DESIGN OPTIMIZATION OF BRIDGE DECKS WITH PRECAST UHPC WAFFLE PANELS

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ABSTRACT

The first full-depth, precast, ultra-high performance concrete (UHPC) waffle panels have been designed and implemented in a bridge replacement project to utilize accelerated bridge construction (ABC) and increase bridge deck longevity. After satisfactory performance of bridge deck under moving loads, this paper examines the options to optimize the bridge deck design to minimize the UHPC volume and associated labor costs. Using the full-scale finite-element model of the bridge, an optimization of the waffle panels was undertaken by varying the number of ribs as well as spacing between the ribs. An optimized panel was achieved by reducing the interior ribs per panel from four to two, or zero, in the longitudinal direction and six to two in the transverse direction, without compromising the panel's structural performance. Using the recommended optimized design, it was shown that the UHPC volume can be reduced by 13.4% compared to the design completed for the bridge, thereby significantly reducing the construction costs.

Author keywords: Precast; Waffle panel; Ultra-high performance concrete (UHPC); Bridge deck; Optimization; Design

1. Introduction

Current bridge infrastructure challenges in the U.S. caused by growing traffic volume and an increasing number of aging, structurally deficient or obsolete bridges, demand accelerated bridge construction (ABC) methods and structural systems with increased longevity. The Better Roads Bridge Inventory survey (2009) indicated that the deterioration of the deck is a leading cause for obsolete and/or a deficient inspection rating of the bridges. Due to the excellent durability and structural properties of ultra-high performance concrete (UHPC), it has been receiving more attention by bridge engineers as a means to increase the bridge service life and reduce life-cycle costs by requiring less maintenance (Piotrowski and Schmidt 2012).

The dense matrix of UHPC leads to enhance durability properties over the conventional concrete as measured by freeze-thaw tests, scaling tests, permeability tests, resistance to alkali-silica reactivity (ASR), abrasion tests, and carbonation (Russell and Graybeal 2013). Hence, the use of UHPC in bridge deck application prevents the detrimental solutions from infiltrating into the matrix when it is designed to be crack free and exposed to the environmental deterioration.

However, currently the UHPC's initial unit quantity cost far surpasses that of conventional concrete, which underscores the need for economy in its use, by optimizing the design as emphasized by the FHWA-HRT-13-060 report (Russell and Graybeal 2013). Additionally, utilizing precast concrete deck panels is gaining significant interest among several State Departments of Transportation (DOTs) for both new and replacement bridges, as a system promoting ABC (Terry et al. 2009). Previously, Issa and Yousif (2000) and Berger (1983) showed that the use of precast, full-depth concrete deck systems can significantly accelerate bridge construction/rehabilitation, resulting in minimized delays and disruptions to the community.

For the reasons noted above, the State of Iowa, which has the third highest number of deficient bridges in the U.S. (ASCE 2013), has been actively implementing UHPC in its infrastructure. The Iowa DOT led the nation with the implementation of UHPC Pi girders (Keierleber et al. 2008) and the development of an H-shaped UHPC precast pile for foundation applications (Vande Voort et al. 2008). In one of the recent projects sponsored by the FHWA Highways for LIFE (HfL), by combining the advantages of UHPC with those of precast deck systems, a bridge system with prefabricated UHPC waffle deck panels and field-cast UHPC connections was developed. Following a successful laboratory evaluation of the structural performance of waffle deck panels and suitable connections (Aaleti et al. 2011), a full-scale, 19.2 m (63 ft) long, single span demonstration bridge with full depth prefabricated UHPC waffle deck panels was constructed on Dahlonega Road in Wapello County, Iowa. This replacement bridge is the first UHPC waffle deck bridge in the U.S. and is used to demonstrate the deployment of the UHPC waffle deck technology from fabrication through construction.

After verifying satisfactory performance of bridge deck under moving loads (Honarvar et al. 2016), this paper investigates cost effective design alternatives to the deck design completed for Dahlonega Road Bridge with an intention of reducing UHPC volume and the waffle deck cost. An optimization of the waffle panels was undertaken by varying the number of ribs as well as the spacing between the ribs, using a finite-element model (FEM) of the bridge developed using ABAQUS. The design guidelines proposed for the implementation of UHPC waffle deck systems in new and replacement bridges, by Aaleti et al. 2013, were given consideration in the optimization study. Furthermore, girder live load moment distribution factors (DFs) of the optimized designs were calculated and compared with the current design to ensure that the optimal designs would not alter the distribution of loads between the girders and that the bridge superstructure would act effectively as an integral system.

