PRECAST GFRP-REINFORCED BRIDGE DECK PANELS

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ABSTRACT

The deck of the Emma Park Road Bridge on US 6 near Price, Utah, was constructed in 2009 using precast concrete panels measuring 41ft -5in. x 6ft-10in. x 9¼ in. (12.624m x 2.083mx235mm), reinforced with glass fiber reinforced polymer (GFRP) bars. The paper presents the construction, instrumentation and monitoring of two precast concrete deck panels during lifting, transportation, post-tensioning, and subsequent static and dynamic truck load tests. The paper also presents construction details of the bridge utilizing the precast concrete panels. Strain measurements were carried out during lifting and placement of the precast concrete GFRP panels. Results from the static and dynamic truck load tests include relative deflections between the bridge deck and the diaphragms, and deflections and vertical accelerations of the AASHTO Type IV prestressed girders. The data collected from the lifting and truck load tests carried out in this research proves that this is a viable construction method, in accordance with accelerated bridge construction. The performance of the supporting prestressed girders during the truck load tests is also examined. The information gathered during the research is compared to the AASHTO Bridge Design Specifications.

Keywords: Bridge; Concrete; Fiber reinforced polymers; Health monitoring; Non-destructive test; Truck load test.

INTRODUCTION

The Utah Department of Transportation (UDOT) has been researching methods for extending bridge deck life in Utah to better match the service life of the entire bridge. Currently Utah bridges are designed to a 75 year design life, but the decks are requiring replacement after 35 to 45 years. Deck replacement projects increase the life cycle cost of the structure as well as adding to user delays. UDOT decided to evaluate Glass Fiber Reinforced Polymer (GFRP) reinforcing bars as an alternative to steel rebar in bridge decks.

The monitoring and analysis of glass fiber-reinforced composite bridge decks made with using acoustic emissions has been carried out by Gostautas et al.[1] One of the conclusions of the study was that the results were based on a single manufacturing process using hand layup and that further study was warranted for different types of FRP decks. Several non-destructive methods exist for assessing common defects in concrete bridge decks including infrared thermography, impact echo, and ground penetrating radar[2]. Jáuregi et al.[3] (2010) were able to demonstrate by truck load testing that measured tensile strains exceeded the concrete cracking strain and that the slab was not a gross section.

Design concepts, construction details, and results of live load field tests for bridge decks cast-in-place and reinforced with GFRP bars have been presented previously by Benmokrane et al.[4,5] However, there is no significant amount of research regarding bridge decks constructed using precast concrete panels reinforced with GFRP bars.

The research described in this paper had two major phases, pre-construction, and an in-situ truck load test. Two GFRP reinforced precast concrete panels were monitored during construction, lifting, placement, post-tensioning, and truck load testing, using electrical strain gauges and vibrating wire strain gauges. The deflections of the bridge deck relative to the two diaphragms connecting the prestressed concrete girders were monitored using linear variable differential transformers. Finally, the absolute deflection of the girders at midspan during a static truck load test, and the dynamic performance of the girders during a dynamic truck load test were monitored using surveys and accelerometers, respectively.

BEAVER CREEK BRIDGE

GFRP reinforcing bars were used for the deck of the Beaver Creek Bridge on US-6 in rural Utah. The bridge is a single span creek crossing with access for wildlife passage. The overall span length is 88’-2” (26.87m), with an out-to-out width of 88’-10” (27.08m), as shown in Figs. 1 and 2. The girders are AASHTO Type IV prestressed beams[6]. The deck was designed in accordance with the ACI 440.1R-06 guidelines[7]. The deck was constructed using precast panels mildly post-tensioned in the longitudinal direction, as shown in Fig. 1. The bridge was constructed in two phases as shown in Fig. 2; this required a closure pour between the Eastbound and Westbound lanes.

