PRECAST UHPC WAFFLE DECK PANELS AND CONNECTIONS FOR ACCELERATED BRIDGE CONSTRUCTION

Sriram R. Aaleti, PhD, Post doctoral research associate, Department of Civil, Construction, and Environmental Engineering, Iowa State University, Ames, IA

Sri Sritharan, PhD, Wilson Engineering Professor and Associate Chair, Department of Civil, Construction, and Environmental Engineering, Iowa State University, Ames, IA

Dean Bierwagen, PE, Final design section leader, Office of Bridges and Structures, Iowa Department of Transportation, 800 Lincoln Way, Ames, IA

Brian P. Moore, Wapello County Engineer, 536 Mill Street, Ottumwa, IA

ABSTRACT

Prefabricated, full-depth, precast deck systems have been previously used to accelerate bridge rehabilitation, construction and extend service life with reduced user delays and lower life-cycle costs. Ultra high performance concrete (UHPC) has proven to be an efficient solution to increase bridge longevity. By combining the advantages of UHPC and precast deck systems, a prefabricated UHPC waffle deck system was developed as part of FHWA's Highways for LIFE program. The constructability of this system and structural performance of critical connections and panels were studied via large-scale tests at Iowa State University. Two prefabricated, full-depth, UHPC waffle deck panels (8-ft x 9-ft 9-in. x 8-in.), connected to two precast prestressed girders, and were subjected to AASHTO defined service and fatigue loads. Additionally, the waffle deck system was subjected to significantly higher loads than the AASHTO specified ultimate load, as well as loads causing punching shear failure. The experimental investigation was successful and validated the use of UHPC waffle decks in bridge applications. The waffle deck system performance was also studied

analytically and the corresponding results correlated well with the experimental data

Keywords: Accelerated Bridge Construction (ABC), Ultra high performance concrete (UHPC), Precast deck system, UHPC connections, waffle deck.

introduction

According to the national bridge inventory[1], over 150,000 bridges in the United States are either structurally deficient or functionally obsolete. Therefore, an urgent need exists to develop new techniques, materials, and systems for rehabilitation and replacement of these deteriorated structures. Additionally, the AASHTO strategic plan in 2005 for bridge engineering identified extending the service life of bridges as one of the greatest challenges. Producing safer, economical bridges at a faster rate, with a minimum service life of 75 years and reduced maintenance costs, is a driving objective to satisfy the country's infrastructure needs.

Precast concrete deck panels are being increasingly utilized by some state Departments of Transportation (DOTs)[2], for both bridge deck replacements and new structures, to reduce rehabilitation times and as a move towards accelerated bridge construction. Previous studies have shown that a prefabricated full-depth precast concrete deck system is an innovative technique that accelerates the rehabilitation process of a bridge deck, extending service life with reduced user delays and community disruptions, and lowering life-cycle costs. However, transverse connections previously used between precast bridge deck panels have exhibited various serviceability issues due to cracking and poor construction of connections.

Ultra high performance concrete (UHPC) is a newly developed concrete material that exhibits high compressive strength, dependable tensile strength and very low permeability. The superior structural characteristics and durability of UHPC could enable major improvements over ordinary concrete and high performance concrete (HPC) bridges, in terms of long-term structural efficiency, durability and cost effectiveness. Hence, the construction of new bridges and renewal of aging highway bridges using UHPC could lead to the construction of structurally-efficient long-life bridges that will require minimum maintenance, resulting in low life cycle costs. Previous use of UHPC for bridge applications (mostly in bridge girders) [3, 4] in the United States has proven to be efficient and economical.

Combining the advantages of UHPC and precast deck systems, a prefabricated UHPC waffle deck system with UHPC joint fills, to address previous connection issues, was developed as part of

FHWA's Highways for LIFE program. To gain an improved understanding of UHPC precast bridge deck panel behavior, the constructability of this system and the structural performance of critical connections, a large-scale test was performed at Iowa State University (ISU). Two prefabricated, full-depth, UHPC waffle deck panels, connected to two precast prestressed girders, and were subjected to service and fatigue loads, as defined by AASHTO[5]. Additionally, the waffle deck system was subjected to significantly higher loads than the AASHTO specified ultimate load and loads causing punching shear failure. Analytical modeling, using commercial finite element analysis software (ABAQUS[6]), was performed to predict the structural response and load distribution among the adjacent transverse ribs, for design recommendations. The summary of both the analytical and experimental results are presented in this paper.

