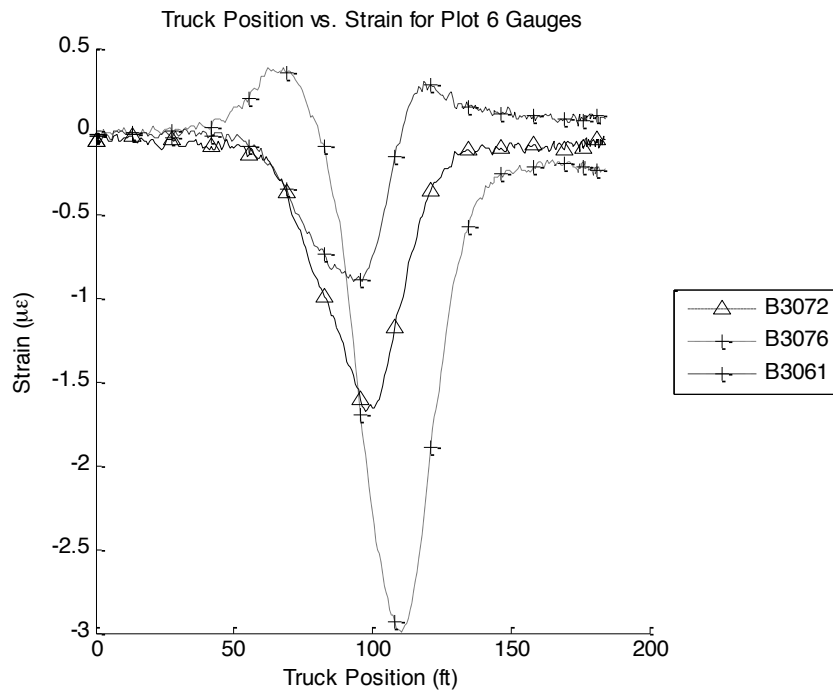


Figure 83: Section 6 Gages

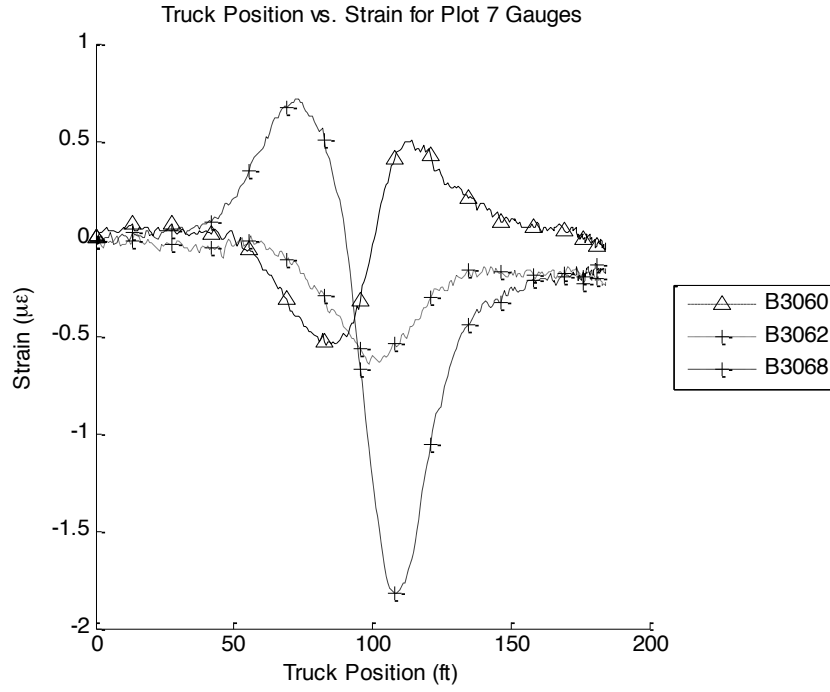


Figure 84: Section 7 Gauges

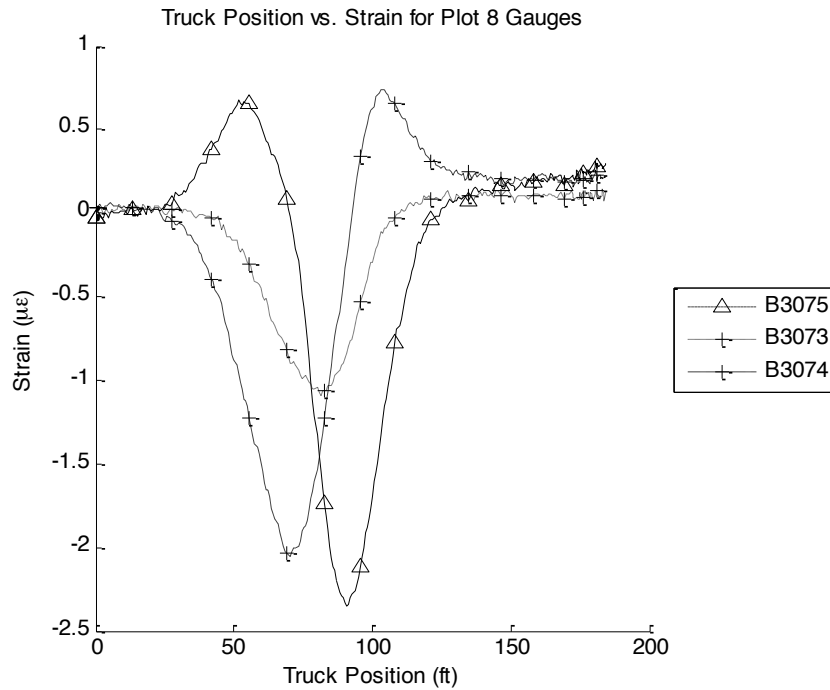


Figure 85: Section 8 Gauges

Y6 Strain Graphs:

