LIFTING STRAINS IN PRE-CAST CONCRETE GLASS FIBER REINFORCED POLYMER DECK PANELS

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ABSTRACT

The Beaver Creek Bridge on US Highway 6 is a pilot project for Glass Fiber Reinforced Polymer (GFRP) post-tensioned bridge decks in the State of Utah. The Utah Department of Transportation has decided to evaluate GFRP reinforcing bars as an alternative to steel rebar in this bridge deck to increase their lifespan. The precast deck panels were post-tensioned in the longitudinal direction to reduce water penetration into the deck panel joints and lifted from below at four points using straps and cables instead of embedded anchors. The panels were lifted in three stages to transport them from the pre-cast yard to the bridge: (i) lifting out of the formwork, (ii) lifting onto the truck, and (iii) lifting onto the bridge. Two concrete panels were instrumented with twenty-eight electrical gauges. Strain data was compared to theoretical ultimate concrete strains to determine whether tensile cracking had occurred. Deflections of the panel were calculated using the finite element model. During all lifting stages, the panels exhibited acceptable levels of deflections and strains, and there was no cracking observed. Experimental strain measurements showed that they were below cracking limits. The lifting method was successful and is recommended for precast panels constructed with GFRP bars.

Keywords: Precast, Deck Panels, GFRP, Lifting Strains

INTRODUCTION

The Utah Department of Transportation (UDOT) has the goal of increasing the lifespan of bridges as well as to decrease user delays with the use of accelerated bridge construction. In addition, research utilizing corrosion resistant materials which decrease scheduled maintenance, is currently being carried out. Construction of the Beaver Creek Bridge, approximately twenty miles north of Price, Utah on US-6 was completed in 2009. The precast bridge deck was constructed using Glass Fiber Reinforced Polymer (GFRP) bars instead of traditional steel reinforcing bars. The GFRP bars have a lower elastic modulus than steel which leads to larger deflections and in turn cracking. The single span bridge is composed of twelve AASHTO Type IV prestressed girders[1]. The bridge has an overall span length of 88 ft-2 in. and an out-to-out width of 88 ft-10 in.

The deck was designed in accordance with ACI 440.1 R-06[2], and constructed using twenty-four precast bridge deck panels in two phases. Phase I was constructed previous to 2009 with two lanes, one lane each way. Phase II construction began in July, 2009 using pre-cast girders and deck panels. The construction was completed with a cast-in-place, three-foot closure pour; the closure pour couples both sections thereby creating a more stable bridge which links the twelve panels of Phase 1 to the twelve panels of Phase II. The closure pour couples the deck response due to traffic and reduces the dynamic response. Phase II bridge deck panels measure 41ft-5in. long, 6 ft-10 in. wide and 9¼ in. thick, as shown in Figures 1 and 2. The designed 28-day concrete compressive strength was to be 4000 psi; at the time of lifting, the concrete deck panel strength had increased to 6200 psi.

Figure 3 shows the double mat of #5 GFRP bars with a spacing of 4 in. in the longitudinal direction; matching bars were used for the closure pour. Figure 4 shows panel EP3 before the parapet was cast and the GFRP bars extending from the concrete for three feet to construct the closure pour between the bridge deck and the approach slab.

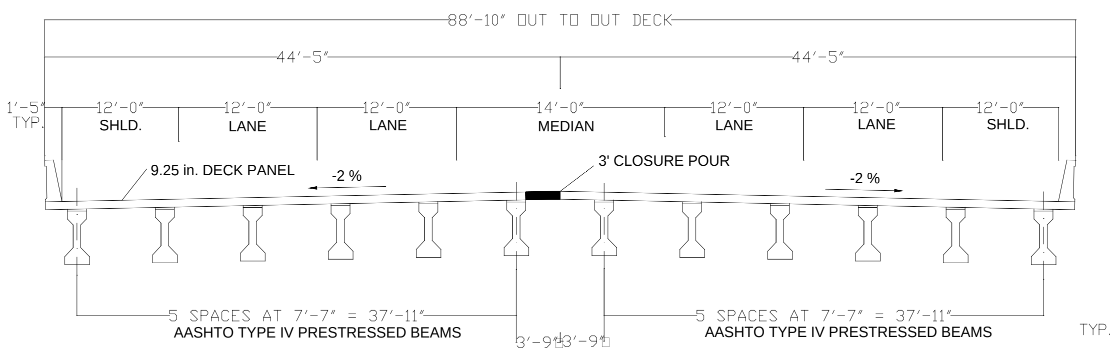

Figure 1: Elevation of Beaver Creek Bridge showing deck and girders.

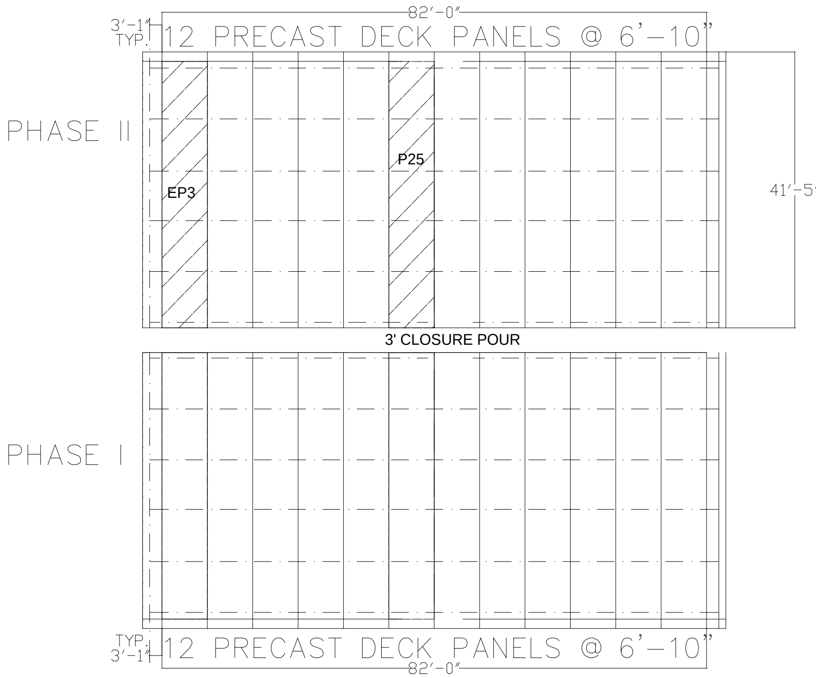

Figure 2: Plan view of Beaver Creek Bridge showing prestressed girders and deck panels.

The panels were cast in Pleasant Grove, Utah, transported sixty-four miles to the bridge site and lifted into place. Each panel was lifted a total of three times: in the first lift the deck panel was removed from the formwork; at this time the panel did not have a parapet. A second lift placed the panel with the parapet on the truck trailer that transported it to the bridge.

Figure 3: GFRP bar layout with bars extended for the closure pour

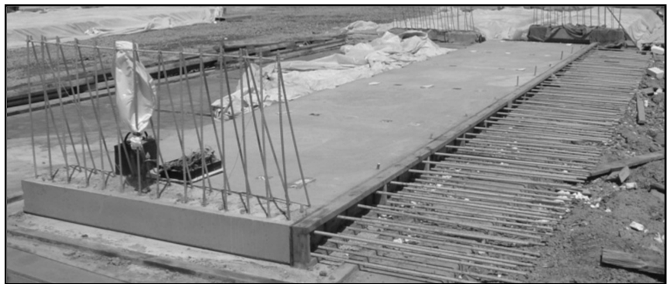

Figure 4. Panel EP3 at the precast yard with approach slab reinforcement on the right.