PROPOSED UHPC Waffle deck system and connection details

Based on the research done at FHWA on structural behavior of a prestressed UHPC waffle deck system[7], a UHPC precast waffle deck system with conventional mild steel reinforcement was developed by Iowa DOT and ISU for accelerated bridge construction purposes. The UHPC waffle deck panel was designed for a two-lane single-span replacement bridge in Wapello County, Iowa (see Figure 1), which is scheduled for construction by the end of 2011. The waffle deck panel was 8 in. thick and was designed to resist current AASHTO load requirements. This resulted in Grade 60 No. 6 (d_b = 0.75-in, where d_b is diameter of the bar) and No.7 (d_b = 0.875-in.) mild steel reinforcement as top and bottom reinforcement respectively. All the reinforcement was provided along panel ribs in both directions. The plan view of the waffle deck panel showing the reinforcement and rib spacing is shown in Figure 2(a). The detailed information regarding the cross-section and reinforcement locations is presented in Aaleti et al.[8, 9]

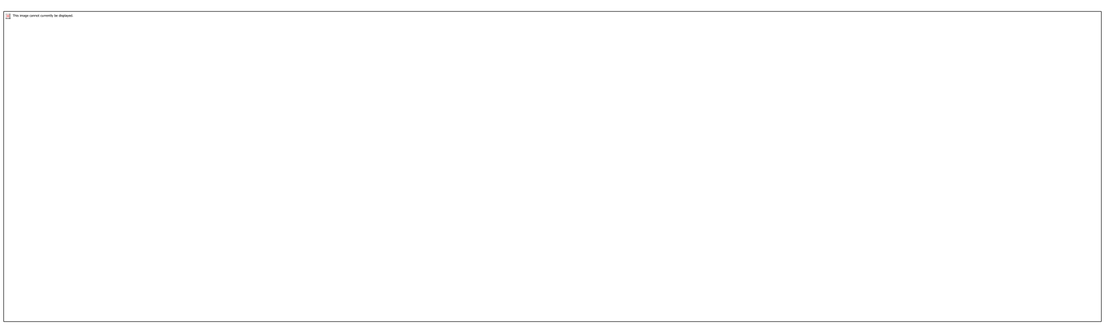
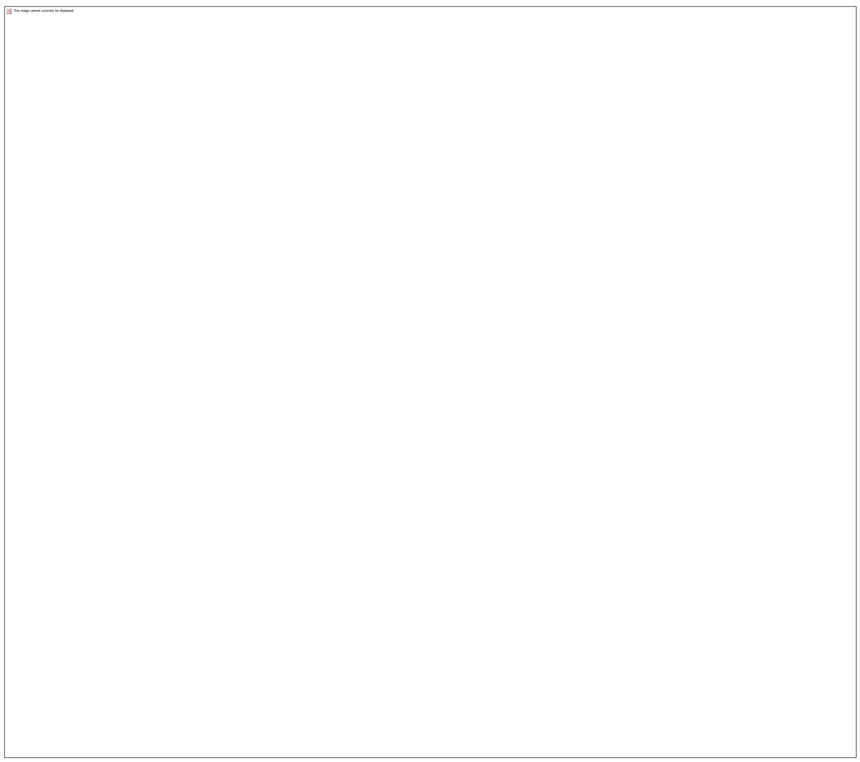
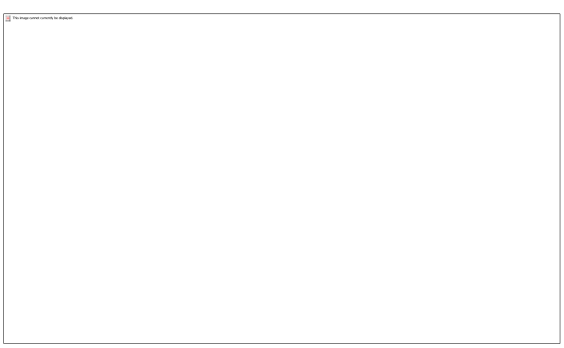