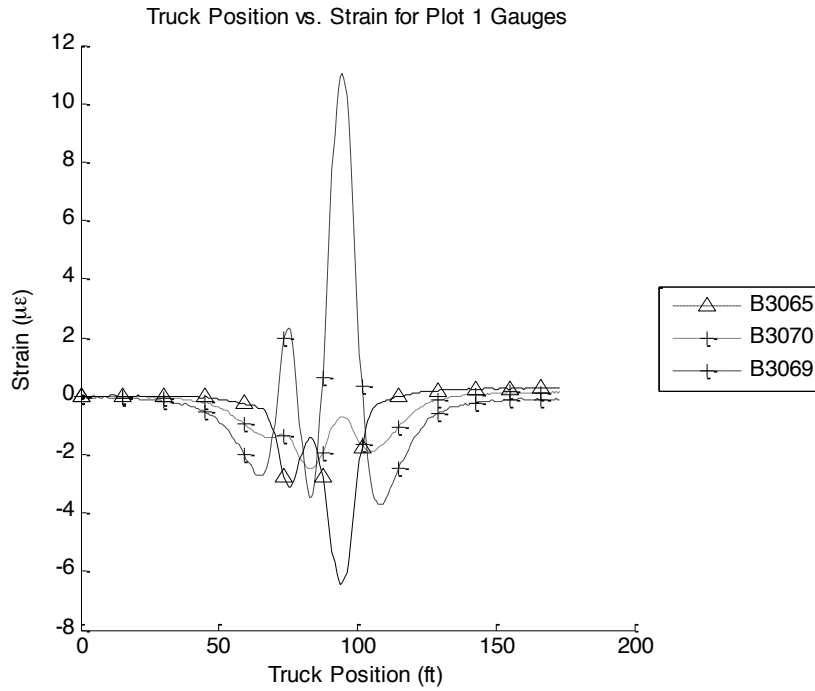


Figure 86: Section 1 Gages

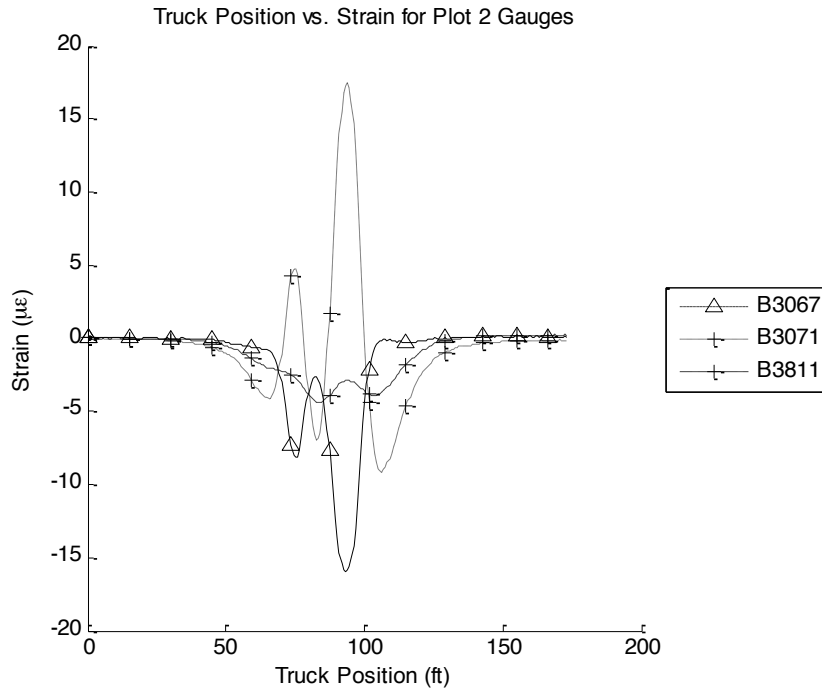


Figure 87: Section 2 Gages

