Four High Performance Concrete Deck Configurations for Louisiana's Movable Bridges

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Four High Performance Concrete Deck Configurations for Louisiana's Movable Bridges

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Abstract

Louisiana has approximately 160 movable bridges, mostly in the southern part of the state. The typical deck systems in these movable bridges are steel grids. Records show that steel grids have had maintenance issues. Four alternative high performance concrete (HPC) bridge deck configurations were developed for Louisiana's movable bridges using four unique concrete mixtures. The development of each concrete mixture is presented. Additionally, each mixture is characterized in terms of its compressive strength, tensile strength, modulus of elasticity, and Poisson's ratio. Several nonlinear finite element analyses are performed to simulate the behaviour of all four deck configurations from the onset of loading to failure. AASHTO's ultimate load demand is met regardless of which deck configuration is selected. The panel that features the LHWPC 130 mix exhibited the highest peak load and offers the simplest geometry.

Keywords:

Movable bridge decks; high-performance lightweight concrete; finite element analysis.

1 Introduction

Louisiana has approximately 160 movable bridges, mostly in the southern part of the state. This places Louisiana among the states with the highest inventory of movable bridges in the nation. The typical deck systems in movable bridges are open steel grids, which typically consist of either diagonal or rectangular grids (Fig. 1). The traditional steel grid decks are supported by steel stringers at typically 1.22 m on center. On average these decks weigh less than 1.20 kN/m^2; while some others can weigh as little as 0.67 kN/m^2 [1]. This deck system is attractive because it is light weight, the panels are prefabricated and they are easy to install and replace. However, records show that steel grids have exhibited durability concerns. The proximity of these exposed steel systems to humid environments leads to rapid deterioration. As a result, decks become loose, causing extreme noise. These problems are aggravated by trapping foreign debris throughout the deck grids. Other problems associated with steel grid decks are unpleasant ride quality caused by panels becoming loose, and possible safety issues caused by a reduction in skid resistance due to use and deterioration.

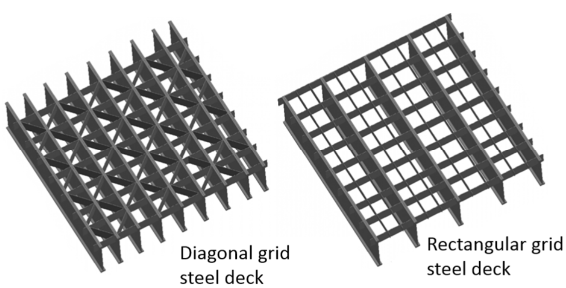

Figure 1. Steel grid decks for movable bridges

The Louisiana Department of Transportation and Development (LADOTD) has an interest in using concrete decks to replace deteriorated steel grids on existing movable bridges as well as in new construction. However, the mechanical systems of moveable bridges are highly sensitive. As a result, any decking used to replace or rehabilitate the existing steel grid decking should match the weight of the existing steel grid such that the mechanical system operates as designed.

Accordingly, four light high performance concrete (HPC) deck configurations are proposed as alternatives to steel grid decking using non-proprietary concrete mixes. The HPC deck systems are intended to provide a continuous driving surface that mimics monolithic construction, provides integral connections with the supporting stringers as well as between adjacent deck panels, and provides traction, which should improve traffic safety.

2 Background and Objective

Baghi et al. [2] developed a concrete deck configuration for Louisiana's movable bridges using the proprietary concrete mix *Ductal*, marketed by Lafarge. Florida Department of Transportation (FDOT) in collaboration with URS Corporation identified several potential alternative lightweight solid deck systems to replace steel open grid decks on typical Florida bascule bridges [1], [3]. The concrete deck systems featured the proprietary concrete mix *Ductal* marketed by Lafarge.