Figure 1: Cross-section details of the replacement bridge with UHPC waffle deck system in Wapello County, Iowa

To make the UHPC waffle deck panels fully composite with the prestressed concrete girders, three different connections were utilized, namely: 1) a shear pocket connection; 2) a longitudinal connection, and 3) a transverse connection. The shear pocket connection consists of a shear hook from the girder extended into a shear pocket in the waffle deck panel, with the shear pocket filled with UHPC (see Figure 2 (b)). The longitudinal connection between the waffle panel and girder was formed by tying dowel bars from the panels with shear hooks from the girder, using additional longitudinal reinforcement, then filling with UHPC (see Figure 2(c)). The transverse connection between the UHPC waffle deck panels contained dowel bars from the panels tied to additional transverse reinforcement, with the gap between the panels filled with UHPC (see Figure 2(d)).



1. Plan view details of the waffle deck panel (8ft by 10ft) used for experiemtnal testing



2. Shear pocket connection

3. Longitudinal connection

4. Transverse connection

Figure 2: Plan and the proposed connection details for the UHPC waffle deck system

test setup and instrumentation

For the experimental investigation, a waffle deck region between two adjacent girders, as identified in Figure 1, was chosen. Accordingly, two waffle deck panels (UWP1 and UWP2), 8-ft long by 9-ft 9-in. wide, were fabricated using a commercially available, standard UHPC mix. The waffle deck panels were cast upside-down for ease of construction. The details about the construction of waffle panels are presented in Aaleti et al.[9]

The test setup used for the UHPC waffle deck system testing was designed to closely replicate the critical regions of the field structure and is shown in Figure 3. The UHPC deck panels were supported on two 24-ft long prestressed concrete girders, which were 7-ft 4-in. apart and simply supported at the ends on concrete foundation blocks, as shown in Figure 3. The joints between the two deck panels, as well as those between the panels and the girders, were then cast using UHPC mixed in the laboratory at ISU. Refer Aaleti et al.[9] for more details about the test specimen construction and preparation of joints. Several string potentiometers and strain gauges were used to monitor the performance of the waffle deck system during testing. The instrumentation details are shown in Figure 4. A 10-in. by 20-in. steel plate attached at the loading end of a ±55 kip fatigue hydraulic actuator was used to simulate a truck wheel load on the panel for all testing described in this paper (see Figure 3).

a) Schematic of test setup

b) Completed UHPC joint fills

Figure 3: Schematic of the test setup used for testing of the UHPC waffle deck panel system

1. Location of string pots

b) strain gauges locations bottom reinforcement

Figure 4: Schematic of the displacement and strain gauges in the test unit

Experimental testing: results and discussion

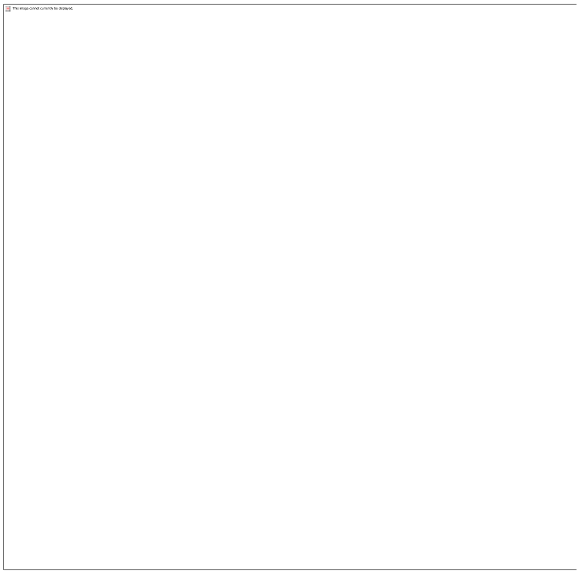
The performance of the UHPC waffle deck system, including the UHPC joints, was examined using nine different tests and a single wheel truck load. Two different locations were chosen to apply the load along the centerline between the two girders: 1) at the center of the deck panel and 2) at the center of the panel-to-panel transverse joint. The details of the load tests conducted are summarized in Table 1. This section focuses only on the results from service load tests and ultimate load tests, while the results and observations from the overload tests and fatigue tests are presented in Aaleti et al.[8]

Table 1: Sequence and details of the tests conducted on the Waffle deck system.