2. Bridge Description

The single-span, two-lane Dahlonega Road Bridge, the replacement of an existing bridge in Wapello County, Iowa, is 9.14 m (33 ft) wide and 19.20 m (63 ft) long. It consists of fourteen prefabricated, full-depth, precast concrete panels installed on five standard Iowa “B” girders (Index of Beam Standards 2011) placed at a center-to-center distance of 2.33 m (7 ft and 4 in.). The bridge plan view, cross section, and construction photos are shown in Figure 1.

A single UHPC waffle panel of the Dahlonega Road Bridge deck is 5.5 m (16 ft and 2.5 in.) wide and 2.44 m (8 ft) long, as shown in Figure 2a. Note that the terms, longitudinal and transverse used throughout this document are relative to the bridge, not the panel. Each of the two cells in a panel have three interior ribs and two interior ribs in the transverse and longitudinal directions, respectively, and two exterior ribs in each direction, as illustrated in Figures 2b and 2c. Hereafter, the interior ribs in each cell of a panel are referred to simply as ribs. Each rib is 101 mm (4 in.) wide at the top with a gradual decrease to 76 mm (3 in.) at the bottom, and 140 mm (5.5 in.) deep. Longitudinal and transverse ribs were both reinforced with No.19 (No.6, d_b = 0.75 in., d_b is diameter of bar) bars at the top and the bottom. Stainless steel dowels with a diameter of 25 mm (1 in.) were used to reinforce the field-cast UHPC joints. The panels were connected across the length of the bridge using a transverse joint connection. In this connection, panel’s dowel bars were tied together with additional transverse reinforcement and the gap between the panels was filled with UHPC. In order to make the girders fully composite with the panels, a shear pocket connection and a waffle panel-to-girder longitudinal connection were provided.

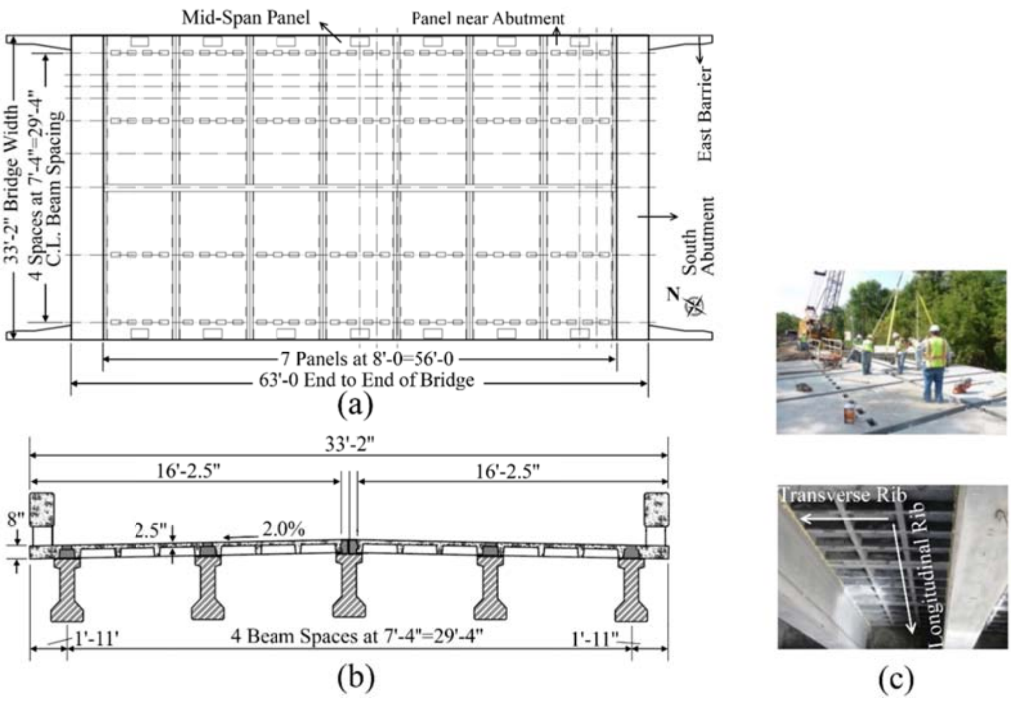

Figure 1. Dahlonega Road Bridge: (a) plan view; (b) cross section; (c) construction; Note: 1 ft = 0.305 m