The final lift placed the panel and parapet on top of the girders at the bridge site. Traditionally, embedded anchors are attached to reinforcing bars. The deck was lifted using straps instead of embedded anchors to decrease the shear forces transferred to the GFRP bars[3]. Additionally, GFRP was used to resist corrosion; steel coils and embeds would allow a potential path for corrosion. Lastly, these precast panels were the longest UDOT had used and wanted to add precautionary measures by lifting from below. From a construction point of view, lifting from below takes the same effort and time as lifting from embeds or steel coils. However, there was a learning curve for maneuverability of the panels. For each lift, the panels were lifted at four points from below with cables attached to steel HSS 4 in.x4 in. tube sections and straps, as shown in Figures 6-8. Lifting was monitored by strain gauges located at 46 in. and 202 in. from the back of the parapet, represented by arrows in Figure 5. The PCI Handbook was referenced for lifting point locations; however, there is no requirement for lifting from below a GFRP reinforced panel[4]. This is a new lifting procedure

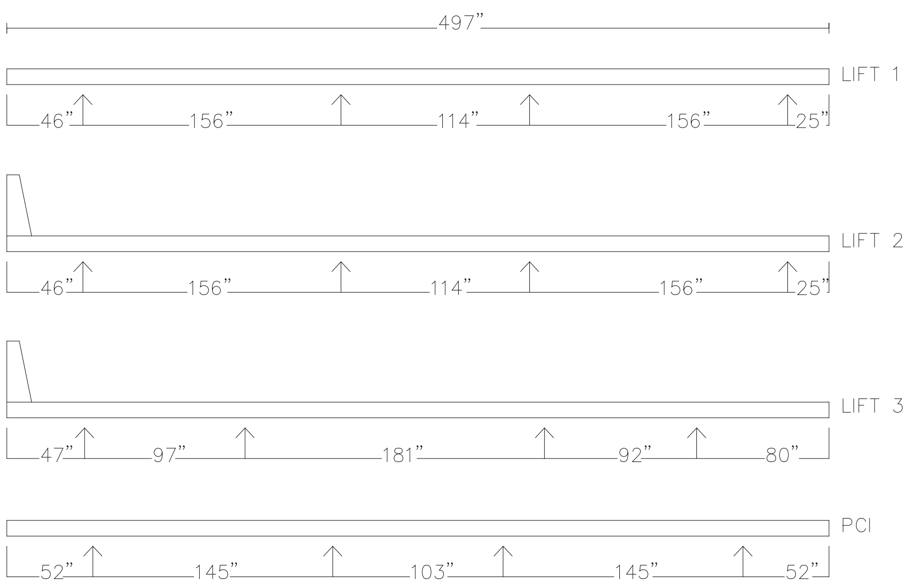

Figure 5. Lifting Point Diagram

Figure 6. Panel lifting using HSS 4x4 steel tubing attached to 1” diameter steel cables.

Figure 7. First lift out of the forms at the casting yard

Figure 8. Third and last lift at the bridge including parapet

for panels reinforced with GFRP bars; hence, curvature of the panels for various lifting arrangements was studied. Lifting strains are important to monitor because the process of lifting may induce large deflections and cracks in the panels. Cracks may lead to reinforcement corrosion[5]. Although GFRP bars are non-corrosive, cracks in the concrete are not desirable since they can reduce aggregate interlock and the shear capacity of the deck panels.

EXPERIMENTAL DATA

Panels EP3 and P25, shown in Figure 2, were instrumented with twenty-eight electrical strain gauges during lifting and transportation. These gauges were attached directly to both the top and bottom GFRP reinforcing mats and recorded the strains in the bars; strain data was collected using dataloggers. Of the twenty-eight gauges, twenty were placed in the transverse direction of the bridge (longitudinal direction of the panel) to record strains in the long dimension of the panel during lifting. The remaining eight gauges were placed in the span direction of the bridge to record post tensioning strains in the short dimension of the panel. For crack monitoring, the twenty gauges placed in the transverse direction of the bridge are used for the analyses presented in this paper. Figure 9 provides a more detailed sketch of the locations of the strain gauges. Electrical strain gauges were chosen due to their high sampling rate potential, relatively low cost, and overall simplicity.

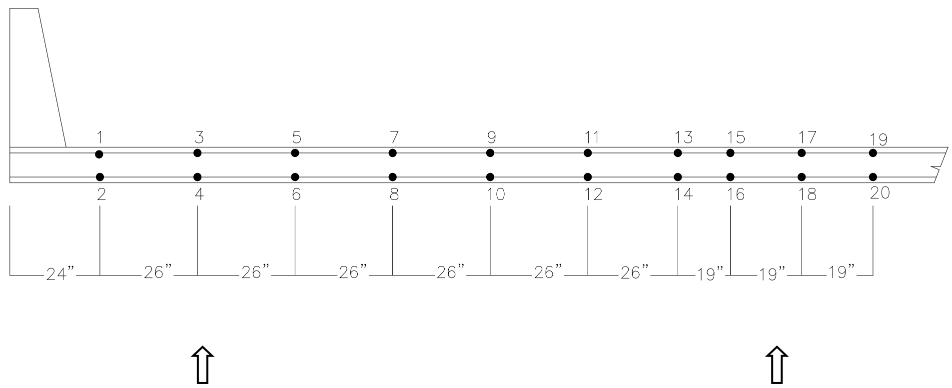

Figure 9. Electrical gauge locations for both panels and lift points located at midline of panel

FIRST LIFT

Panels P25 and EP3 were lifted out of the forms on July 27th and August 6th 2009, respectively. The lifts occurred before the parapets had been cast. The maximum strain profile for half of the length of panel P25 is shown in Figure 10 with tension taken as positive; strain gauge pair distances are measured from the back of the parapet (Figure 2). From the graph, the maximum tensile strain in the bottom mat is 128 microstrain (με) while the maximum tensile strain in the top mat is 57 με. These strains are smaller than the theoretical tensile strain of 138 με that would cause cracking to occur. The tensile cracking limit is obtained by using Equations (1-3), where f_r represents the modulus of rupture[6], and E_c represents the elastic modulus of concrete. The lifting points are located at 46 in. and 202 in. from the back of the parapet, and are represented by arrows. Panels were notched at 47 in., 144 in., 325 in. and 417 in. to allow for strap removal for Lift 3 after the panel was located at its permanent position on the bridge deck.

$$f_r = 7.5\sqrt{f'_c} \tag{1}$$

$$E_c = 57000\sqrt{f'_c} \tag{2}$$

$$\varepsilon_{cr} = \frac{f_r}{E_c} \tag{3}$$

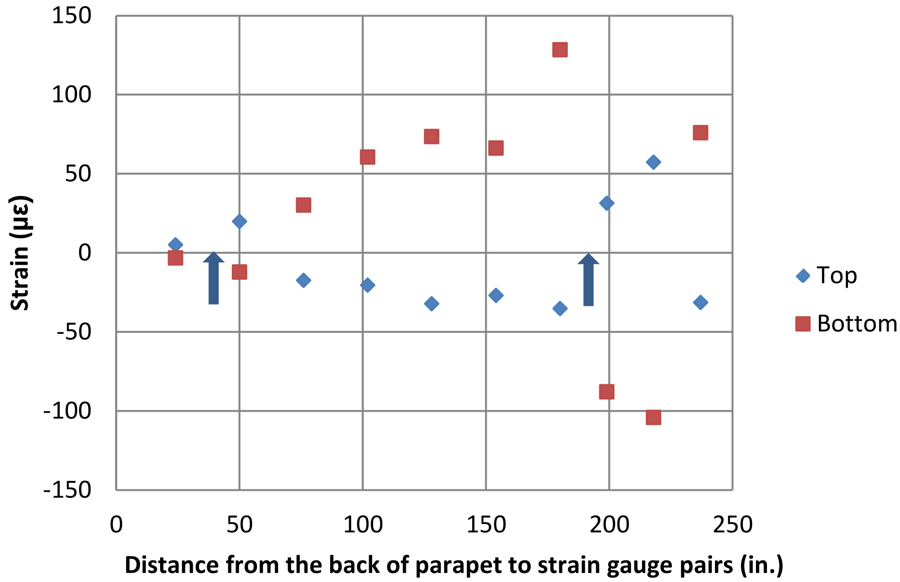

Figure 10. Strain Profile for P25 during its first lift removing it from the formwork

SECOND LIFT

The panels were transported from the precast yard to the bridge site on September 3rd, 2009. The transportation process involved two lifts (referred to as Lifts 2 and 3), and a sixty-four mile journey on a flatbed trailer. Before the second lift, the parapets had been cast. The collected data from this day is limited to the first thirteen strain gauges from each panel (Figure 9). The maximum strain profile during the second lift for half the panel is shown in Figure 11; the maximum tensile strain in the bottom mat was 5με while the maximum tensile strain in the top mat was approximately 15 με. These strains are much less than the tensile cracking strain.

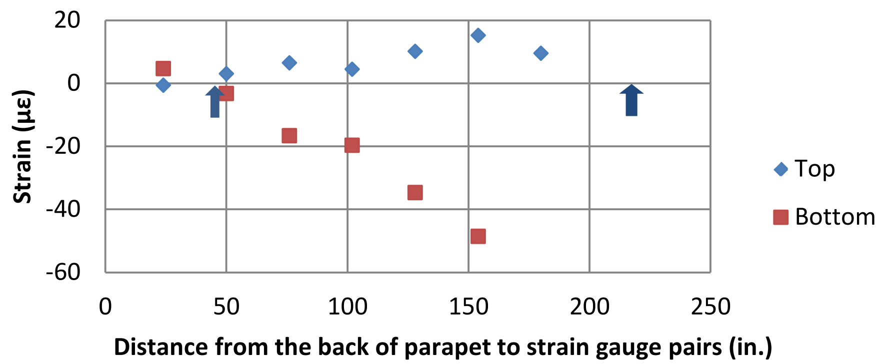

Figure 11. Strain Profile for panel EP3 during its second lift

TRANSPORT

The panels were placed on trucks for a sixty-four mile journey to the bridge site. The maximum and minimum of all strains during transport are shown in Figure 12. During transport of panel EP3 careful observations were made and correlated to the strains on the graph.