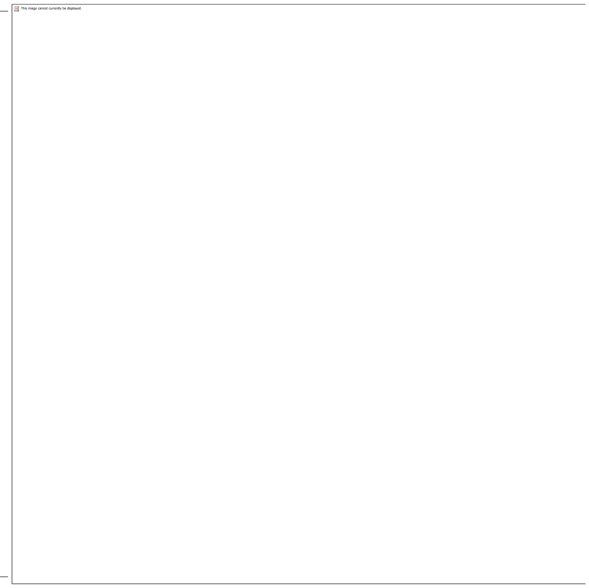
Test	Test Description	Location	Maximum Load
1	Service load test panel-2 (UWP2)	Center of the panel	1.33^{a} x 16 k = 21.3 k
2	Service load test on transverse joint	Center of the joint	1.75^{b} x 16 k = 28 k
3	Fatigue test on the transverse joint	Center of the joint	28 k (1 mil cycles)
4	Overload test of transverse joint	Center of the joint	48 kips
5	Fatigue test on the panel-1 (UWP1)	Center of the panel	21.3 k (1mil cycles)
6	Overload test of the panel	Center of the panel	40 kips
7	Ultimate load test on panel UWP1	Center of the panel	160 kips
8	Ultimate load test on the transverse joint	Center of the joint	155 kips
9	Punching shear failure test on UWP1	Between transverse ribs	155 kips

Panel and Joint Service Load Tests

In the panel service load test, a maximum load of 21.3 kips, representing the AASHTO truck service load plus 33% impact, approximately, was applied at the center of panel UWP2 (at rib TR2 and between the girders). In the joint service load test, a maximum load of 28 kips, approximately representing the AASHTO truck service load plus 75% impact, was applied at the center of the transverse joint. The load-deflection curves for both cases are shown in Figure 5. In both cases, a linear relationship was observed between the load and deflection. A maximum deflection of 0.034-in. and 0.022-in. were measured in the panel and joint service load tests, respectively.



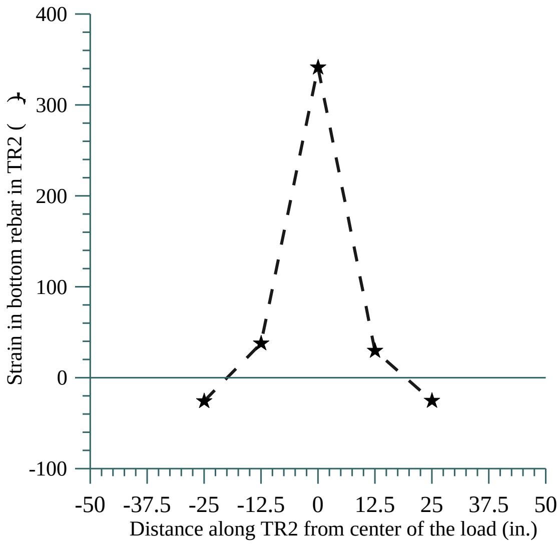
1. Force vs. displacement in panel service load test

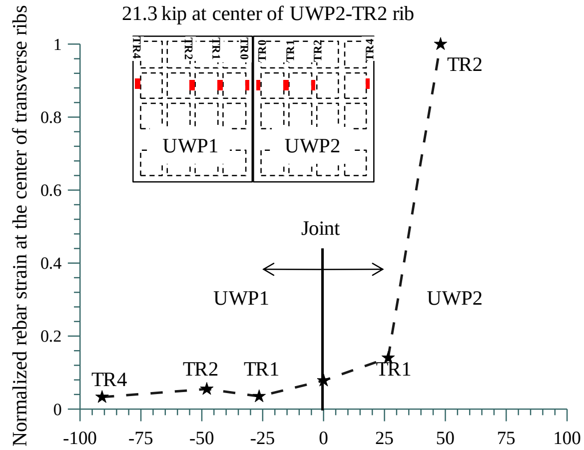


2. Force vs. displacement in joint service load test

Figure 5: Measured force-displacement response at the center of the waffle deck panel and the transverse panel-to-panel joint under service loads