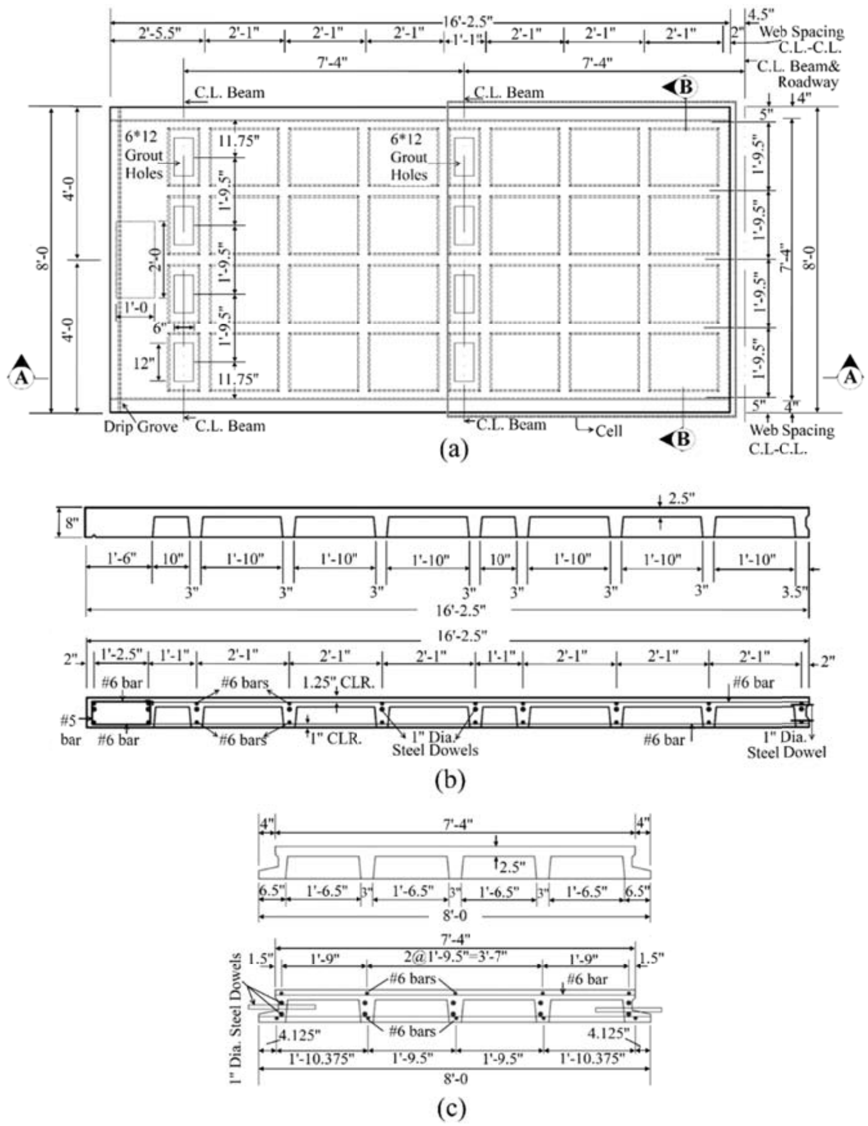

Figure 2. Single UHPC waffle panel: (a) plan view; (b) longitudinal cross section A-A (c); transverse cross section B-B; Note: 1 ft = 0.305 m

3. Numerical Assessment

A 3D nonlinear FEM was developed using ABAQUS software, Version 6-12. The geometric and reinforcement details were accurately employed in the FEM, as well as nonlinear material properties. The waffle deck, girders, and abutments were modelled with deformable 8-node linear 3D stress elements (i.e., C3D8R in ABAQUS). The steel reinforcement in the deck panels and the abutments were modelled using two-node linear 3D truss elements (i.e., T3D2 in ABAQUS), with perfect bonding to the concrete. The integral abutments were modeled in accordance with the bridge design to impose a compatible movement of the superstructure (i.e., panels and girders) with the abutments.