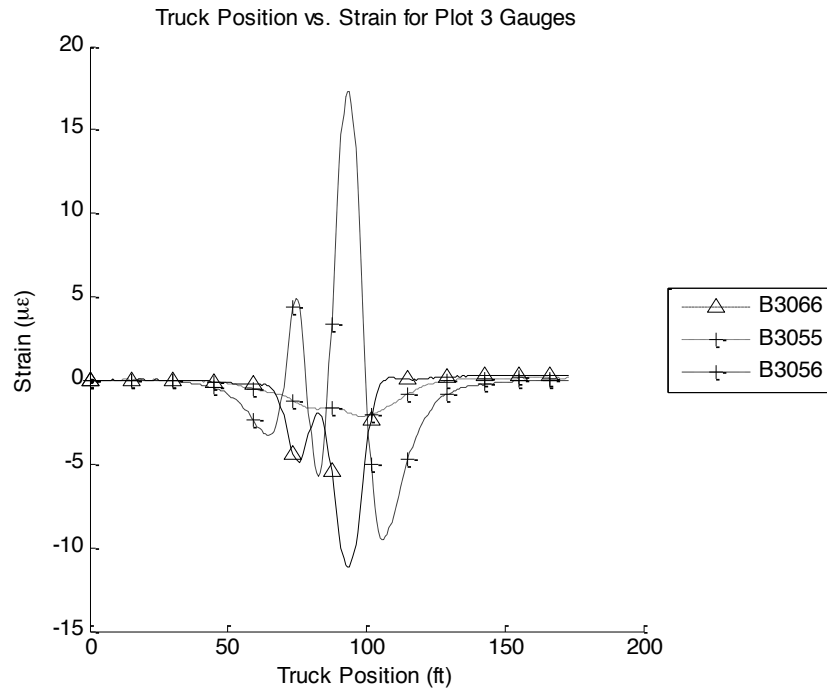


Figure 88: Section 3 Gages

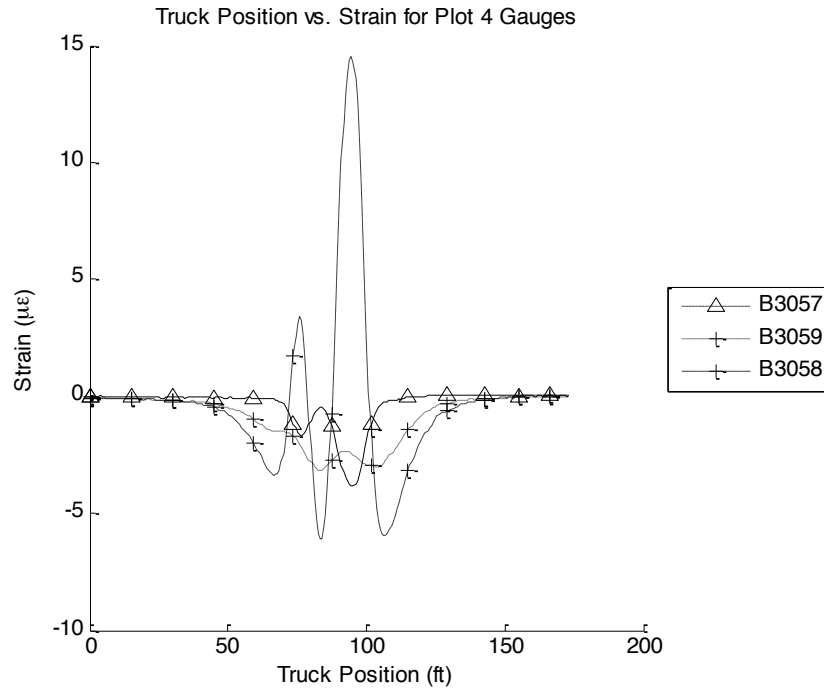


Figure 89: Section 4 Gages