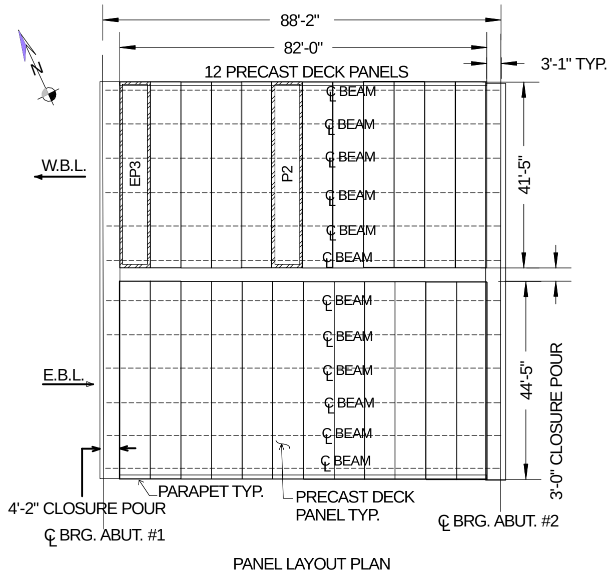


Fig. 1. Layout of Beaver Creek Bridge on US-6

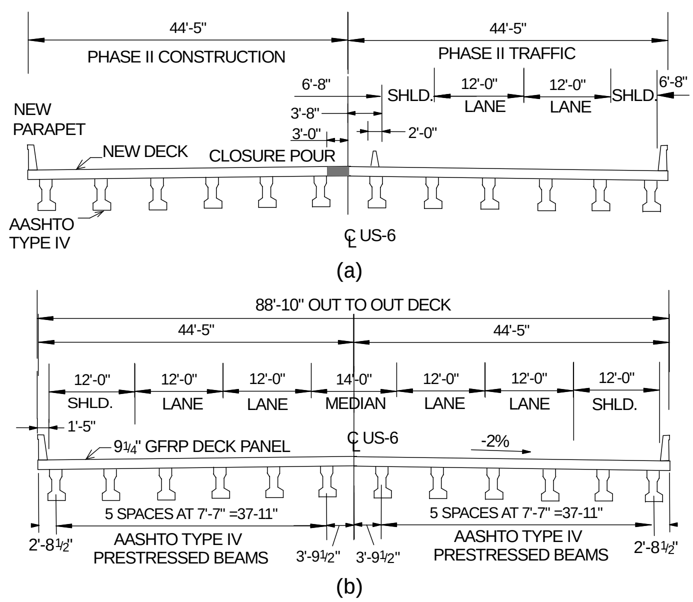


Fig. 2. Beaver Creek Bridge: Construction sequence and layout

GFRP REINFORCED PRECAST CONCRETE PANELS

The design of the deck panels was controlled by crack width and deflection. The relatively low modulus of elasticity of GFRP bars leads to wider crack widths than with traditional steel reinforcement. Acceptable crack width tolerances can be relaxed some with GFRP due to its non-corrosive nature, but this does not completely counteract the wider crack widths, which can lead to loss of aggregate interlock and shear capacity due to loss of shear friction. GFRP reinforced panels also exhibit higher deflections than steel reinforced panels. Due to these design limitations, several adjustments were made to the structural design. The first adjustment was to the bar spacing. In the transverse direction the spacing was reduced from 8in. (203mm) down to 4in. (102mm), as shown in Fig. 3. It was not practical to reduce the spacing any further, so alternative methods for decreasing crack width and deflection had to be used. A balance between thickening the deck and decreasing girder spacing was used. The deck was increased from the standard 8½ in. (216mm) thickness, up to 9¼ in. (235mm). The girder spacing was decreased from 9ft-4in. (2.84m), down to 7ft-7in. (2.31m), increasing the number of girders needed by two.

To reduce construction time and user impact, the bridge was constructed using precast concrete deck panels with mild longitudinal post-tensioning. Post-tensioning consisted of 11 tendons as shown in Fig. 3; each tendon was made of three 0.6 in. (15mm) Grade 270 (1862MPa) low relaxation steel strands that were grouted. Typical deck panels are moved and placed using embedded anchors. This was not practical with the GFRP reinforced panels because of the low shear strength of the bars. The panels had to be lifted with straps wrapping around and under the panels, making their placement more difficult.