The peak strain recorded in the bottom reinforcement of the center rib, running in the transverse direction, during the panel service load test was only $375\mu\varepsilon$, or 18% of the yield strain. The strain variation along the length of the bottom reinforcement, in the transverse rib TR2 of panel UWP2, and the variation of normalized bottom reinforcement strains at the center of the transverse ribs at the peak load are shown in Figure 6a. Figure 6b illustrates that for an applied load P at the center of the panel, the transverse rib TR2 provides 70% of the resistance. The adjacent ribs on either side of TR2 (i.e., TR1 and TR0; and TR3 and TR4) provide 10% and 5% of total resistance, respectively.





Strain distribution in the bottom rebar in transverse rib TR2 at 21.3 kips

2. bottom reinforcement strains at the center of transverse ribs

Figure 6: Measured strain distribution along the transverse rib in the center of the panel and normalized strains at the center of the transverse ribs along the longitudinal direction under service load conditions.

Panel Ultimate Load Test

The ultimate load test was carried out to investigate the adequacy of the precast deck system and its connections under ultimate load conditions. The ultimate load referred to in this study was arrived based on the recommendations from the Iowa DOT personnel. A total load of 160 kip, equivalent to 10 times the AASHTO truck service load, was applied at the center of panel UWP1. The load-deflection curve established at the center of this panel during testing is shown in Figure 7.

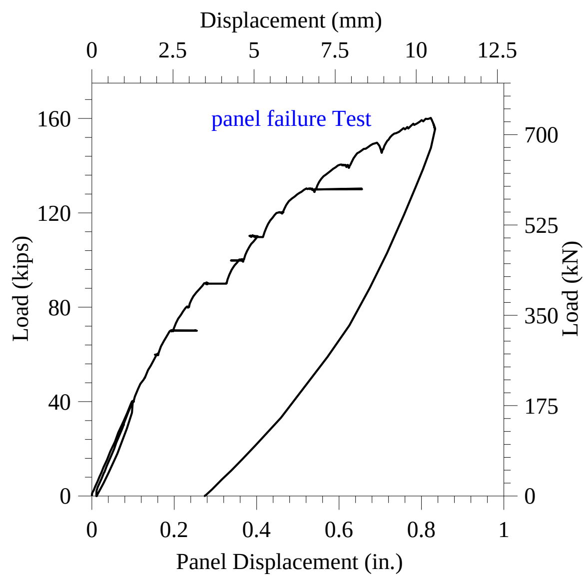
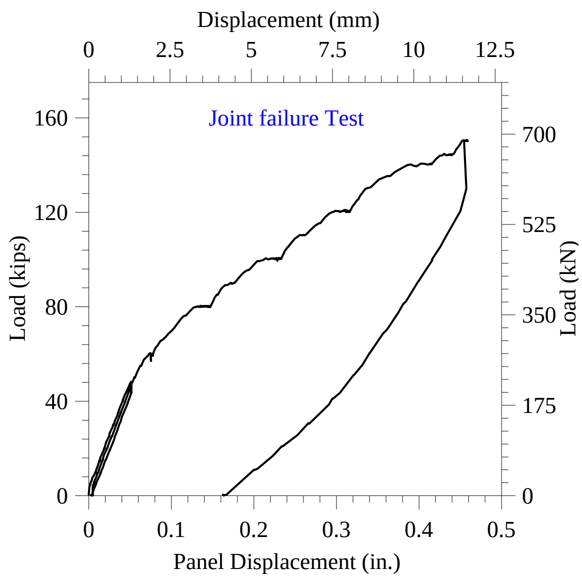


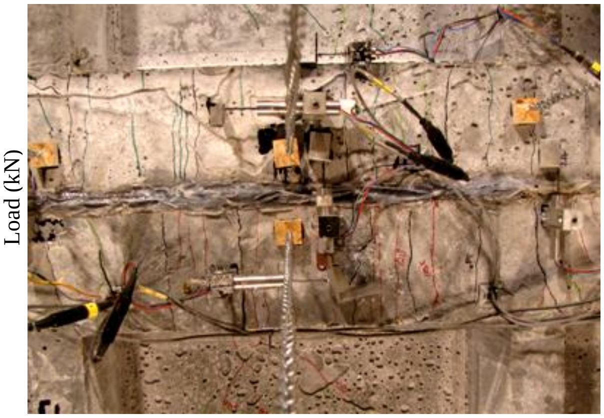
Figure 7: Measured force-displacement response of waffle deck system