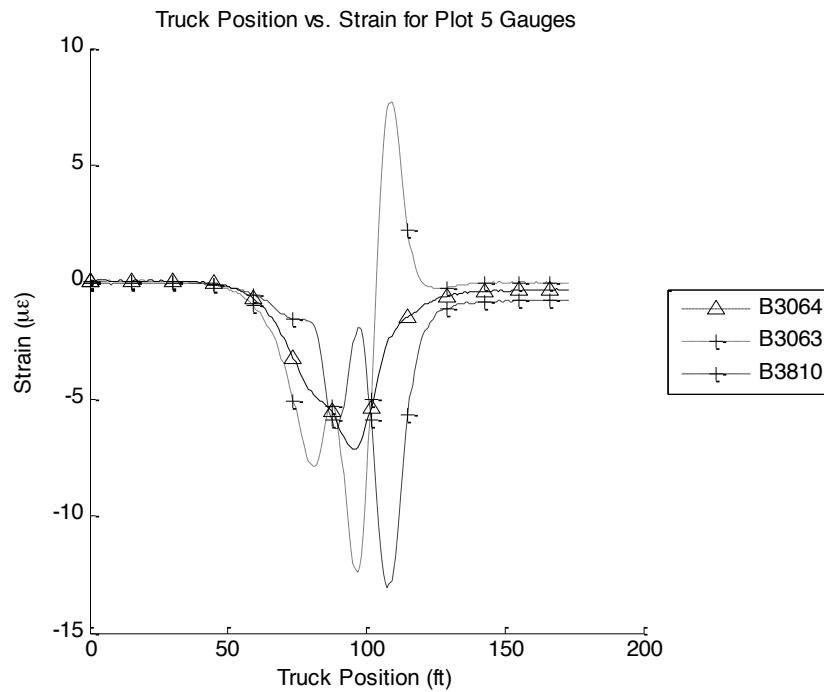


Figure 90: Section 5 Gages

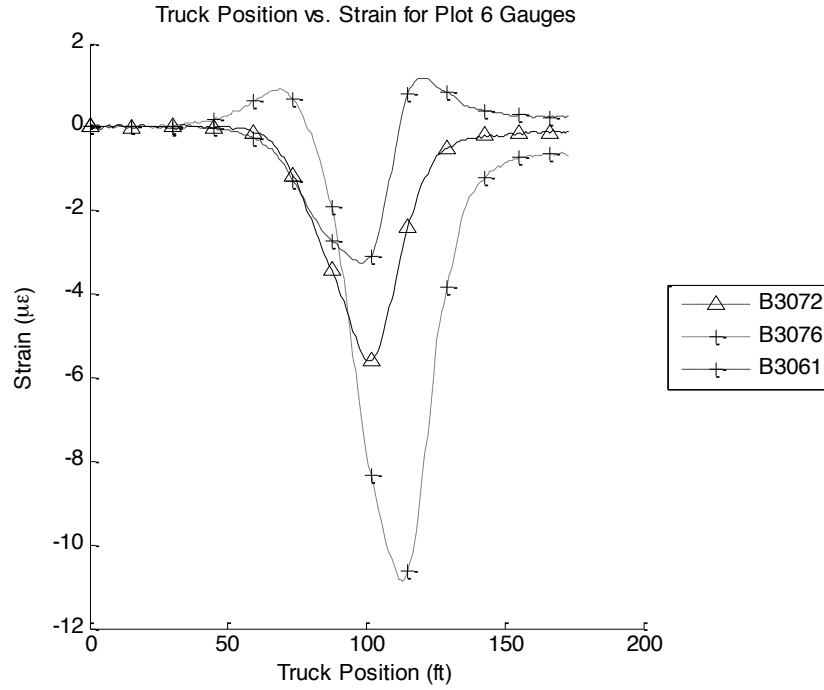


Figure 91: Section 6 Gauges