The low shear strength also affected the post-tensioning anchors. GFRP bars could not provide adequate shear strength for the anchorage. Some steel bars were placed on the end panels for anchorage of the post-tensioning. Bars extend from the panels into closure pours at the abutments and along the centerline of the bridge to tie the approach slabs in, as well as to connect the two phases of bridge construction, as shown in Figs. 1 and 2. Traditionally, the steel bars have been bent to avoid conflicts during placement. GFRP bars cannot be bent, making placement more difficult. A few bars had to be cut during placement because of this fact. New bars were drilled and epoxied in at the locations where bars were cut.

INSTRUMENTATION OF PRECAST PANELS AND PRESTRESSED GIRDERS

In the summer of 2009, construction began on the Beaver Creek Bridge, located approximately 20 miles (32 Km) north of Price, Utah on US-6. GFRP bars offer many advantages over traditional steel bars, including increased tensile strength, reduced unit weight and corrosion resistance. The pre-construction phase focused on instrumentation and monitoring of two precast concrete deck panels; end panel EP3 and center panel P2 were chosen for instrumentation due to their location in the westbound lanes, as shown in Fig. 1. Monitoring included the initial lift from the formwork, the lift from the precast yard to the truck, transit of the panel to the bridge, a second lift placing the panel on the bridge, and post tensioning. Each panel was instrumented with 28 electrical strain gauges, to be used during the lifting and transportation of the panel. These gauges were attached to both the top and bottom GFRP mats; of the 28 electrical strain gauges, 20 were placed in the transverse direction of the bridge (along length of the panel) to record strains during lifting. The remaining 8 gauges were placed in the longitudinal direction to record strains in the short dimension of the panel. Panels EP3 and P2 were each instrumented with four vibrating wire strain gauges (VWSG) placed in the longitudinal direction of the bridge. These gauges were used to record strains induced by post tensioning as well as the change in strain due to creep and shrinkage and for long-term monitoring. In addition to the 4 longitudinal VWSGs, panel P2 was equipped with an additional 16 VWSGs placed in the transverse direction of the bridge. These gauges were primarily used during the truck load test and for long-term monitoring. Fig. 4 shows electrical and vibrating wire strain gauges for panel P2.

The relative deflection from the bottom of the bridge deck to the top of the steel diaphragms joining the prestressed girders was measured using Linear Variable Differential Transformers (LVDTs). The bridge was instrumented with six LVDTs. LVDTs 1-5 were placed above the west diaphragms between girders 1 and 6; LVDT 6 was placed between girder 2 and 3 above the east diaphragm. Fig. 5 shows the typical instrument arrangement.