The panel exhibited a linear force-displacement behavior response up to 80 kips. A maximum deflection of 0.82-in. was measured at the center of panel UWP1 (center of transverse rib TR2). The peak strain measured in the bottom reinforcement of transverse rib TR2 was around 1600 $\mu\varepsilon$, which is about 76% of the yield strain of the reinforcement. A significant amount of cracking was observed on both the transverse ribs (TR1, TR2 and TR3) and longitudinal ribs (LR1 and LR2) of panel UWP1. The maximum crack width measured along the transverse rib TR2 in UWP1 was 0.08-in.

Joint Ultimate Load Test

A total load of 155 kip equivalent to 10 times the AASHTO truck service load was applied at the center of the transverse joint. The load-deflection curve established at the center of the panel-to-panel joint is shown in Figure 8a. The peak strain measured in the bottom reinforcement of transverse rib TR2 was around 1475 $\mu\varepsilon$, which is about 70% of the yield strain of the reinforcement. At the end of the test, a large number of cracks were formed in transverse ribs of the joint (see Figure 8b). The maximum load applied was controlled by the shear cracking initiation in the prestressed girders.





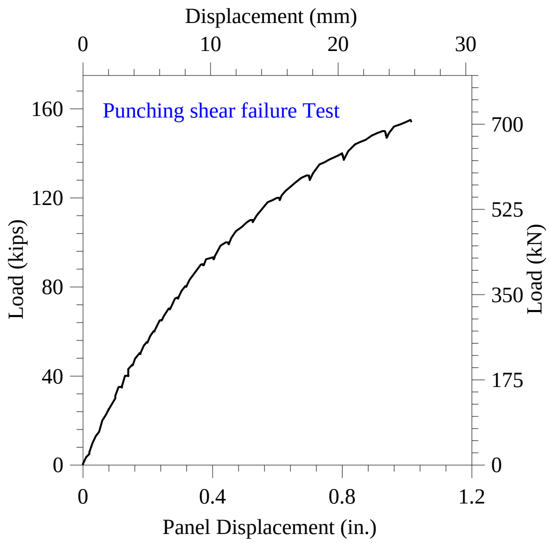
1. Force-displacement response

2. Cracks in the transverse joint at 150 kip load

Figure 8: Measured force-displacement response and cracking at the center of the panel-to-panel joint under ultimate loads

Punching Shear Failure Test

In this test, a wheel load was applied at the center of the waffle deck cell bounded by transverse and longitudinal ribs TR2, TR3, LR1 and LR2. Load was applied at increments of 5 kips on the waffle deck panel, using a 200 kip actuator. The 10-in. by 20-in. plate at the loading end of the actuator was replaced with a 6-in. by 8-in. steel plate to cause the punching shear failure in the panel. As the loading increased a large number of radial cracks in the top surface and flexural cracks in the ribs were formed. The measured load-displacement response at the center of the cell is shown in Figure 9a. The crack pattern on the bottom surface of the waffle deck was as expected for a typical punching shear failure, and is shown in Figure 9b.



1. Load-displacement response

2. Crack pattern in the cell

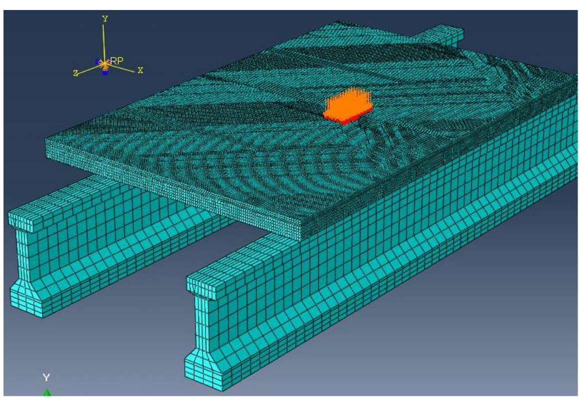
3. Punching failure surface

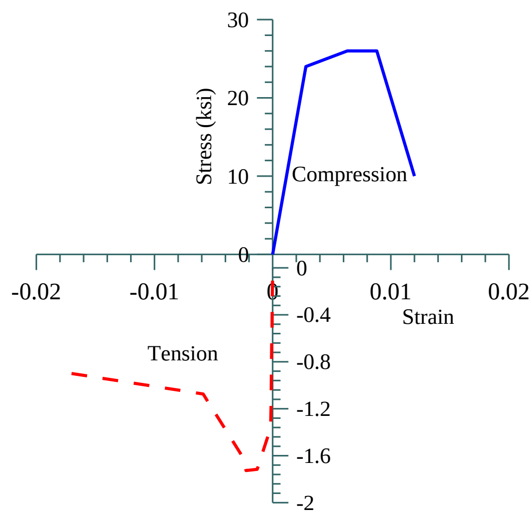
Figure 9: Measured load-displacement behavior and failure surface during the punching shear failure test of waffle deck system