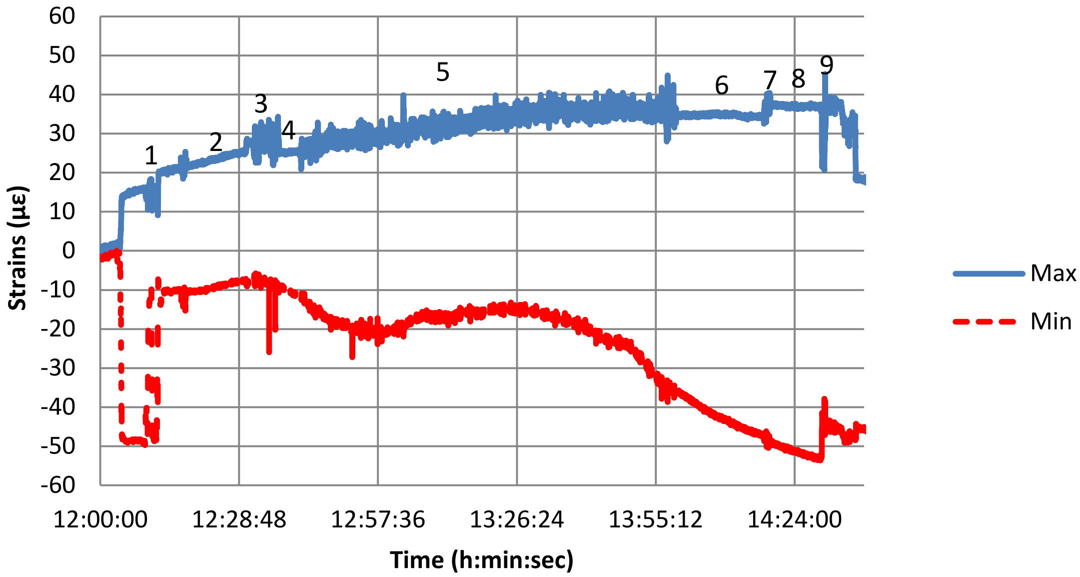

Figure 12. Maximum and minimum of all gauges during transport of panel EP3

The maximum strain for the entire day did not surpass 60 με in either tension or compression. Observations made during the day panel EP3 was moved to the bridge site are shown in Figure 12. The following stages are noted:(1) The second lift, placing the panel on the truck; (2)Truck is stationary while panel is strapped down and straps are tightened resulting in slight increase in strain; (3) Truck begins to move through the precast yard; (4) Truck is stationary while waiting to get on the freeway; (5) Truck travels the sixty-four miles to the bridge site and vibration is evident; (6) Truck arrives at the bridge site and remains stationary until it is unloaded; (7) Truck maneuvers the construction site and moves into position; (8) Tie-downs are removed and (9) Panel is lifted onto the bridge and into place with straps.

THIRD LIFT

The maximum strain profile during the third lift is shown in Figure 13. The third lift occurred directly after the transportation of the panel. The straps were slipped out after the panels were placed on the girders. From Figure 13one can see that the maximum tensile strain in the bottom mat is 37 με while the maximum tensile strain in the top mat is 18 με.

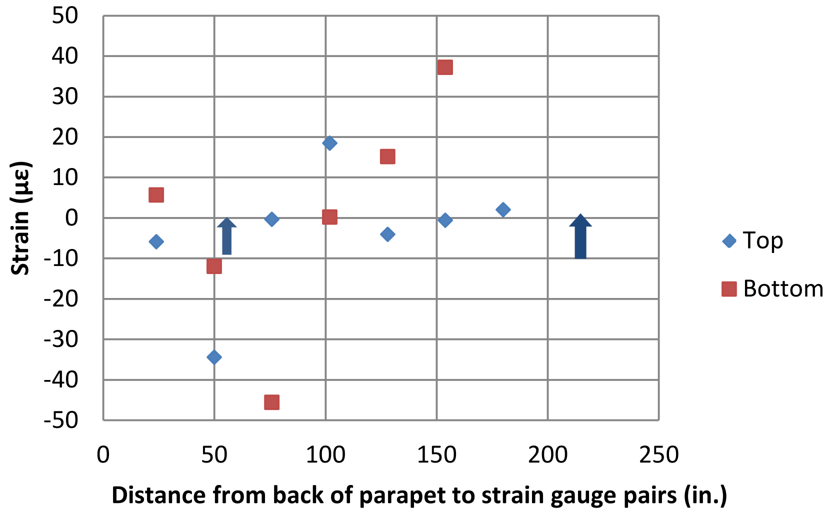

Figure 13. Strain profile for EP3 during its third lift

ANALYTICAL RESULTS

From the recorded strains during lifting, the curvature of the panel was determined using Equation (4), where "ε" is the strain recorded and "$|d - d'|$" is the distance between the top and bottom reinforcement mats. The measured curvature diagram from the first lift of EP3 is shown in Figure 14. Instrumentation was only attached to half the panel; therefore, only two of the four lifting points are included in the curvature diagram. The curvature under the first lifting point at 46 in. is 1.2x10^{-6} /in. The curvature under the second lifting point at 202 in. is 2.5x10^{-5} /in.

$$\varphi_{measured} = \frac{\varepsilon_{top} - \varepsilon_{bottom}}{|d - d'|} \tag{4}$$

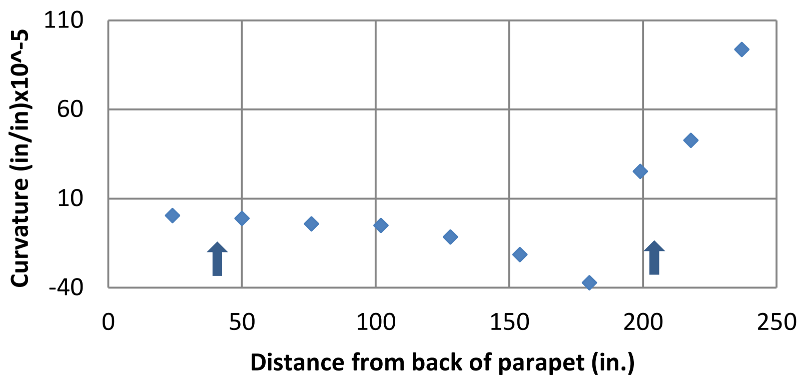

Figure 14. Curvature diagram from experimental results EP3 Lift 1

FINITE ELEMENT MODELING

The three panel lifts, shown in Figure 5 marked Lifts 1-3 were modeled as 5-layer composite area elements of concrete and GFRP reinforcement in the finite element program[7]. The GFRP was modeled as extruded area shell elements and made up 2 of the 5 layers. Lifting devices were modeled as undeformed cables and supporting frame elements. Deflections obtained from the FEM analysis are compared to the AASHTO code limit of span/800 or 0.62 in. The AASHTO code limit was used for lifting because the panels would later be monitored continuously for two-years in service at the bridge; the code limit needed to be consistent throughout the entire research. The calculated deflection of the panel while it was resting on the ground supported by four HSS pipes was a maximum of 1.66 x10^{-3} in. as shown in Figures 15-16. Maximum tensile strains are also reported from the FEM analysis to compare with the tensile cracking limit in the deck panels of 138 με, per ACI 318 and Equation 3. Stresses were modeled at the top and bottom surface of the concrete; stresses were then converted to strains by dividing by the elastic modulus of the concrete. However, the gauges measuring the strain data were located on the GFRP bars, 1.5 inches below the top and bottom surface of the panels. Therefore, a more accurate bottom tensile strain of 12 με at the gauges is shown in Figure 18.The calculated top and bottom maximum tensile stresses of the panel on the ground were 43 psi and 79 psi as shown in Figure 17 with tensile strains also shown in Figure 18 for comparison with experimental data.

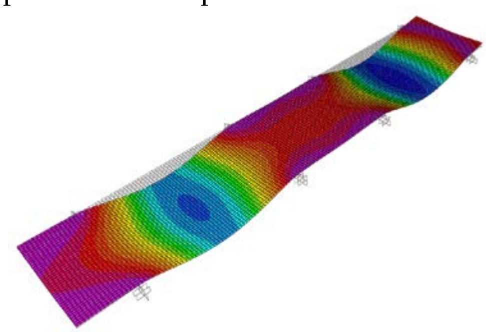

Figure 15. Deflected panel with a maximum deflection of 0.00166 in.