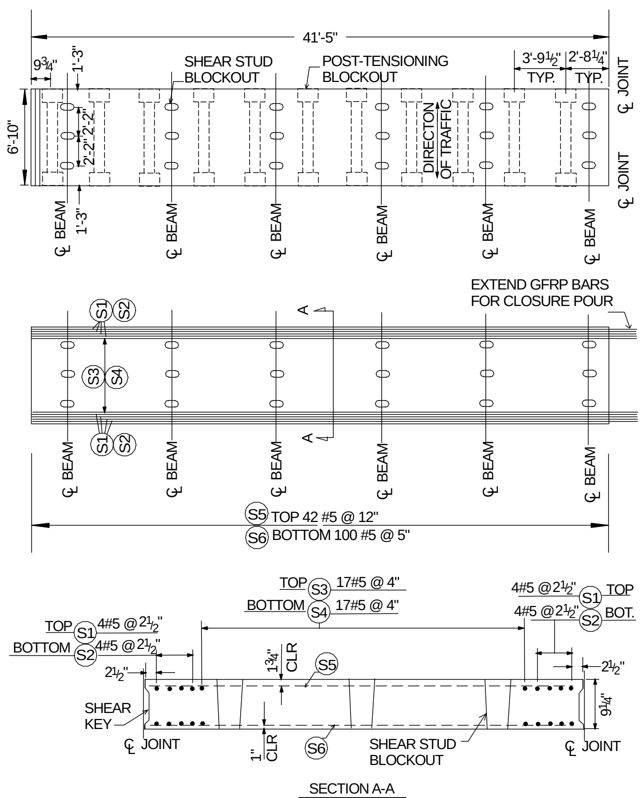


Fig. 3. Plan and GFRP reinforcement of precast concrete panels for W.B.L. of Beaver Creek Bridge on US-6

Fig. 4. Electrical and vibrating wire strain gauge installation in panel P2

Fig. 5. LVDT installation on west diaphragm

Six single-axis accelerometers were attached at midspan of each girder under the Westbound lanes to measure vertical acceleration of the girders. All instrumentation data was collected by an electronic data acquisition system at a sampling rate appropriate for each test. Monitoring of lifting strains in the precast panels was achieved wirelessly using a modem. During the truck load test, data was also recorded using the modem.

MATERIAL PROPERTIES

The properties of the GFRP bars were as follows: tensile strength = 95,000 psi (655 MPa), modulus of elasticity = 5,920,000 psi (40.8 GPa). The reinforcement consisted of #5 (Φ 16mm) GFRP bars. The concrete used for the precast concrete panels had a 28-day compressive strength of 6,200 psi (43 MPa).

LIFTING STRAINS

Two different lifting arrangements were used to lift the precast panels: (a) at the casting yard, and (b) at the bridge site; the arrangement used at the casting yard is shown in Fig. 6; the arrangement used at the bridge site employed a steel truss with four straps placed in slightly different positions than the ones shown in Fig. 6. The maximum strain profile for half the panel P2 during its lift out of the casting yard is shown in Fig. 7(a). The tensile strains in the GFRP bars was less than 150 microstrains (με); the corresponding strain in the concrete is estimated to be slightly higher due to the concrete cover. This indicates that the cracking strain was exceeded at some locations, since the theoretical cracking strain obtained by dividing the modulus of rupture by the modulus of elasticity is 132 με. The curvature diagram is obtained as shown in Fig. 7(b), which compares favorably with a theoretical evaluation.

STATIC TRUCK LOAD TESTS

The truck load tests were performed on September 29th 2009, and consisted of nine static tests and five dynamics tests. The static tests are shown in Table 1. The static tests have been broken down into three groups depending on the lanes being loaded. Tests 1-3 were preformed on the slow lane and were conducted using truck “A”. Tests 4-6 were preformed on the left lane using truck “B”. Tests 7-9 used both trucks “A” and “B” in their respective lanes, as shown in Fig. 8. The geometric properties of the trucks are shown in Fig. 9 and the axle weights for both trucks are given in Table 2.