The goal of this research is to develop four HPC deck systems for Louisiana's movable bridges using non-proprietary concrete mixes. The performance requirements for these deck configurations are: 1) The maximum weight should not exceed 0.96 kN/m^2, 2) The maximum span length and overall depth of the deck system to be considered is 1270 mm, and 132 mm, respectively, 3) they need to meet load demands specified in AASHTO [4], 4) they need to feature corrosion resistant reinforcement, and 5) they need to feature panel to panel and panel to stringer connections that are intended to emulate monolithic action.

3 Mixture development

A total of four non-proprietary HPC mixes were investigated with the purpose of using them in the development of alternative deck panel configurations for Louisiana's movable bridges. The investigation included a material characterization study in terms of compressive strength, tensile strength, modulus of elasticity, Poisson's ratio, flow, and unit weight. The objective was to develop concrete mixes that featured low unit weights and high compressive strengths.

The four investigated mixes are called VHPC, LWHPC140, LWHPC130, and LWHPC120. Table 1 provides the mix designs for all four concrete mixes. VHPC stands for very high performance concrete. Coarse limestone aggregate (6.35 mm) and fly ash were only used in the VHPC mix. The designation LWHPC stands for lighter weight high performance concrete and it was used for the three mixes that were expected to have a lower unit weight. The ground quartz typically used in UHPC mixes was replaced by Louisiana Glass Powder supplied by Vitro Minerals. The most unique feature of the LWHPC mixes was the introduction of expanded quartz Poraver beads manufactured by Poraver. The expanded quartz Poraver beads were introduced to reduce the unit weight of the mix by replacing a portion of the fine sand. Chryso Fluid Premia 150 was used as a high range water reducer (HRWR) (superplasticizer) in all mixes.

Table 1. Mix design for four high performance concrete mixes

Constituent	Mix ID [kg/m^3]			
	VHPC	LWHPC 140	LWHPC 130	LWHPC 120
Portland cement I/II	665	712	712	712
Silica Fume	143	231	231	231
Fly Ash	143	0	0	0
Glass powder	0	211	211	211
Fine Sand	860	673	449	192
Poraver Beads	0	120	199	287
6.35 mm max. coarse aggregate	368	0	0	0
Water	189	168	183	199
HRWR	14.4	47	52	57
Steel Fibers	157	156	156	156
W/Cm	0.2	0.18	0.19	0.21

All mixes contained 2% fibers by volume. The steel fibers were 0.2 mm in diameter and 12.7 mm in length. Based on the specific gravity and bulk density, the Absolute Volume Method (AVM) [5] was utilized to obtain the mix designs.

Compressive strength, tensile strength, and modulus of elasticity tests were performed on 51×102 mm^2, 102×203 mm^2, and 152×305 mm^2 cylinders, respectively. The compression tests were performed in accordance with ASTM C39 [6]. The uniaxial behaviour of mixes in tension was characterized by performing splitting tensile strength tests using the approach recommended by Graybeal [7], who concluded that an adaptation of ASTM C 496 [8] splitting tensile test showed to provide a practical means for determining the tensile cracking strength of UHPC. Modulus of elasticity tests were performed based on ASTM C469 [9]. All test specimens were moist cured until the day they were tested.

The wet unit weight of the mixes was measured as soon as mixing operations finished and the wet concrete was placed in the cylinder molds. Static and dynamic flow tests were conducted for all mixes to measure the ability of each mix to self-consolidate. Because the VHPC mix was relatively stiffer compared to the other four mixes a slump test was performed in addition to the flow test to compare the viscosity of this mix with typical values reported for normal strength concrete.

Table 2 provides a summary of the unit weight and flow measurements. Both dry and wet unit weight were measured because the movable bridge deck will feature precast and cast-in-place components. VHPC was the densest mix and therefore its unit weight was the highest compared to the other three mixes. The measured dry and wet unit weights for the four mixes varied from 1858 kg/m^3 to 2370 kg/m^3 and 1826 kg/m^3 to 2355 kg/m^3, respectively.