The concrete in the prestressed girders and abutments was modelled using an elastic material with Young's modulus of 32,874 MPa (4768 ksi), estimated using recommendations in

AASHTO 2010. The UHPC behavior in the deck panels was represented with an inelastic material with the softening behavior, and was modelled using the Concrete Damaged Plasticity (CDP) model, available in ABAQUS. The stress-strain behavior of UHPC in tension and compression used in the FEA is shown in Figure 3 (Aaleti et al. 2013). An idealized bilinear elastic plastic stress-strain material constitutive model was used to simulate mild steel reinforcement with Young's modulus of 199,947 MPa (29000 ksi), a yield strength of 414 MPa (60 ksi), an ultimate stress of 620 MPa (90 ksi), and an ultimate strain of 0.12. The load was applied in line with the truck configuration and load paths. Each axle weight was equally distributed between two wheels located 2.44 m (8 ft) apart from each other. Then, the analysis was solved using the Static Riks solver in ABAQUS.

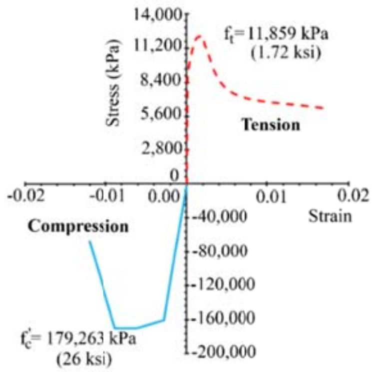

Figure 3. Stress-strain behavior of UHPC in tension and compression

4. Finite-Element Analysis Verification and Results

The FEM's accuracy in predicting the global bridge's response to loads applied during the field test was verified by comparing the calculated live load deflections and girder strains for load paths to the corresponding values measured during the test, as presented by Honarvar et al. (2016).

5. Optimization of Waffle Panels

The use of UHPC is limited in current day practice, partly due to high material costs, even though it exhibits superior structural characteristics, such as high compressive strength, reliable tensile strength, and improved durability. Therefore, for economical systems, an optimized design should be adopted to minimize the UHPC volume in structural members, without affecting the structural performance (Russell and Graybeal 2013). The newly developed design guide for the UHPC waffle deck (Aaleti et al. 2013) provides recommendations about the geometrical design of waffle panels, including, panel width, length, and thickness as well as rib dimensions and their spacing in transverse and longitudinal directions. Panel width and length are primarily governed by the bridge span and width, while the panel plate thickness is dictated by the punching shear capacity of the panel (Aaleti et al. 2013). The adequacy of punching shear capacity of 63.5 mm (2.5 in.) thick UHPC slab for bridge decks, subjected to AASHTO HL-93 truck [71.2 kN (16 kips) per tire] or Tandem truck [55.6 kN (12.5 kips) per tire] with the standard wheel load dimensions [254 mm (10 in.) by 508 mm (20 in.)], was validated. Aaleti et al. (2013) reported the measured average punching shear strength of 7.4 MPa (1.07 ksi), which was nearly 2.3 times the estimated value using the equation recommended by Harris and Wollmann (2005). Therefore, both punching shear capacity values reported by Aaleti et al. (2013) and Harris and Wollmann (2005) are greater than the punching shear that would be experienced by a bridge deck when subjected to AASHTO truck.

In the context of minimizing the volume of UHPC for waffle deck panels, the number of ribs and ribs spacing can be potentially altered to reduce the UHPC volume. The remaining structural properties of components, such as panel dimensions and deck reinforcement, were retained during optimization. In this study, two designs were investigated as alternatives to the waffle panel used in the Dahlonega Road Bridge, with the prospect of reducing the UHPC volume in line with the design guideline (Aaleti et al. 2013). The guideline recommends a maximum spacing of 0.91 m (36 in.) for the ribs in both longitudinal and transverse directions. However, these limits were slightly exceeded due to geometric constraints of the panel in the alternative designs.

The first alternative design reduced the number of ribs per cell, to one, in both longitudinal and transverse directions with a transverse and longitudinal rib spacing of 0.95 m (37.5 in.) and 1.05 m (41.5 in.), respectively. In the second alternative design, the longitudinal rib was eliminated as the load was primarily transferred in the transverse direction for the bridge deck. Therefore, the two longitudinal ribs in the original panel design were removed, while one transverse rib was retained. The elimination of the longitudinal ribs transformed the waffle slab effectively into the ribbed slab. It should be noted that the rib reinforcement [one continuous No. 19 (No. 6) reinforcing bar at the top and bottom of each rib] as well as rib tapering along the depth [101 mm (4 in.) wide at the top with a gradual decrease to 76 mm (3 in.) at the bottom] in the proposed designs were kept the same as the original design. Hereafter, the recommended designs are referred to as redesign 1 (i.e., the design with one rib in both directions) and redesign 2 (i.e., the ribbed slab). Panel geometrical details for the original design, and redesigns one and two, are demonstrated in Figure 4.