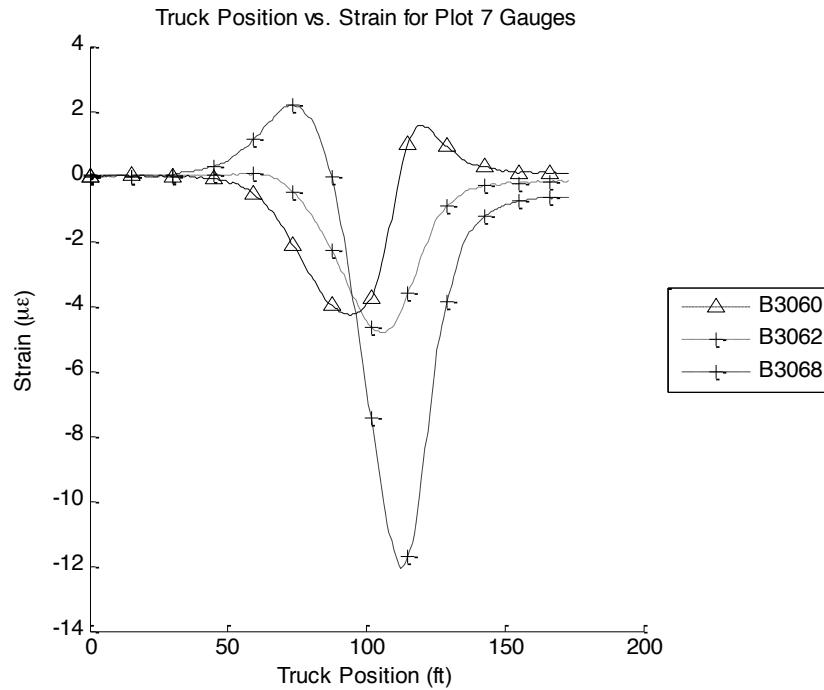


Figure 92: Section 7 Gauges

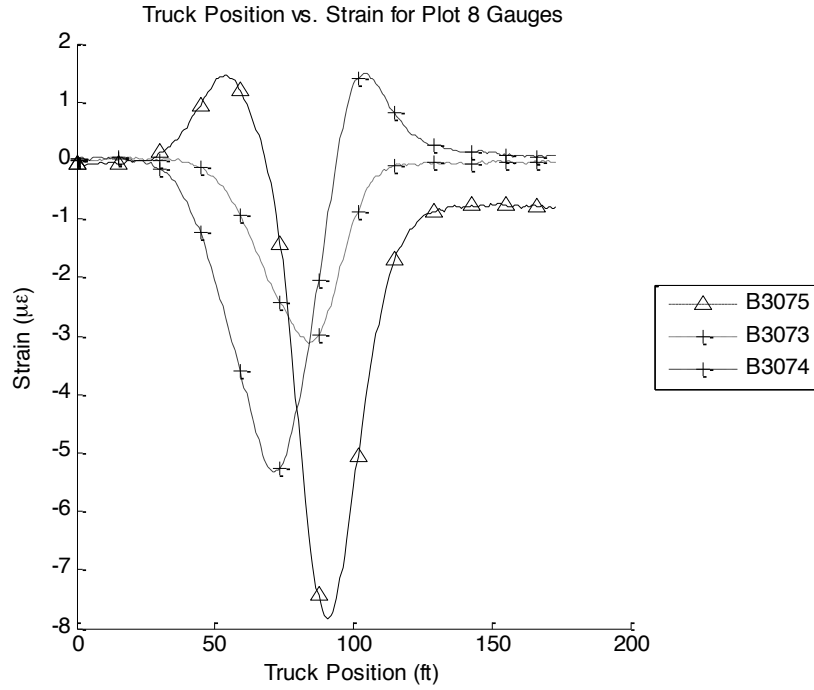


Figure 93: Section 8 Gages

Y7-Y9 Strain Graphs:

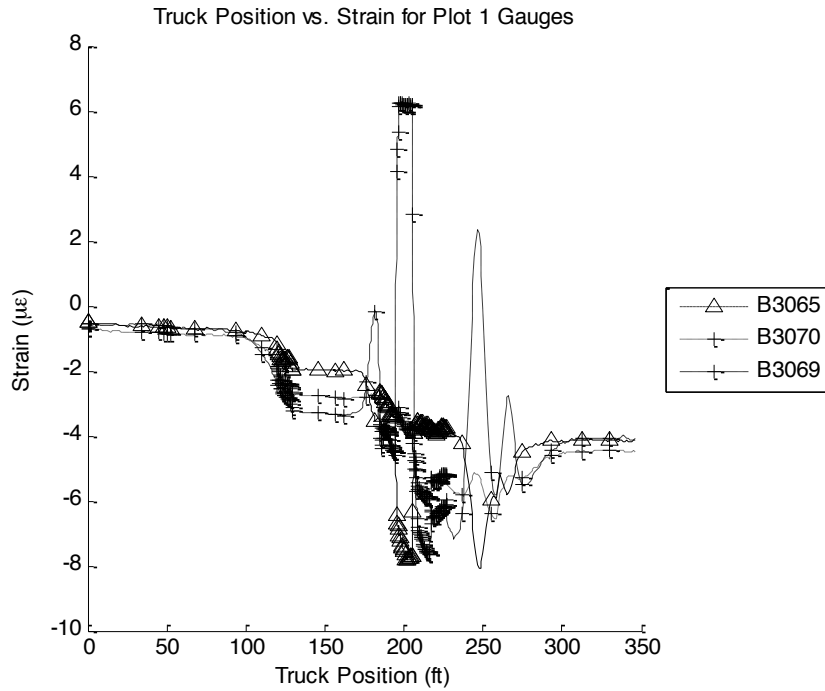


Figure 94: Section 1 Gages

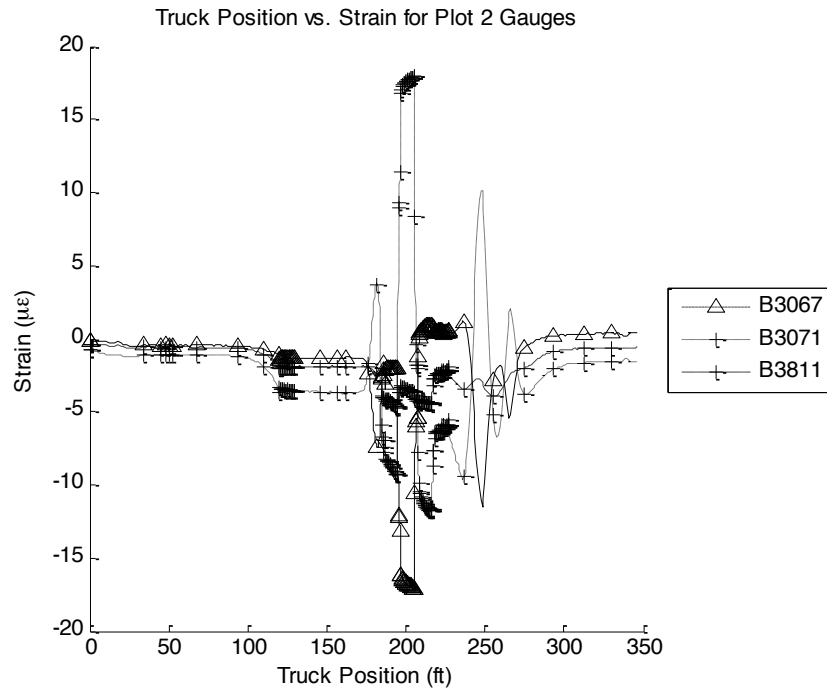


Figure 95: Section 2 Gauges

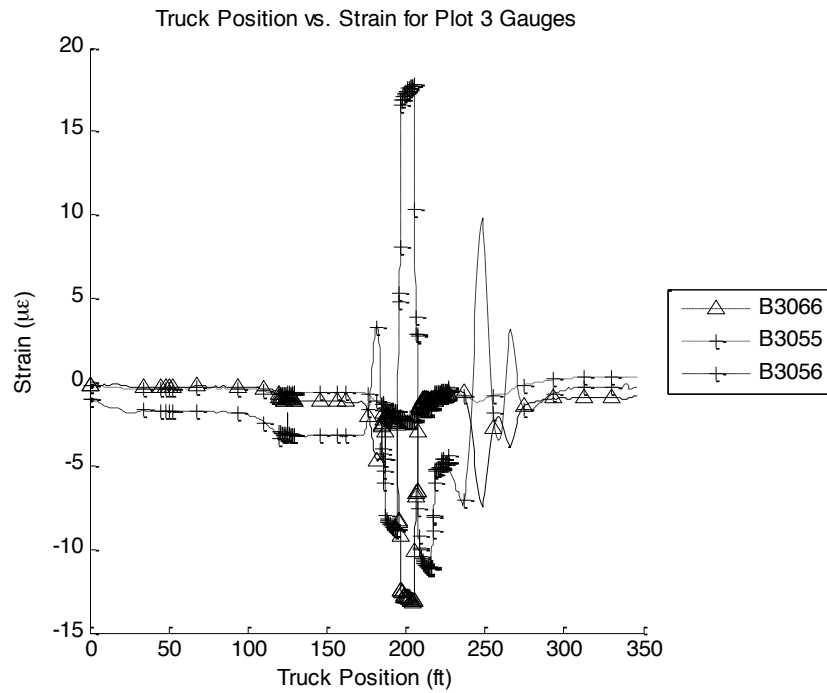