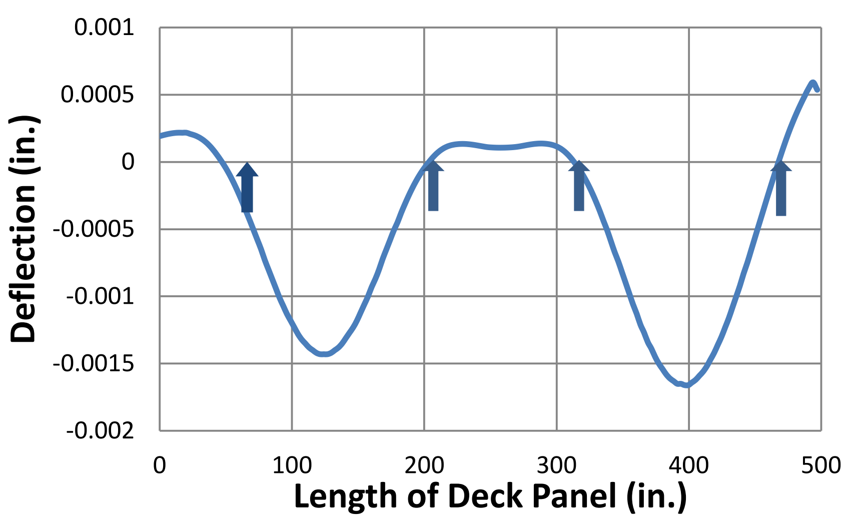

Figure 16. Midline displacement for panel sitting on the ground

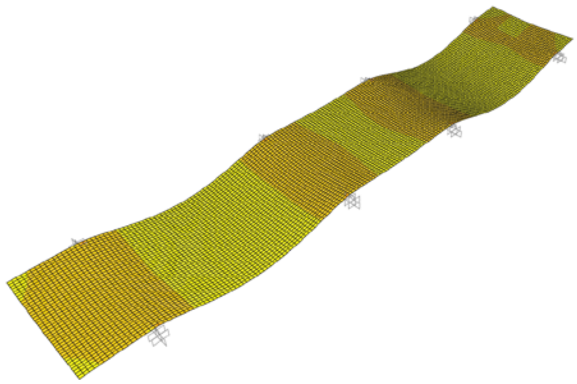

Figure 17. Maximum bottom tensile panel stress of 79 psi in the panel and 217 psi at supports.

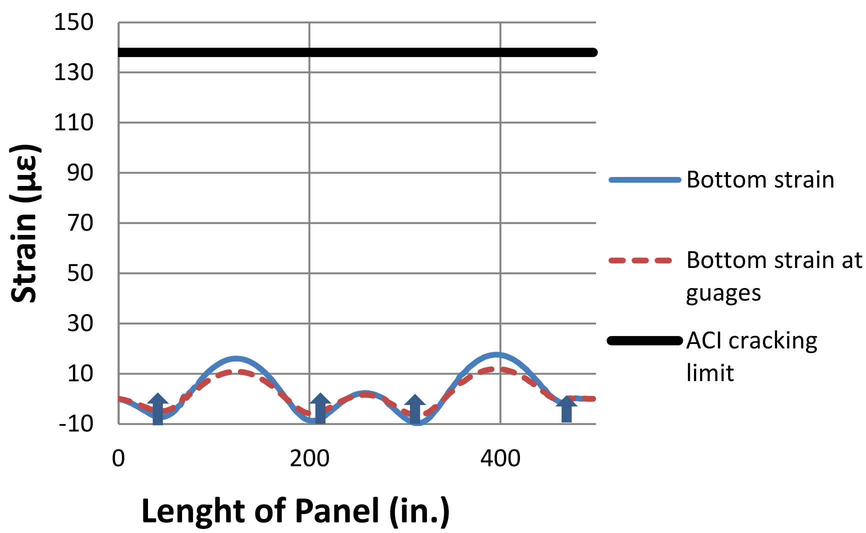

Figure 18. Mid-line bottom tensile strains of panel while on ground

In Lift 1, the panel system was modeled with four, 1- in. diameter A990 grade steel cables lifting the panel from the formwork with two steel beams and without the parapet. The connections from the cables to the frame that was used to lift the panels were modeled as pins, free to rotate. The maximum tensile stress on the top of the slab while in air was 19 psi with a corresponding strain of 4 $\mu\varepsilon$ shown in Figures 19-20. The maximum tensile stress on the bottom of the slab while in air was 129 psi with a corresponding strain of 29 $\mu\varepsilon$ shown in Figures 21-22.The maximum deflection in air was 0.035 inches, shown in Figures 23-24. The curvature was plotted with a maximum of 6.18×10^{-5} in./in. as shown in Figure 25. The model did not correlate well with collected data shown in Figure 14 because it is difficult to simulate the bond between the panel and the bottom of the formwork being broken prior to total panel weight distribution to the lifting cables.

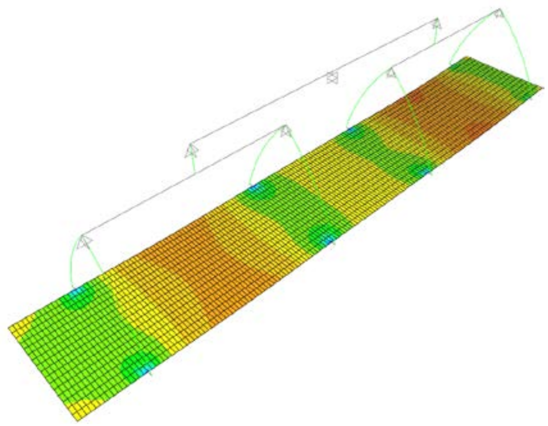

Figure 19. Maximum tensile stress of 19 psi at the top of deck panel while lifting-Lift 1

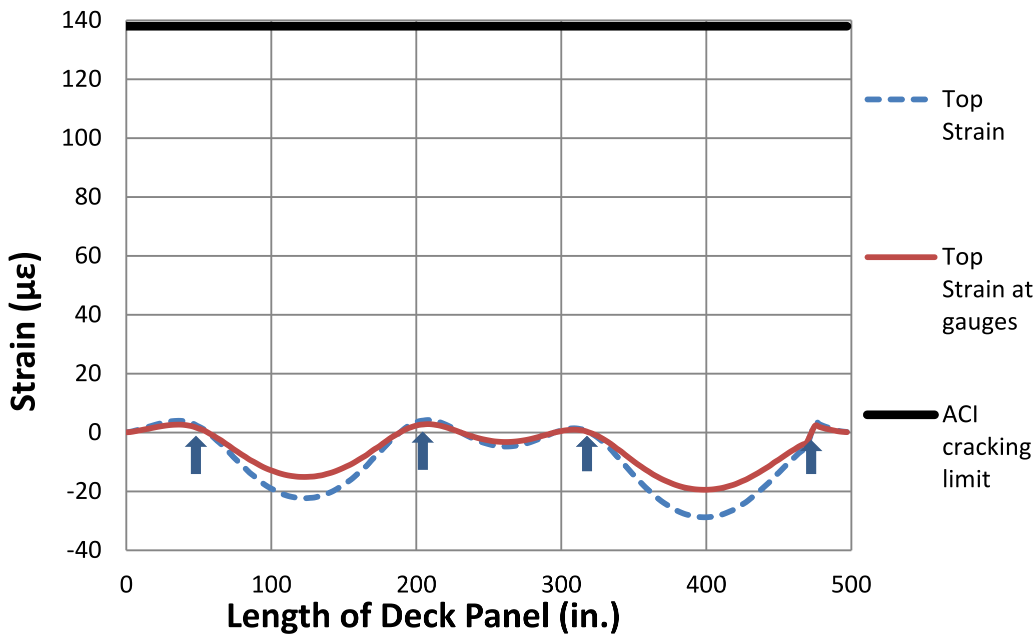

Figure 20. Mid-line top strain without parapet- Lift 1.