The field test results indicated that peak strains in the deck panels occurred primarily for load path two (center of traffic lane) and load path three (straddling bridge centerline). Thus, evaluating the performance of the alternative designs, the analysis was conducted for these load paths. The location of the maximum transverse strain at the bottom of each panel for load path two is demonstrated in Figure 4. The maximum estimated live load tensile strains at the bottom of the panel for the three designs are reported in Figure 5. It can be seen that the original design produced the smallest transverse strains, while redesign 2 produced the highest transverse strains. However, these strains are still lower than the UHPC cracking strain, thereby demonstrating satisfactory structural performance of the two proposed alternative designs. As expected, the longitudinal strains are fairly similar for the different designs. The strain distributions for the different designs at the critical location along the bottom of the mid-span panel were compared in Figure 6. The results indicate that the proposed redesigns do not significantly change the strain distribution trend when compared to the original design and field measurements.

In the design guide (Aaleti et al. 2013), it was recommended to provide at least one interior longitudinal rib between two consecutive girder lines in addition to the exterior longitudinal ribs to ensure adequate connections between two adjacent panels. However, the load transfer in the current bridge seems to be in the transverse direction rather than the longitudinal direction. Hence, the adequacy of the connection between the two adjacent panels was analytically examined for redesign 2. As an extreme case, it was assumed that no bonding existed between the two adjacent panels except for the regions where there were exterior longitudinal ribs, which provided connectivity. The analysis showed that the maximum differential vertical deflection between the two adjacent panels was 0.0002 m (0.01 in.), when the rear axle of the truck was placed in the mid-span panel. Consequently, the longitudinal rib can be removed without affecting the structural performance of the panels. The deflected shape of the two adjacent panels at the mid-span is illustrated in Figure 7.