The waffle deck failed suddenly at a maximum load of 154.6 kips, with a 6-in. by 8-in. hole (i.e., the same size as the steel plate placed at the top of the deck) at the center of cell. The punching shear failure surface had edge slopes of approximately 45 degrees, as shown in Figure 9c. The measured average punching shear strength is around 1.068 ksi, which is equivalent to 6.62 $\sqrt{f_c'}$(psi). The measured punching shear failure capacity is nearly 2.3 times the estimated value using the ACI equation recommended by Harris and Wollmann.[10]

finite element modelling

Nonlinear finite element analysis (FEA) was carried out to model the system using ABAQUS software, Version 6.10. In this paper, selected results of the FEA are presented to support the experimental results and extent of damage. The exact geometric and reinforcement details, as well as the nonlinear material properties of the system components, were employed in the FEA. The finite element model (FEM) was constructed using three-dimensional (3D) deformable elements. Meshing of the waffle deck panel and the prestressed concrete girders was completed using linear 3D stress elements (i.e., C3D8R in ABAQUS), with 8 nodes and 1 integration point per element. A mesh size between 1 and 2-in. was chosen for the deck panels, to provide more realistic stress and strain predictions in the critical regions. The panels were appropriately partitioned to allow structured meshing to be used, resulting in rectangular dominated elements. The mild steel reinforcement was modeled as wire beam elements with an appropriate cross-sectional area, with perfect bond between the steel reinforcement and concrete. The longitudinal and shear pocket connections between the UHPC waffle deck panels and the girders were modeled using kinematic constraints. The meshed assembly of the test specimen FEM is shown in Figure 10a.

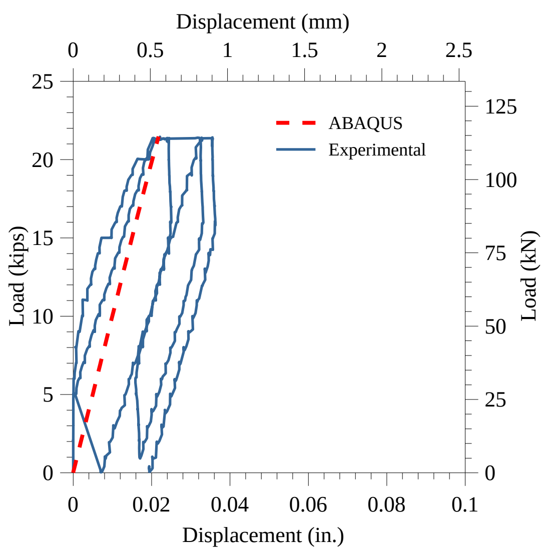




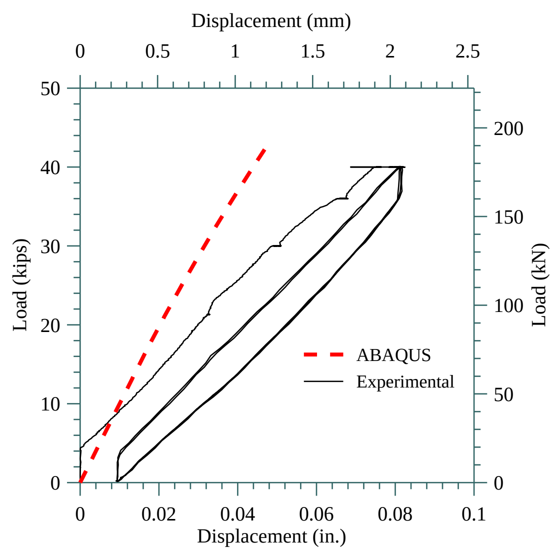
1. FEA model of the test specimen
2. Stress-strain behavior of UHPC

Figure 10: Test specimen discretization and material behavior of UHPC used in FEA software (ABAQUS)