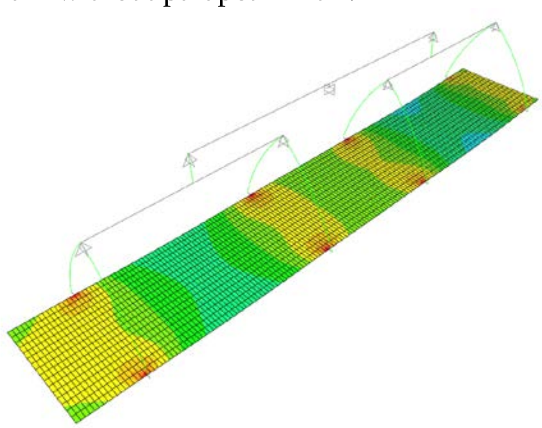

Figure 21. Maximum tensile stress of 129 psi at the bottom of deck panel while lifting- Lift 1

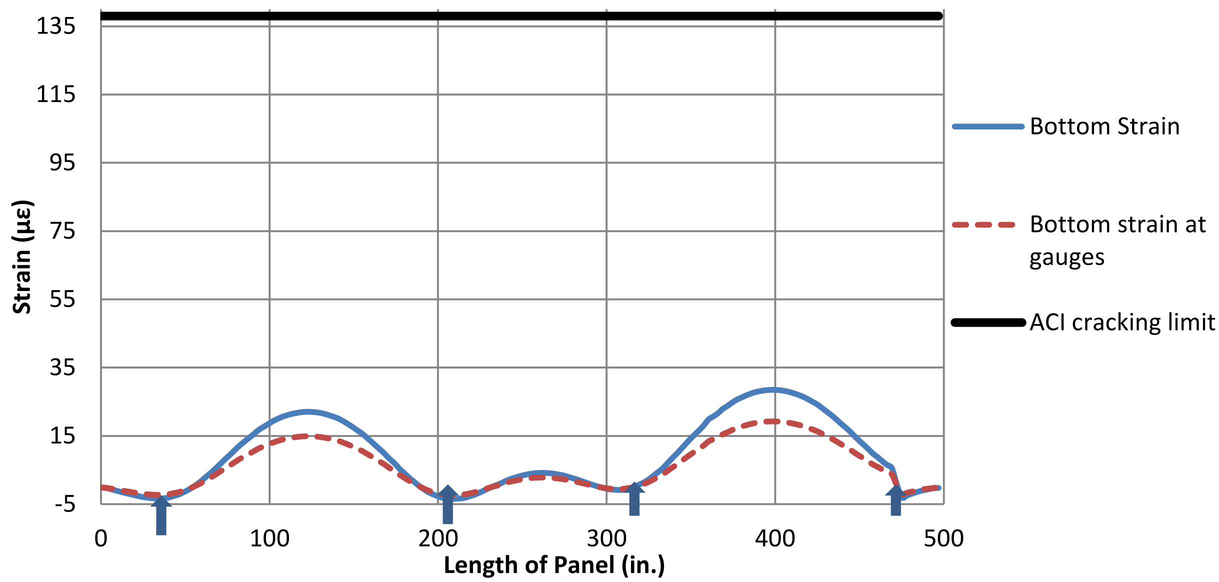

Figure 22. Mid-line bottom strain without parapet- Lift 1

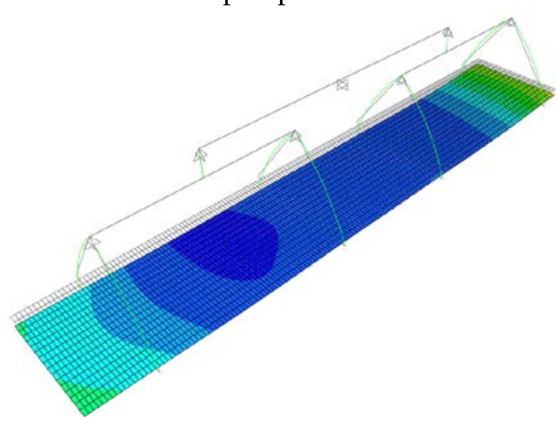

Figure 23. Deflected panel with a maximum midline deflection of 0.035 in.- Lift 1

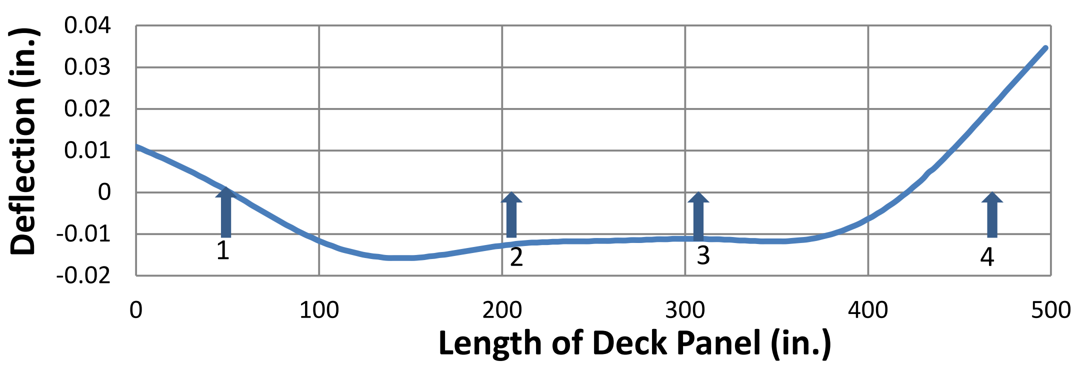

Figure 24. Midline displacement for panel lifting without the parapet- Lift 1

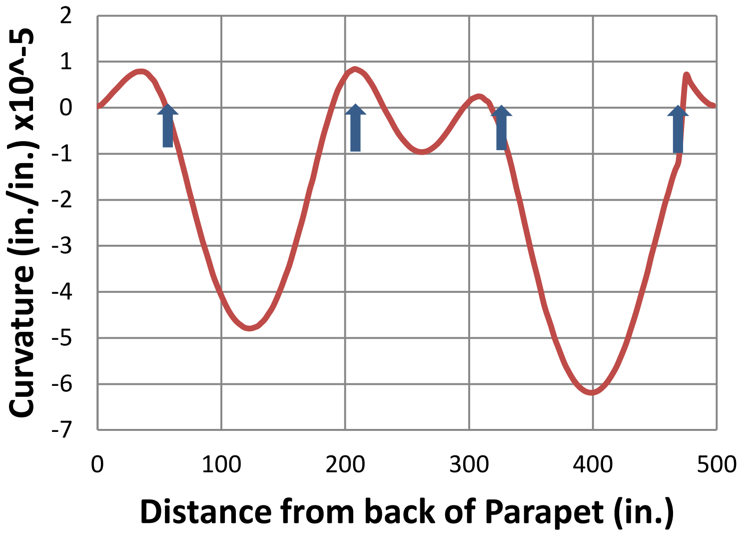

Figure 25. Curvature for panel lifting without the parapet- Lift 1

Lift 2 was modeled with cables lifting the panel with the parapet on the left side of the panel. The parapet was modeled as an equivalent distributed load placed at the end of the panel. The maximum tensile stress on the top of the slab while in air was 125 psi with a corresponding midline strain of 28 με, as shown in Figures 26-27. The maximum tensile stress on the bottom of the slab while in air was 127 psi with a corresponding midline strain of 28 με, as shown in Figures 28-29. The maximum deflection in air was -0.12 inches, as shown in Figures 30-31.

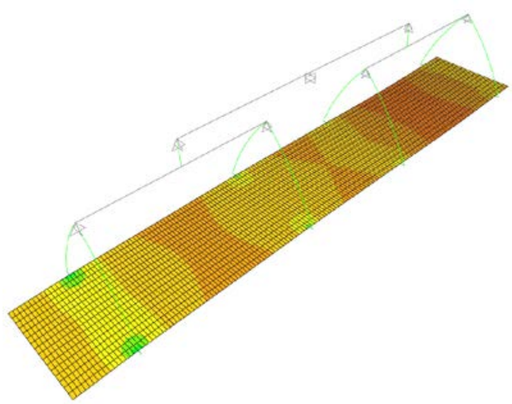

Figure 26. Maximum tensile stress of 128 psi at the top of deck panel while lifting- Lift 2

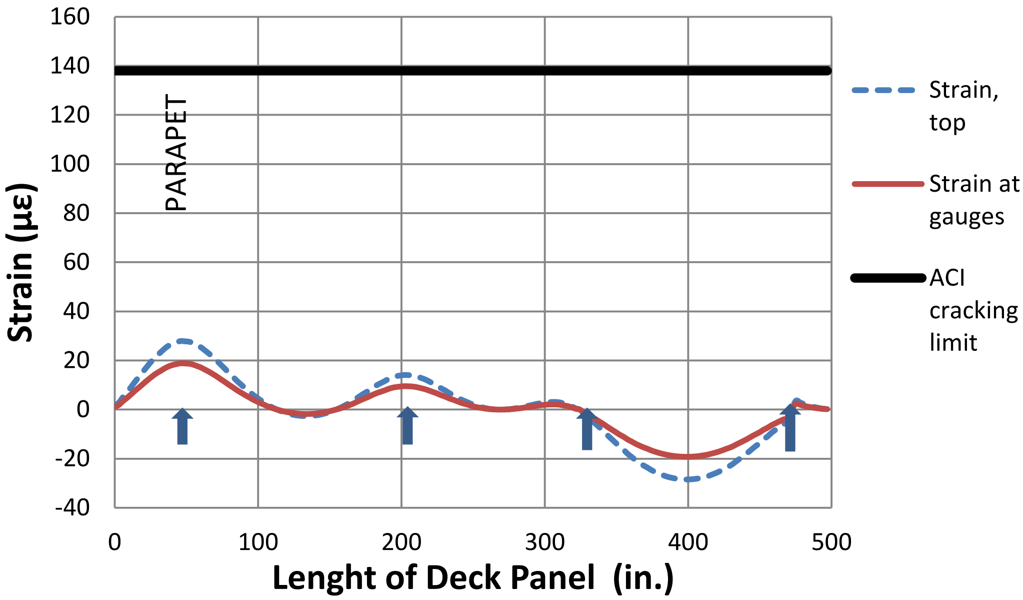

Figure 27. Midline strains of top of deck panel while lifting- Lift 2

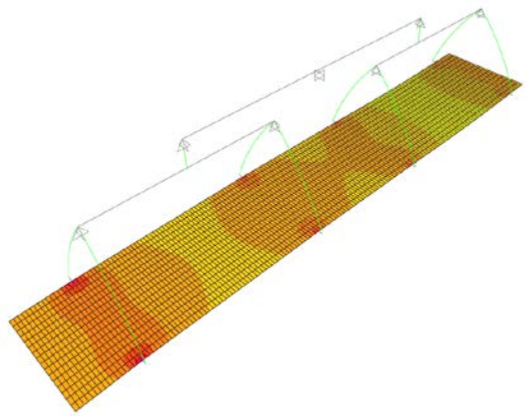

Figure 28. Maximum stress of 127 psi (tension) at the bottom of deck panel while lifting- Lift 2