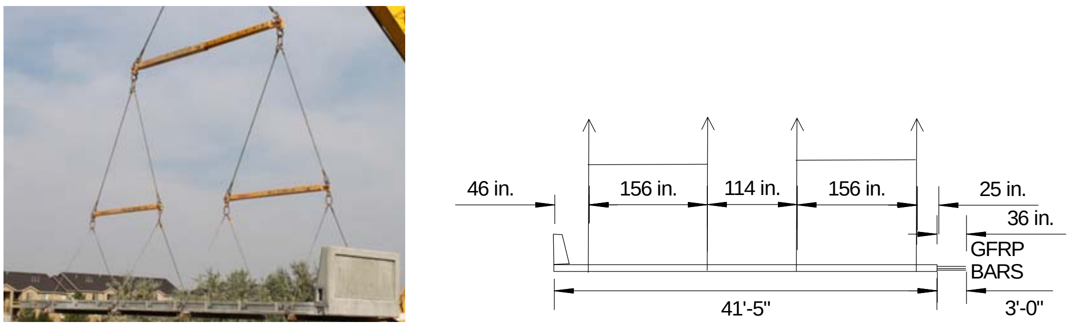
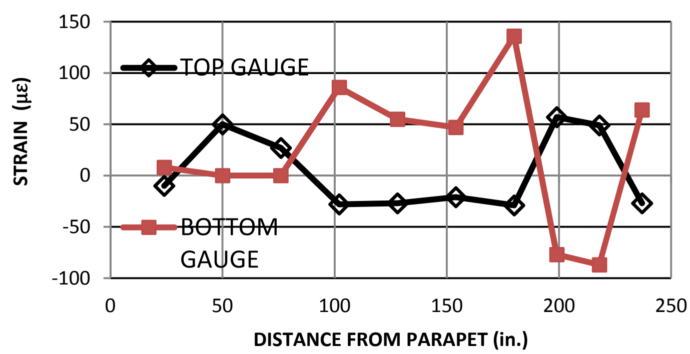
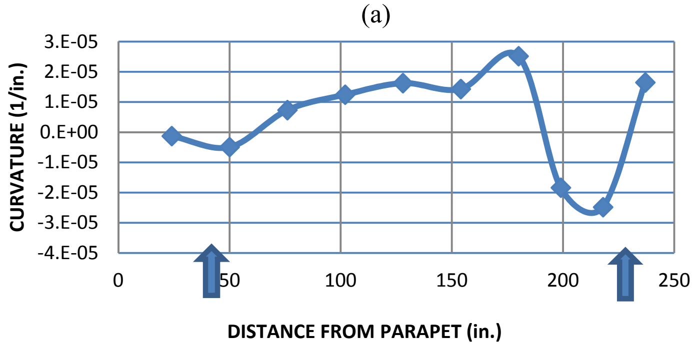


Fig. 6. Lifting points of GFRP reinforced precast concrete panels at casting yard



(a)



(b)

Fig. 7. Lifting at the casting yard for half the length of panel P2: (a) GFRP bar maximum strains (positive = tension); (b) Curvature diagram (arrows represent the lifting points)

Table 1. Static truck load tests

Test	1	2	3	4	5	6	7	8	9
Truck	A	A	A	B	B	B	A,B	A,B	A,B
Point*	1,1	2,2	3,3	1,1	2,2	3,3	1,1	2,2	3,3