Figure 96: Section 3 Gauges

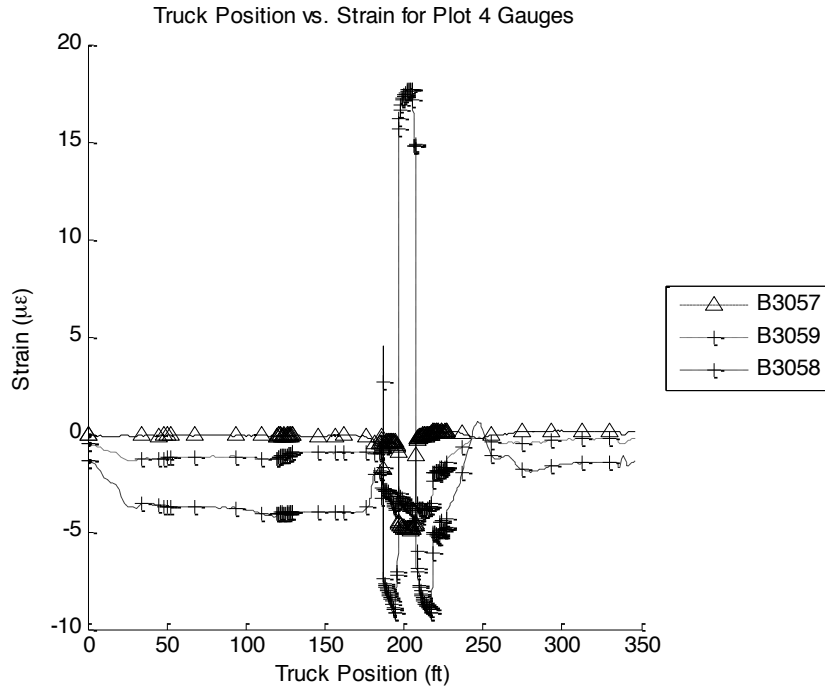


Figure 97: Section 4 Gages

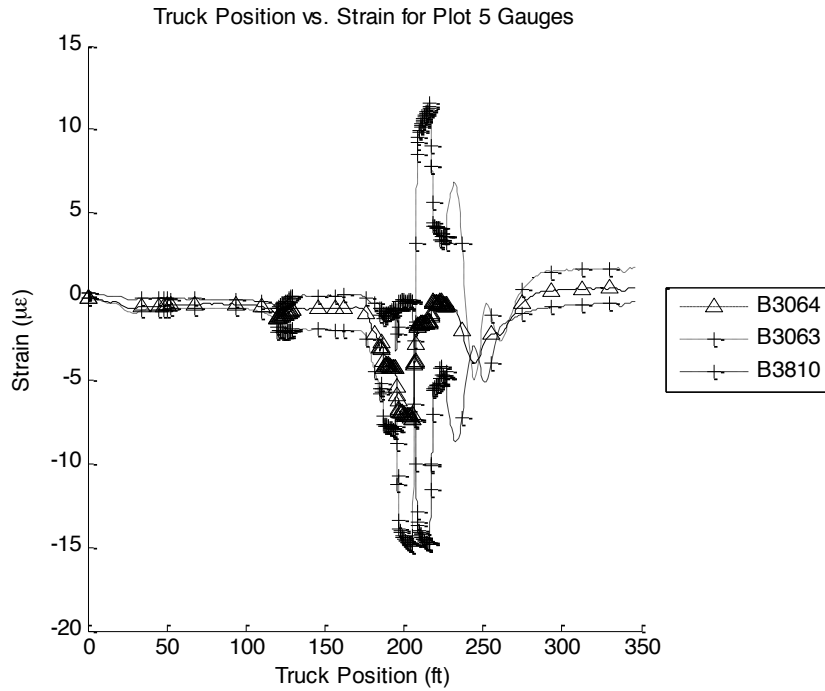


Figure 98: Section 5 Gages

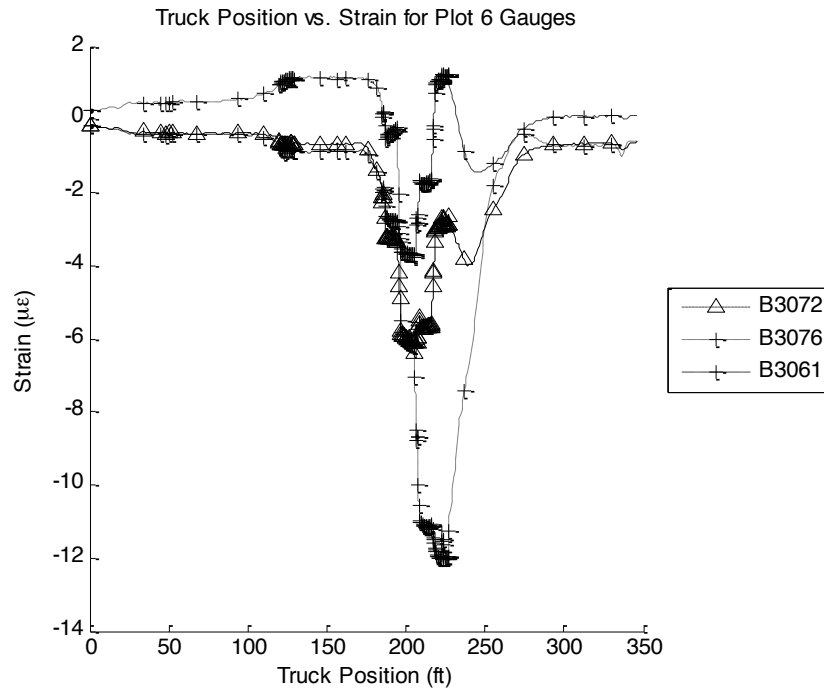