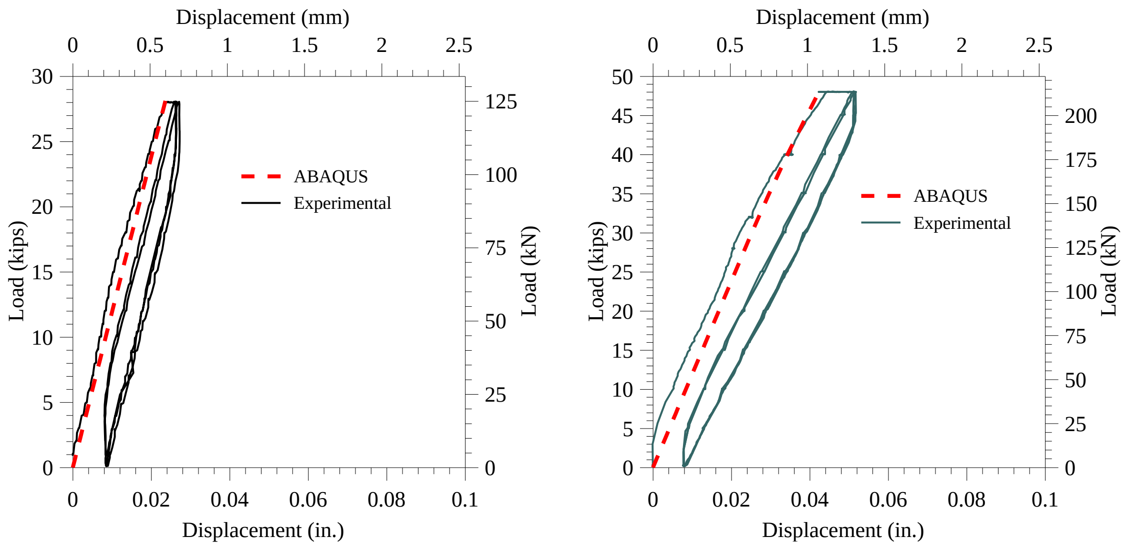
The UHPC in the deck panels and joints was defined using the “concrete damaged plasticity” model available in FEA software (ABAQUS). The stress-strain definition for UHPC was derived for an assumed 26 ksi compressive strength for deck panels and 18.5 ksi for the connection regions. The tensile stress-strain behavior of the UHPC was adopted from results of a direct tension test on dog-bone shaped UHPC coupons. A steel material model was defined to simulate the mild steel reinforcement properties, with an idealized bilinear stress-strain material model used, based on an elastic modulus of 29000 ksi, a yield stress of 60 ksi, an ultimate stress of 90 ksi, and an ultimate strain of 0.12. The UHPC stress-strain definition input into the FEM is shown in Figure 10b. The load was applied as a pressure load on the UHPC panel. The static-risks solver in ABAQUS was used for the analysis. Comparisons of the force-displacement responses from the FEM, with the measured response for service and overload cases, are presented in Figure 11. From this figure, it is evident that the FEM was able to accurately capture the force-displacement response at the transverse joint. However, the FEM underestimated the load-displacement response at the center of the panel by 30% in the overload case.



1. Panel service load test



2. Panel overload test



3. Joint service load test

4. Joint overload test

Figure 11: Comparison of experimental and FEA force-displacement responses.

conclusions

Based on the experimental testing of the UHPC waffle deck system under service and ultimate load conditions, and finite element modeling, the following conclusions were made:

1. The UHPC waffle deck system showed exceptional structural behavior in overall, confirming the adequacy of the reinforcement provided to sustain the design loads, and supporting the usage of UHPC infill for the joints.

2. The displacements in the bridge deck system under service loads were well below the AASHTO specified limits.

3. The UHPC waffle deck system was able to sustain loads up to 10 times the AASHTO specified design truck loads, implying girder spacing can be increased without causing any damage to the deck system.

4. It is expected that the waffle deck panels, when used in the prototype bridge, would form hairlines cracks on the underside of the deck only under service load conditions.

5. The UHPC waffle deck system will not experience punching shear failure under traditional 10-in. by 20-in. wheel loads. The measured punching shear capacity of the waffle deck panel system was nearly 2.3 times the estimated value using the ACI equation recommended by Harris and Wollmann.[10]. This supports the possibility of reducing the thickness of the waffle deck top surface and improving the cost-

effectiveness of the system. However, the service level cracking should be given due consideration.

6. The load applied at the center of the waffle deck panel was distributed among all the transverse ribs. When the wheel load was applied at the center of the waffle deck panel, the load was distributed at a ratio of 7:2:1 among the central transverse rib, adjacent ribs and the edge ribs.

7. A 3D finite element model in ABAQUS was able to accurately capture the force-displacement response and the extent of damage of the waffle deck system under various load conditions.

acknowledgement

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