* Location of rear axle centerline as given in Fig. 8

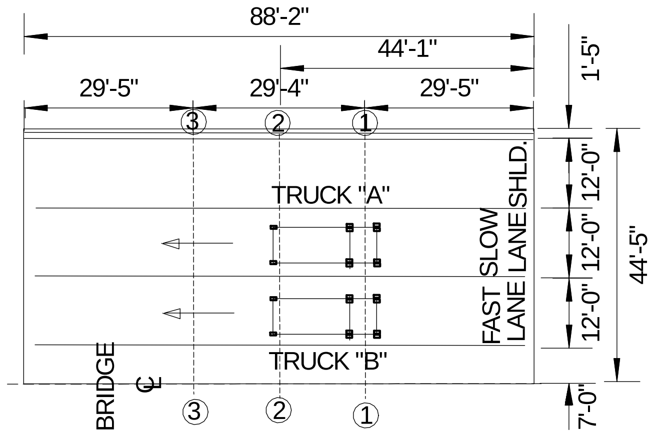


Fig. 8. Truck load Test 7

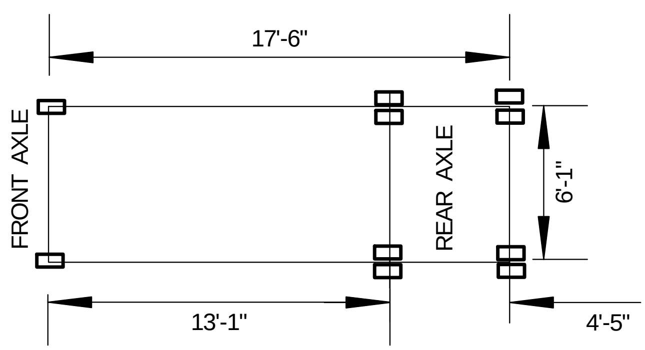


Fig. 9. Typical truck used in truck load tests

Table 2. Truck load axle weights

Truck	Truck Weight kips (kN)	Front Axle kips (kN)	Rear Axle Kips (kN)
"A"	43.88 (195.2)	14.78 (65.8)	29.10 (129.4)
"B"	43.16 (192.0)	14.48 (64.4)	28.68 (127.6)

BRIDGE DECK DEFLECTIONS

The relative deflections from the five LVDTs between the bridge deck and the west diaphragm for static tests 7, 8 and 9 are shown in Fig. 10. The highest deflections were found to be in the slow lane between girders 4 and 5 during Test 8 and Test 9; this is reasonable since both trucks "A" and "B" are parked close to the west diaphragm during these tests. The magnitude of the relative deflection is very small, well within the AASHTO LRFD Specifications[6] allowable limit of span/800 or 0.11 in. (2.9mm), which shows that the bridge deck and the girders have a good composite action.

PRESTRESSED GIRDER DEFLECTIONS

During the static truck load tests, a survey was carried out to measure the total deflection of all the prestressed concrete girders at midspan for each test. The maximum deflection occurred during Test 8 for girder 4, located below the right side of the left lane, as shown in Fig. 11. This is reasonable since both trucks "A" and "B" would be located at midspan in this configuration. It is also of note that even though two trucks side by side would be smaller than the HL-93 AASHTO Design Load, the deflection is significantly smaller than the allowable deflection equal to span/800 in the AASHTO LRFD Specifications[6]; the allowable deflection is 1.32 in. (34mm), whereas the maximum deflection observed is only 0.12 in. (3mm).

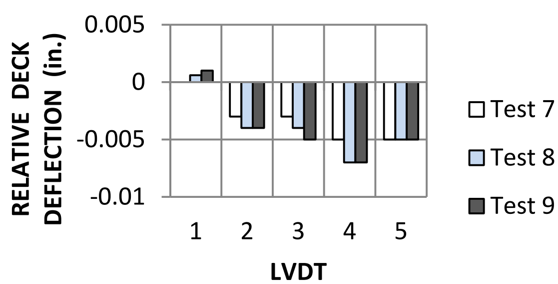


Fig. 10. Deck to girder relative deflections at west diaphragm

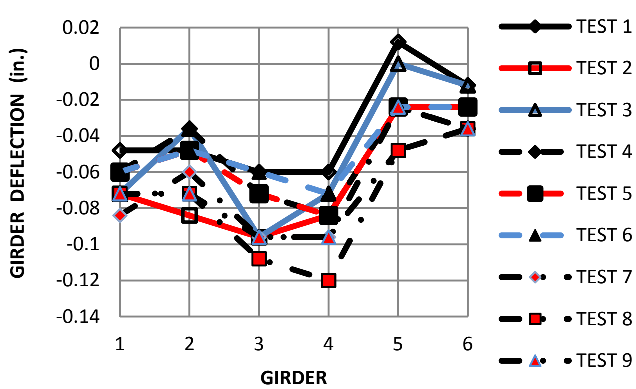


Fig. 11. Deflections at midspan of each girder during static truck loads

DYNAMIC TRUCK LOAD TESTS

The dynamic truck load tests were performed on September 29th 2009, and consisted of five tests. The dynamic tests are shown in Table 3. Accelerations measured at the midspan of all six girders during Test 14, for which Truck "B" was traveling in the fast lane at 65 mph (105 Km/h) are shown in Fig. 12. The maximum accelerations occurred in prestressed girders 4 and 5, located directly below the left lane which the truck was traveling. The maximum acceleration recorded during this test was 0.026g which was typical of all the tests. These accelerations are not expected to be the maximum accelerations that could possibly occur at the bridge. Long-term monitoring will continue for a period of three years.