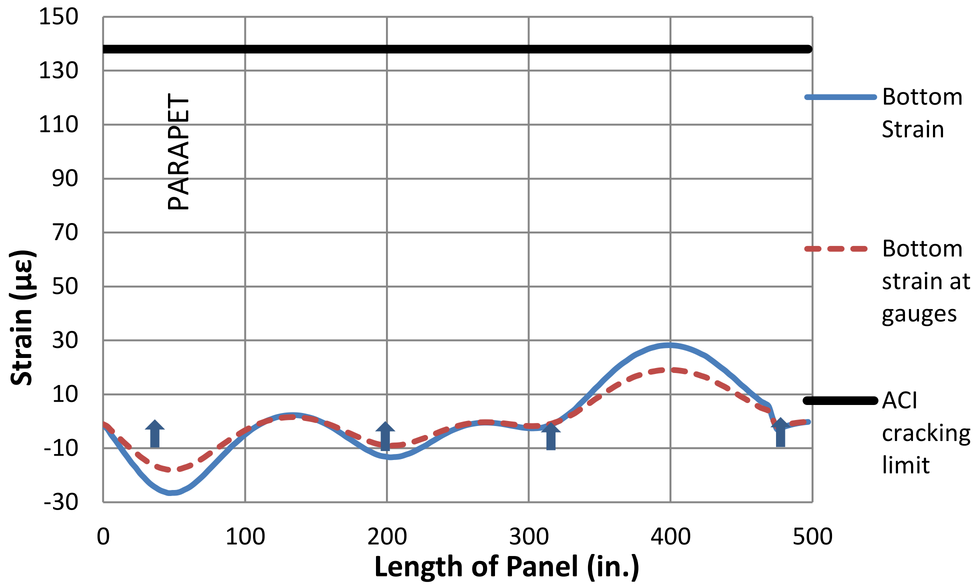

Figure 29. Midline strains of bottom of deck panel while lifting- Lift 2

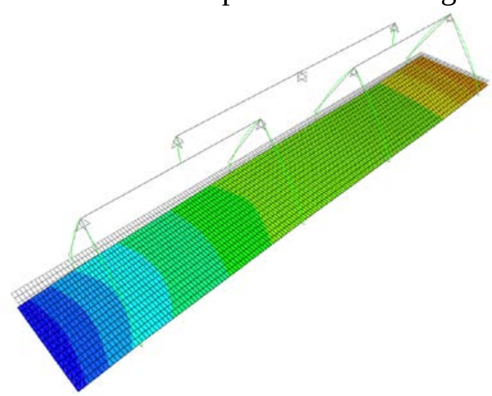

Figure 30. Deflected panel with a maximum midline deflection of -0.12 in.-Lift 2

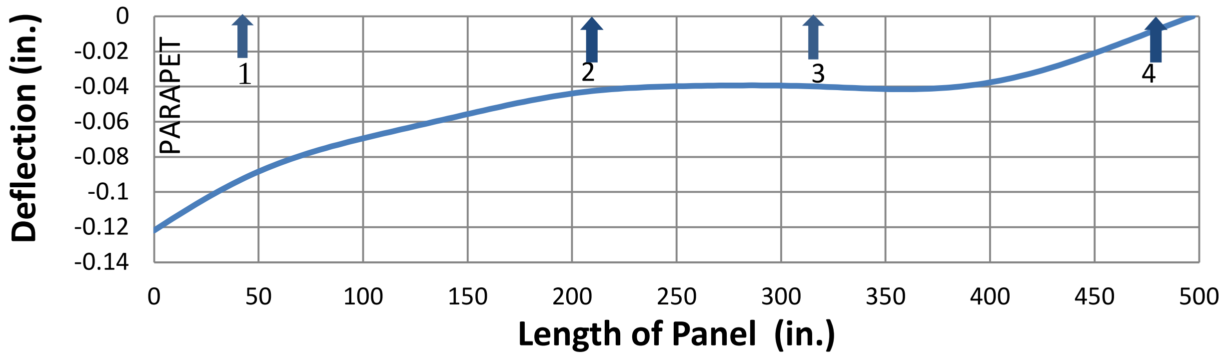

Figure 31. Midline displacement for panel lifting with the parapet-Lift 2

Lift 3 was modeled with straps lifting at the revised pick point locations and the parapet. The parapet was modeled as an equivalent distributed load placed at the end of the panel. The connections from the straps to the truss were modeled as pins, free to rotate. The maximum tensile stress on top of the slab while in air was 146 psi with a corresponding midline strain of 33 με shown in Figures 32-33. The maximum tensile stress on the bottom of the slab while in air was 61 psi with a corresponding midline strain of 14 με shown in Figures 34-35. The maximum deflection in air was -0.11 inches shown in Figures 36-37; the AASHTO design limit for deflection is 0.62 inches.

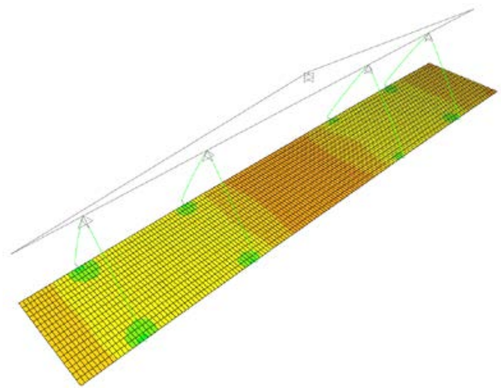

Figure 32. Maximum tensile top stress of panel with parapet while lifting is 146 psi- Lift 3

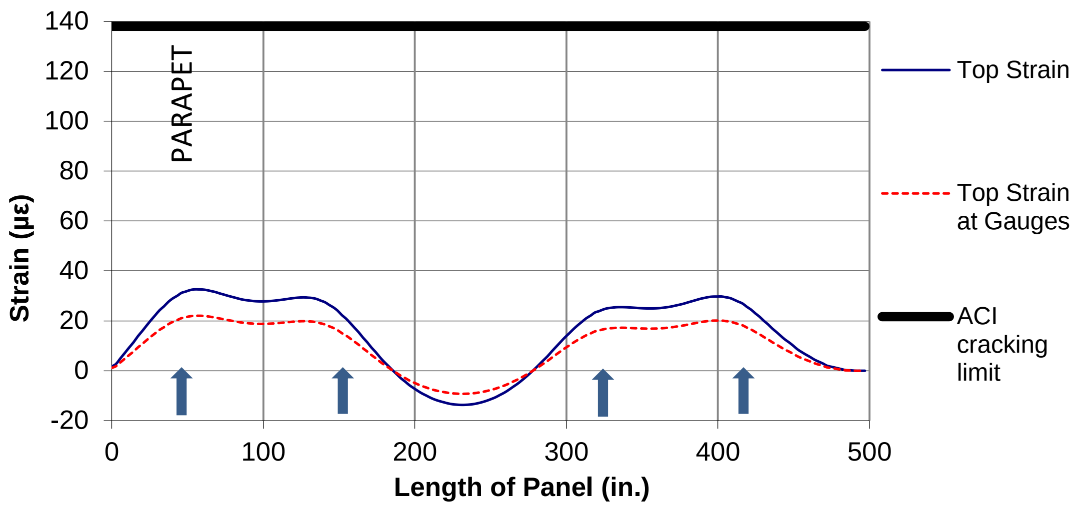

Figure 33. Mid-line top strain with parapet- Lift 3

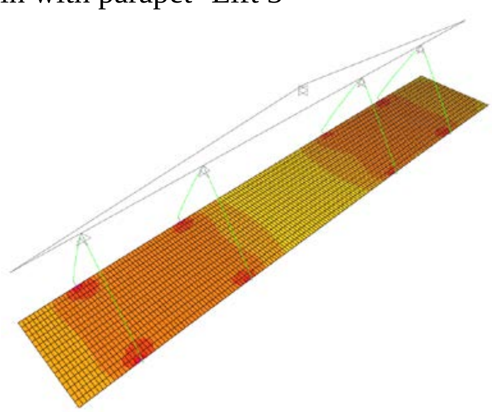

Figure 34. Maximum Bottom Stress panel with parapet is -144 psi (compression)-Lift 3