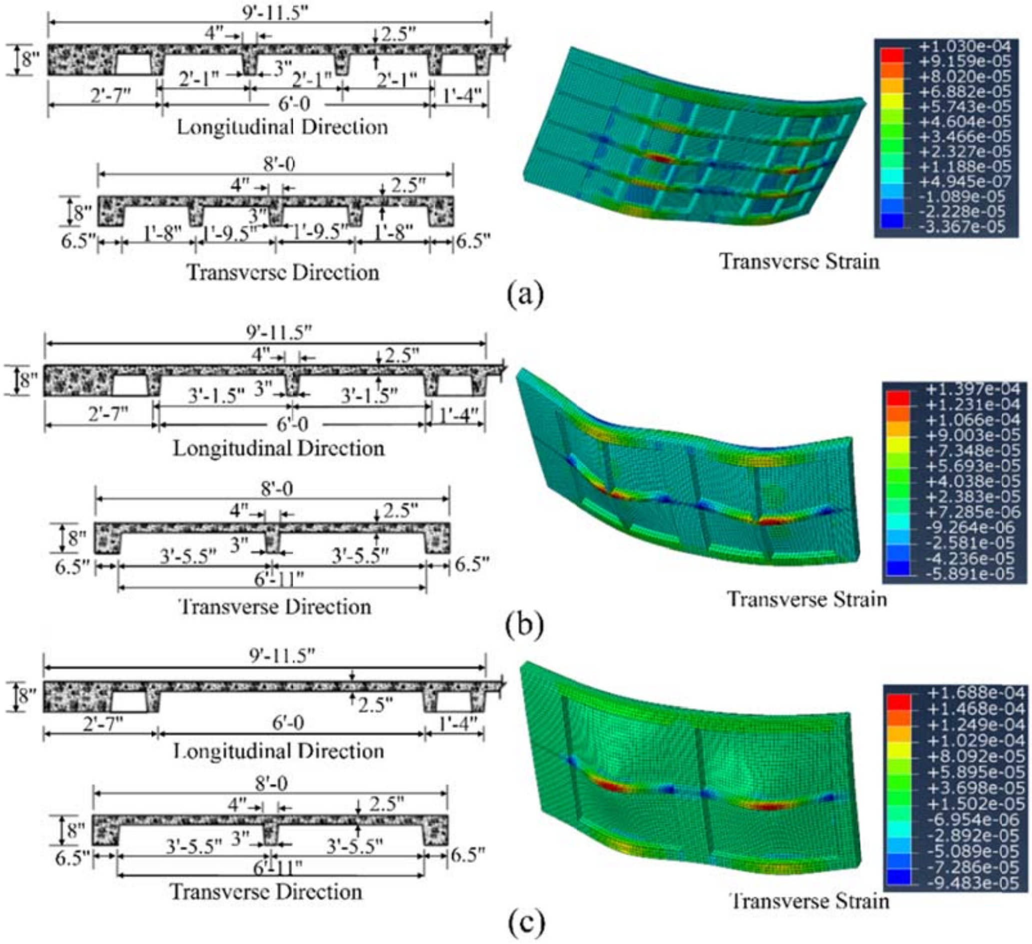

Figure 4. Panel transverse and longitudinal cross sections juxtaposed with transverse strain results for load path 2 at the mid-span panel: (a) original design; (b) Redesign 1; (c) Redesign 2; Note: 1 ft = 0.305 m

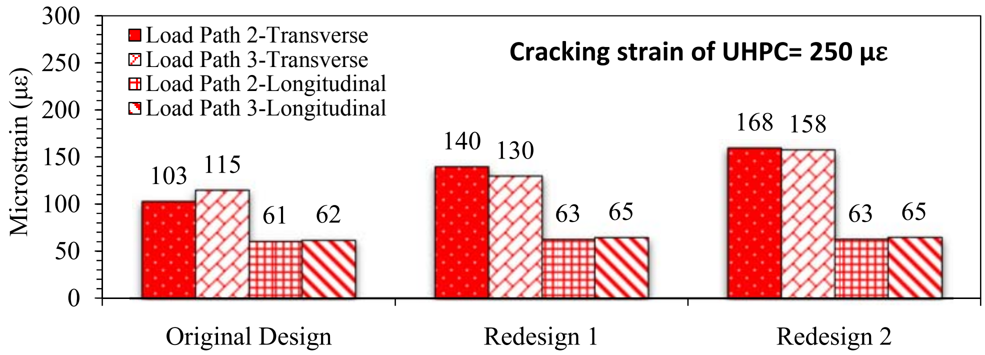

Figure 5. Maximum estimated live load strains for load paths 2 and 3

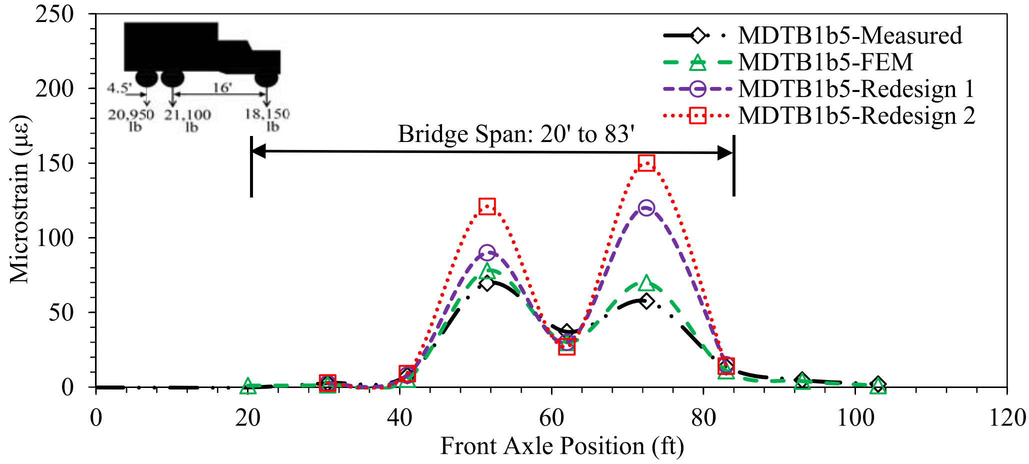

Figure 6. Comparison of transverse strains between field test and different designs at the bottom of the mid-span panel for load path 2. Note: MDTB1b5 is the strain gauge located on the central rib at the bottom of the midspan panel in the transverse direction; 1 ft = 0.305 m; 1 lb = 0.004448 kN