Figure 99: Section 6 Gages

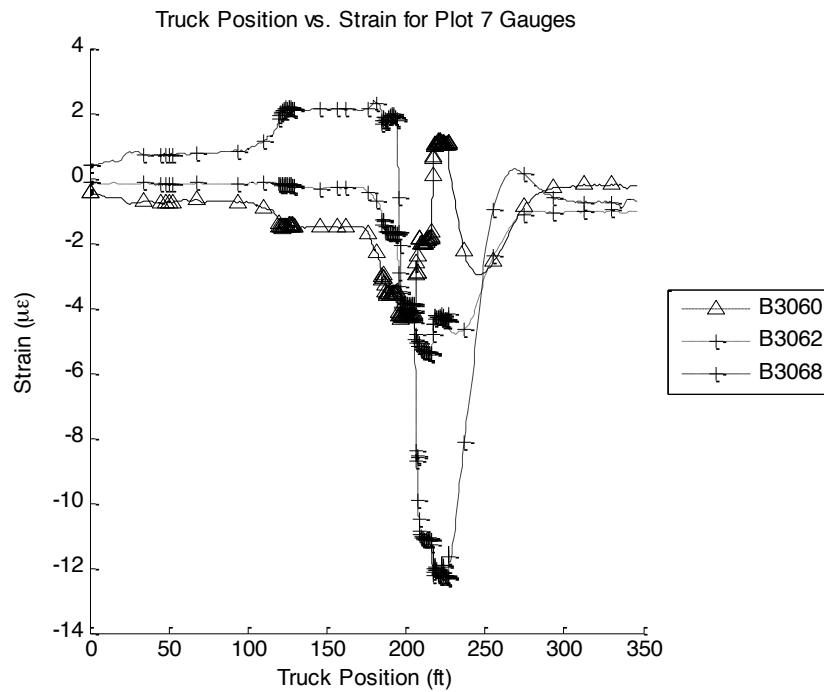


Figure 100: Section 7 Gages

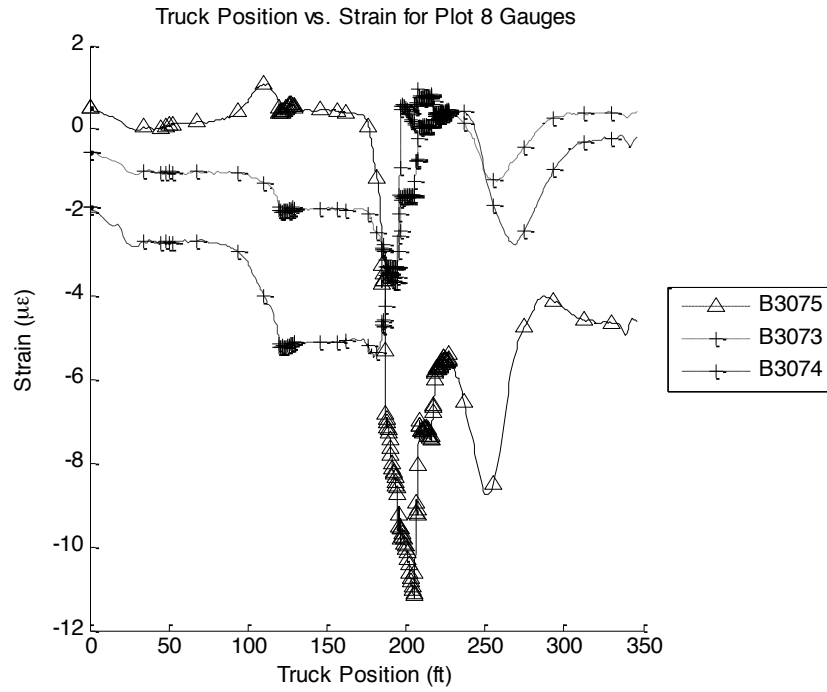


Figure 101: Section 8 Gages