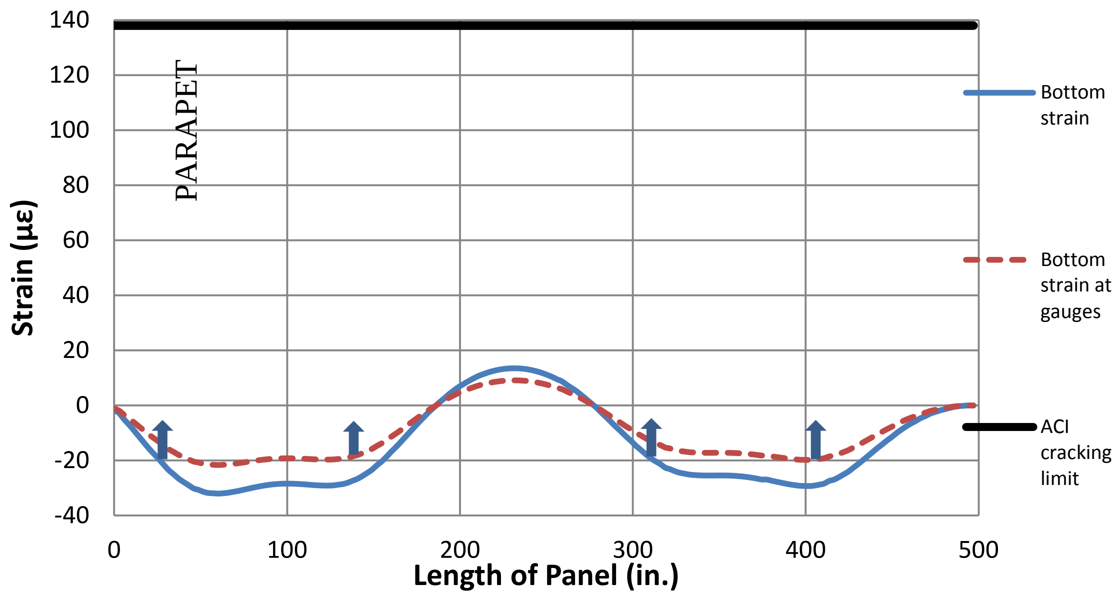

Figure 35. Mid-line bottom strain with parapet- Lift 3

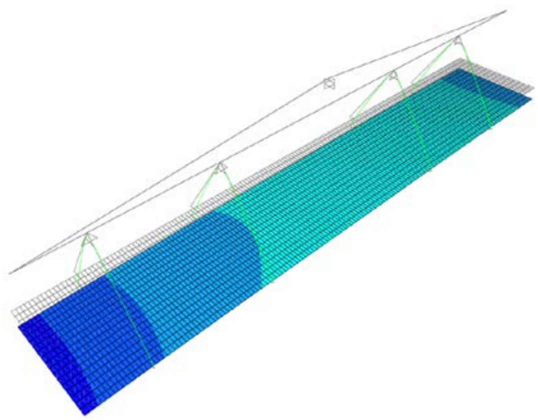

Figure 36. Deflected panel with a maximum midline deflection of -0.11 in.-Lift 3

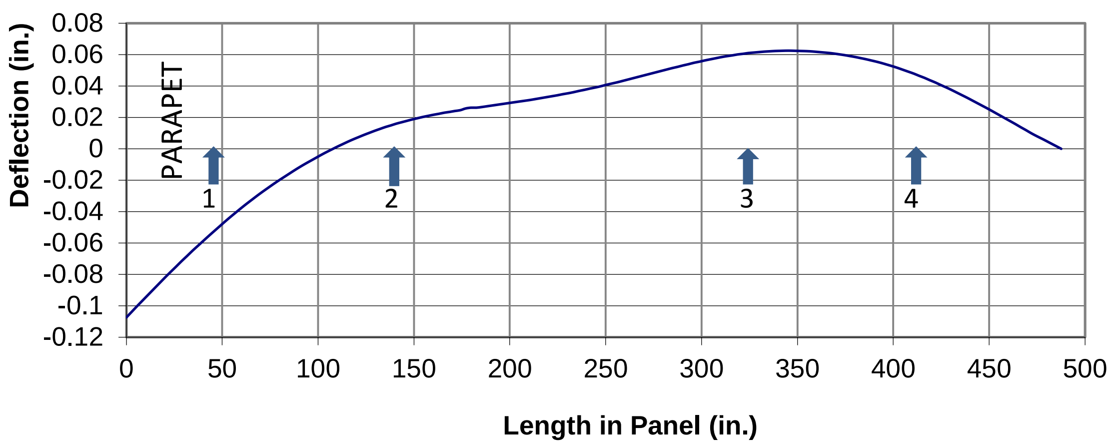

Figure 37. Deflected panel with a maximum midline deflection of -0.11 in.- Lift 3

In the design of the bridge, the deflection of the panel relative to the girders was calculated using the HL-93 truck load and the deflection equation from ACI 440.1R-06. The deflection of the deck relative to the girders due to positive live load moment was calculated as 0.10 in. The AASHTO design limit is 0.625 inches of deflection and a tensile cracking strain of 138 $\mu\varepsilon$.

Lift 3 was modeled for a parametric study of lifting points, investigating if fewer pick points would have met the AASHTO design requirements. Strains and deflections were modeled to compare to code limits. Pick points 1 and 3 were modeled for top and bottom strains; midline deflections were plotted. Similarly, pick points 1 and 4 as well as pick points 2 and 3 were modeled. A table of comparisons between the collected data and the finite element model displaying maximum tensile strain and deflection is shown in Table 1.