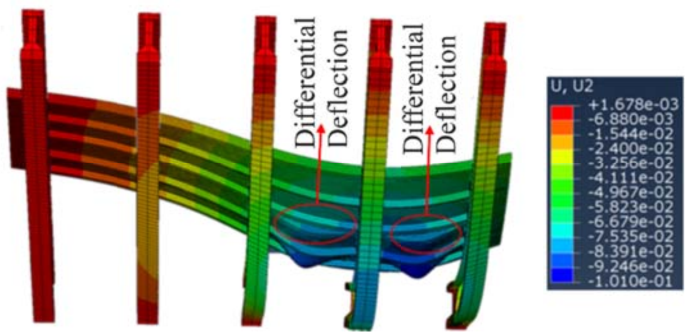

Figure 7. Differential vertical deflection between two adjacent panels at mid-span (in.); Note: 1 in. = 25.4 mm

Additionally, girder live load moment DFs for the proposed panel designs were estimated using vertical deflections of girders, and subsequently compared to those from the original design calculated with measured and estimated deflections using the FEM. The results from Table 1 indicate that DFs calculated for the different designs are fairly close to one another, as anticipated, since DF is mainly governed by the girders' spacing.

To quantify the cost effectiveness of the proposed designs, the volume of the UHPC used in a single panel, then the bridge deck, are calculated and presented in Table 2. For this specific bridge, the UHPC volume is reduced by 8.8% and 13.4% for the first and the second redesigns, respectively. This reduction in volume would decrease UHPC material costs as well as associated labor expenses. Furthermore, reducing the number of joints would also provide additional cost savings.

The positive and negative moment demands at the strength-I limit state (AASHTO 2010) were also computed and compared to factored flexural resistance (M_r) of each panel redesign, in accordance with a design guide for UHPC waffle deck (Aaleti et al. 2013). The results indicated that each redesigned panel would provide adequate flexural resistance to satisfy strength-I limit state loading (see Table 3).

According to the results of this finite element analysis, the two alternative designs can be used instead of the original design with acceptable structural performance. Evidently, the second redesign is more economical than the first redesign. Nevertheless, proper experimental validation of the two recommended deck panel redesigns is recommended prior to implementation in practice.

Table 1. Comparison of Girders Live Load Moment DFs for the Different Designs

Girder	Original design: Measured deflections	Original design: Estimated deflections	Redesign 1: Estimated deflections	Redesign 2: Estimated deflections
Interior	0.44	0.46	0.47	0.46
Exterior	0.34	0.31	0.30	0.31

Table 2. UHPC Volume for the Different Designs

Design	Single Panel Volume (m^3)	Bridge Deck Volume (m^3)
Original Design	1.61	22.54
Redesign 1	1.48	20.72
Redesign 2	1.42	19.88

Table 3. Strength I Limit State Moments for the Two Redesigns

Redesign	Positive moment (kN-m/m)		Negative moment (kN-m/m)	
	Demand	M_r	Demand	M_r
1	43.5	49.9	49.9	93.7
2	43.4	49.9	49.8	93.7

6. Summary and Conclusions

Following the satisfactory structural performance of the bridge under live load testing (Honarvar et al. 2016), cost effective deck panel alternatives, to that implemented in the field, were then explored with the objective of minimizing the UHPC volume and associated labor and material costs. Using the FEM, the optimization of the waffle panels was undertaken by varying the number of ribs as well as spacing between ribs, such that the structural performance of the panels would not be compromised.

The following conclusions can be drawn from this study:

- For the first recommended optimized design, the number of transverse and longitudinal interior ribs, per panel, was effectively reduced from six to two and four to two, respectively. This design was found to be appropriate, which reduced the UHPC volume by 8.8% compared to the original design.
- The analyses showed that the longitudinal interior ribs could be completely removed without affecting the connectivity of two adjacent panels. Therefore, in the second recommended optimized design, all longitudinal interior ribs were removed while retaining only two interior transverse ribs per panel. This alternative was also shown to be effective, which reduced the UHPC volume by 13.4% compared to the original design, with potential additional saving, that resulted from a reduced labor cost.

- For both optimized deck panel designs, the live load moment distribution factors and strain distributions remained the same as those obtained for the original design.

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