Two of the lighter weight mixes, LWHPC130 and LWHPC120, exhibited static and dynamic flows of 178 mm and 229 mm. These values are consistent with those typically observed for proprietary mixes such as *Ductal* and are an indication of the self-consolidating nature of these mixes. The LWHPC140 mix was stiffer than the other two lighter weight mixes but more viscous than the VHPC mix. The measured slump for VHPC was 190 mm, which is indicative of a workable mix provided that vibrators or similar concrete consolidating equipment are used to place the mix.

Table 2. Unit weight, temperature, and flow

Mix ID	Unit Weight [kg/m^3]		Flow [mm]		Slump [mm]
	Dry	Wet	Static	Dynamic	
VHPC	2370	2355	102	152	190
LWHPC 140	2066	2050	102	178	NC
LWHPC 130	1954	1938	178	229	NC
LWHPC 120	1858	1826	178	229	NC

NC = not conducted

The compressive strengths at 28 days varied from 63 MPa to 119 MPa (Table 3). In all cases a reduction in unit weight came at the expense of a reduction in compressive strength. The load versus deformation response was recorded and was used to obtain a stress strain curve, which was subsequently utilized in the design of the deck panels (Fig. 2).

Table 3. Material properties

Mix ID	f'_c [MPa]	f_t [MPa]		E [MPa]	υ
		f_{tm}	f_{tu}		
VHPC	119	9.1	20	37000	0.15
LWHPC 140	91	8.0	18.2	30000	0.18
LWHPC 130	73	6.2	14.3	28000	0.17
LWHPC 120	63	5.9	11.8	23000	0.19

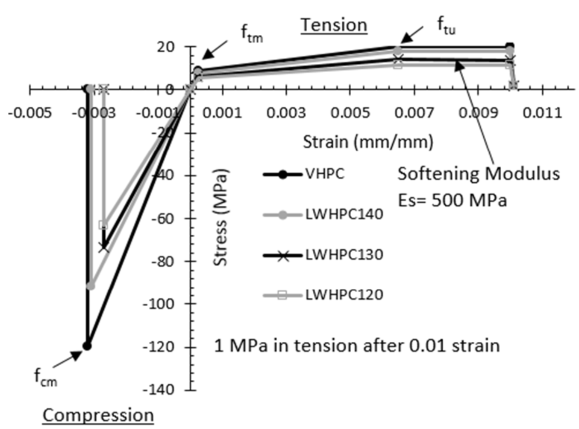

Figure 2. Stress-strain relationship for VHPC, LWHPC 140, LWHPC 130, and LWHPC 120

The tensile behavior of the test cylinders was characterized by the formation of the first crack along the height of the cylinder immediately underneath the load, the widening of the crack, and the formation of additional cracks in that vicinity under higher loads. The load that caused the first crack was used to calculate the first cracking strength (f_{tm}), and the peak load was used to calculate the ultimate strength (f_{tu}) at 28 days. The peak load was always higher than the load that caused the first crack, which is an indication of a strain hardening cementitious composite mix. The first cracking strengths varied from 5.9 MPa to 9.1 MPa. The ultimate tensile strengths varied from 11.8 MPa to 20 MPa.

Two values were obtained from the modulus tests: 1) the modulus of elasticity, and 2) Poisson's ratio. The moduli of elasticity varied from 23000 MPa to 37000 MPa. The modulus of elasticity decreased as the unit weight decreased. Poisson's ratio was calculated by diving the measured lateral strain to the measured longitudinal strain recorded during the modulus of elasticity tests and was subsequently used in the nonlinear finite element simulations. Poisson's ratios varied from 0.15 to 0.19.

4 Deck Panel Configurations

The four non-proprietary concrete mixes were used in the development of four deck panel configurations for Louisiana's movable bridges. The biggest challenge in developing the four deck panel configurations was the limitation on the maximum panel weight while satisfying the panel thickness and span length recommendations. Fig. 3 shows the top views of the deck panel configurations developed using each concrete mixture. These top views feature two adjacent precast deck panels in the transverse direction of the bridge (perpendicular to traffic). The gray bands represent the cast-in-place concrete continuity diaphragm and cast-in-place concrete fill between adjacent precast deck panels. The number of transverse and longitudinal ribs in the VHPC and LWHPC140 configuration is identical. The deck configurations for these two concrete mixes have four transverse ribs and six longitudinal ribs. The deck configuration for the LWHPC130 mix features only transverse ribs and the deck panel configuration for the LWHPC120 mix features four transverse and four longitudinal ribs. All deck configurations feature corrosion resistant reinforcement; either carbon fiber or glass fiber reinforcement.