Table 3. Dynamic truck load tests

Test	10	11	12	13	14
Truck	"A"	"B"	"A"&"B"	"A"	"B"
Lane	SLOW	FAST	SLOW & FAST	SLOW	FAST
Velocity mph (Km/h)	40 (64)	40 (64)	35 (56)	65 (105)	65 (105)

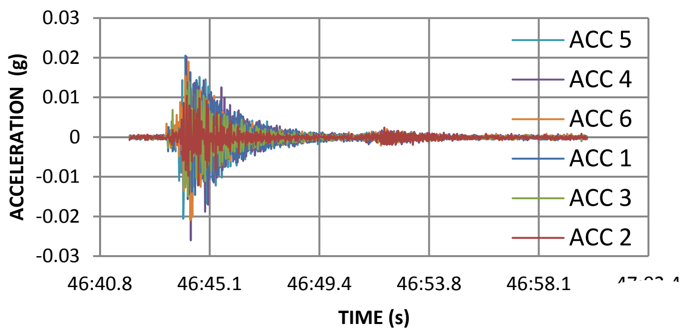


Fig. 12. Dynamic truck load Test 14: Truck "B" in fast lane at 65 mph (105 Km/h)

COST COMPARISON

The square foot cost of the precast concrete deck panels reinforced with GFRP was $59.25, a 20.8% increase over the average cost of precast deck panels reinforced with epoxy coated steel rebar. In addition, two additional girders were added to the Beaver Creek Bridge to better control deflection and cracking. Each additional girder was $48,000, equivalent to a $6.82 per square foot increase to the deck cost. Given a 45 year design life for the deck panels reinforced with steel and a 60 year design life for the panels reinforced with GFRP, the cost per year would be $1.09 and $1.10 per year per square foot respectively.

CONCLUSIONS

This paper presented the design, bridge construction details, and monitoring of the GFRP-reinforced precast concrete deck panels of the Beaver Creek Bridge on US 6 in Utah. In addition, a comparison of the cost of the GFRP reinforced deck versus a steel reinforced deck was carried out. Based on the measurements carried out during lifting, placement, as well as static and dynamic truck load tests, the following conclusions can be drawn:

1. The GFRP bars withstood normal handling at the precast yard and placement without any problems. In addition, their light weight made them easy to carry and easier to place.

2. During lifting, the maximum tensile strain in the GFRP bars was 150 microstrains. This strain is much smaller than the maximum tensile strain of 16,000 microstrains (only 0.94%). This suggests that the ACI 440 flexural design method is very conservative. However, the measured tensile strain indicates that the cracking strain was exceeded at some locations.

3. The relative deflections between the bridge deck and the west diaphragm were measured during the static tests. The magnitude of the relative deflections was found to be less than 0.007 in. (0.2mm) which is very small, well within the AASHTO LRFD Specifications allowable limit of span/800 or 0.11 in. (2.9mm), which shows that the bridge deck and the girders have a good composite action.

4. The live load deflection of the prestressed girders during the static truck load tests was found to be significantly smaller than the allowable deflection of span/800 specified in the AASHTO LRFD Specifications (2009). For two trucks located at midspan weighing a total of 87.04 kips (387kN) the maximum deflection observed was 0.12 in. (3mm), whereas the allowable deflection is 1.32 in. (34mm).

5. Accelerations measured at the midspan of the prestressed girders for a 43.16 kip (192.0kN) truck traveling in the fast lane at 65 mph (105 Km/h), the maximum vertical acceleration recorded during this test was 0.026g, which is acceptable from the serviceability point of view.

6. The tests carried out for the precast concrete deck panels reinforced with GFRP bars show that this is a viable construction method. The life cycle cost difference between GFRP and steel reinforced deck in this case is minimal. The cost savings for using GFRP come in the form of reduced user cost delays, due to less work and deck replacements required in the future of this structure. Long-term monitoring of the bridge is continuing.

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