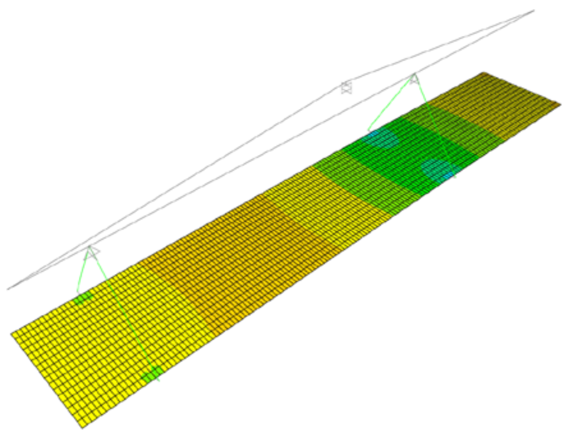

Figure 38. Hypothetical pick points 1 and 3 top strain

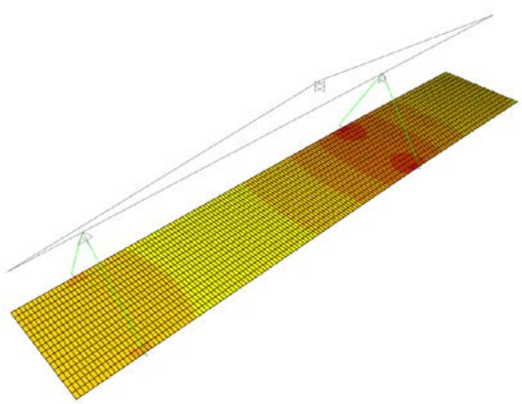

Figure 39. Hypothetical pick points 1 and 3 bottom strain

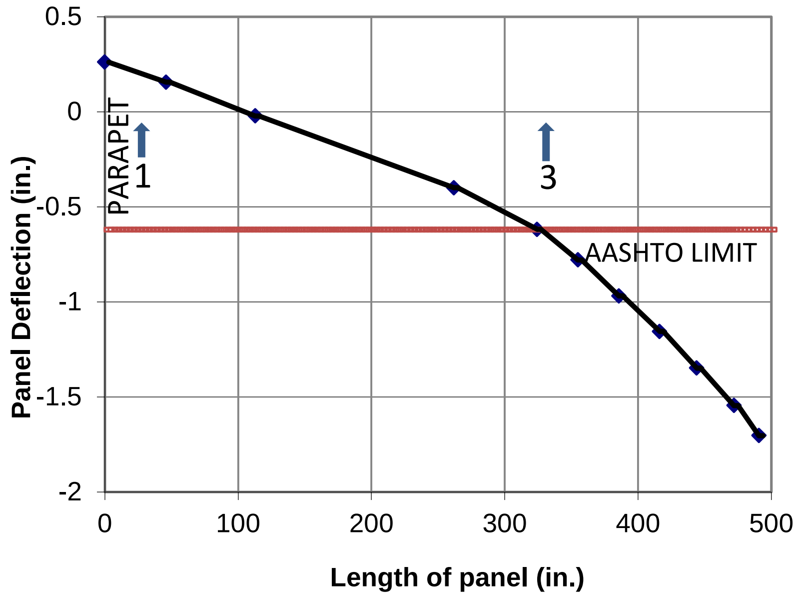

Figure 40. Pick points 1 and 3 deflected panel

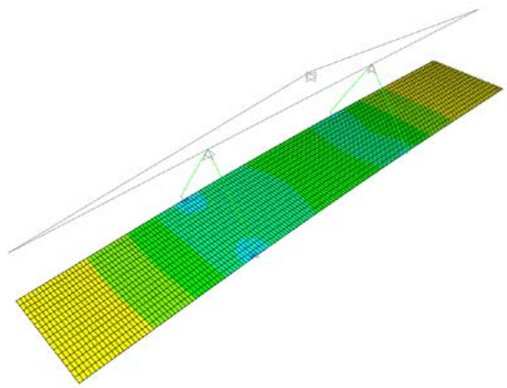

Figure 41. Hypothetical pick points 2 and 3 top strain

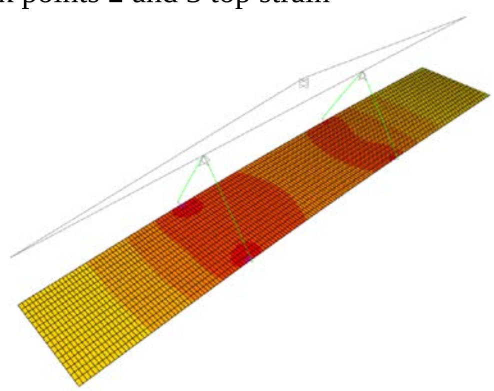

Figure 42. Hypothetical pick points 2 and 3 bottom strain

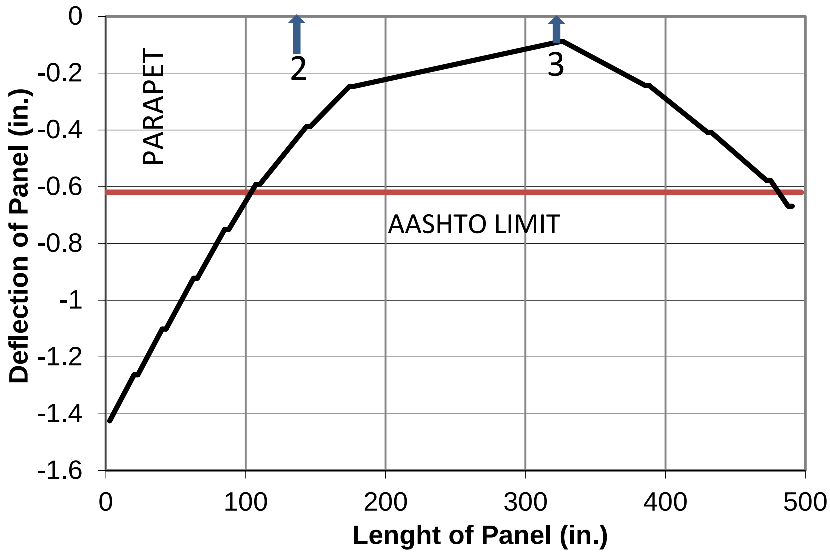

Figure 43. Pick points 2 and 3 deflected panel

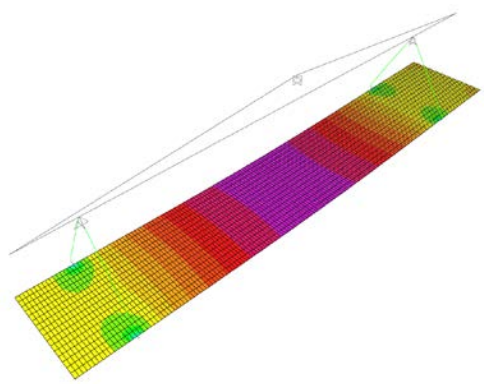
Figure 44. Hypothetical pick points 1 and 4 top strain

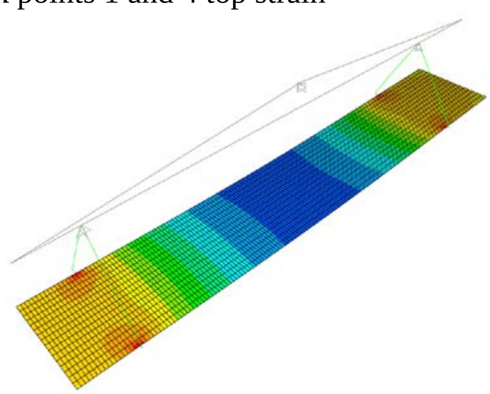
Figure 45. Hypothetical pick points 1 and 4 Bottom strain

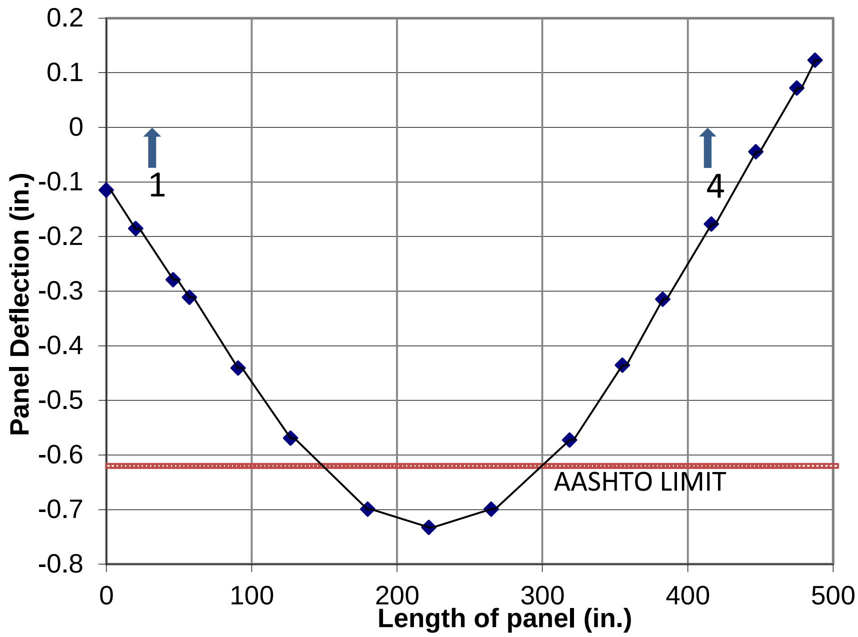

Figure 46. Pick points 1 and 4 deflected panel

Table 1. Comparison between collected data and model

	Collected Data		Finite Element Model		
	Tensile Strain at gauge, με	Curvature EP3	Maximum deflection, in.	Tensile Strain at gauge, με	Curvature
Lift 1	50	2.5×10^{-5}	0.035	19	6.18×10^{-5}
Lift 2	15	9.8×10^{-6}	-0.12	19	8.7×10^{-5}
Lift 3	18	8.6×10^{-6}	-0.11	22	9.9×10^{-5}
Points 1,3	N/A	-	-1.7	154	-
Points 1,4	N/A	-	-0.73	173	-
Points 2,3	N/A	-	-1.4	201	-

CONCLUSIONS

Because GFRP has a low elastic modulus, large deflections which lead to cracking are a concern. GFRP panels were constructed and placed without damaging them in a way that would compromise their integrity and functionality in-service at the bridge. Instead of lifting by embeds attached to the bars, lifting the deck panels from below decreased stress concentrations in the panel and shear stresses in the bars and reduced deflections; this is a viable lifting method for such precast deck panels. Monitoring the panels during lifting and transportation gave a strain history to check against the tensile cracking limit. The strains for all three lifts were in the same range and never exceeded 128 με; the tensile limit of the concrete used for the precast deck panels is 138 με. There were no signs of cracking.

The finite element model predicted a maximum deflection of 0.12 in. (Lift 2, with parapet) which is within the AASHTO design limit for service loads of span/800 or 0.62 in. Furthermore, the collected data compared well with the model. Curvature results from the finite element model did not correlate well with the collected data due to the bond stresses from the panel being lifted out of formwork that were not modeled in the finite element analysis. All two-point hypothetical lifting scenarios violate AASHTO deflection limit and display tensile cracking. The four point lifting method displayed good results and is recommended for precast panels constructed with GFRP bars as well as traditionally reinforced deck panels. The deck panels were monitored at the bridge for three years; recorded strains never exceeded the lifting strains[8].

ACKNOWLEDMENTS

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