Fig. 4 illustrates sections cut in the longitudinal direction of the bridge featuring two adjacent deck panels. Fig. 5 illustrates interior rib details for all deck panel configurations. All reinforcing bars are GFRP V-ROD HM – 60 GPa Grade III. These GFRP bars are corrosion resistant and are manufactured by Pultrall Inc. The thickness of the flange varies from 22 mm to 32 mm and the flange is reinforced with either a two-way carbon fiber mesh called C-grid manufactured by Chomarat North America, or GFRP bars. The diameter of the strands in the C-grid is 2 mm and the spacing of the strands is 41 mm in the longitudinal direction and 46 mm in the transverse direction (C50-46×41). The self-weight of the deck panels in all cases considering the cast-in-place concrete is less than the 0.96 kN/m^2 limit.

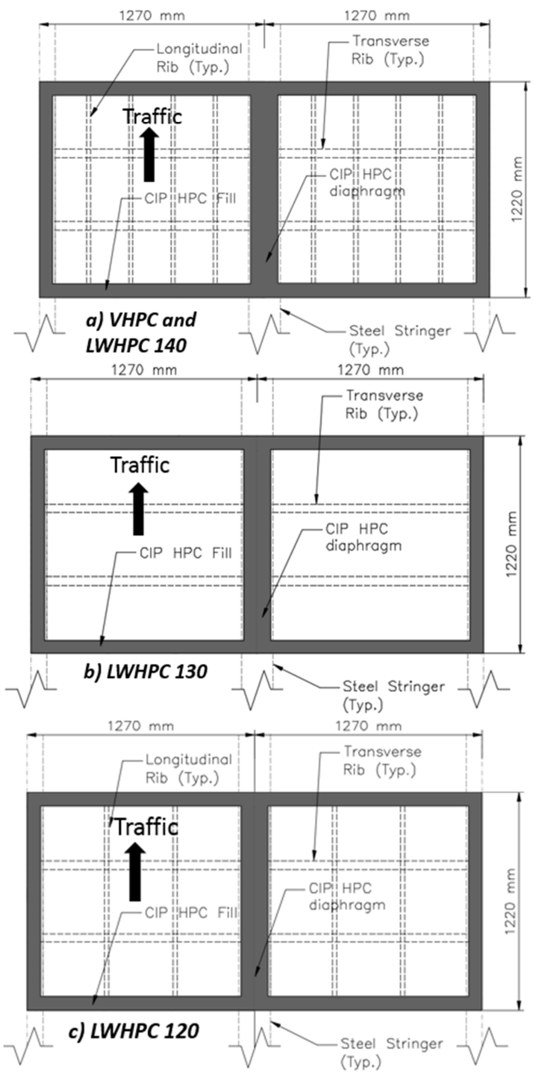

Figure 3. Top view of all deck panels (all dimensions in mm)

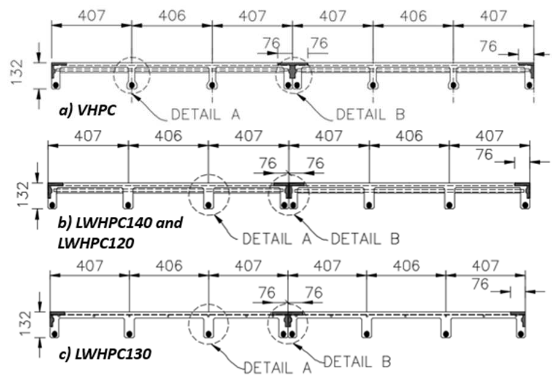

Figure 4. Longitudinal section (all dim. in mm)

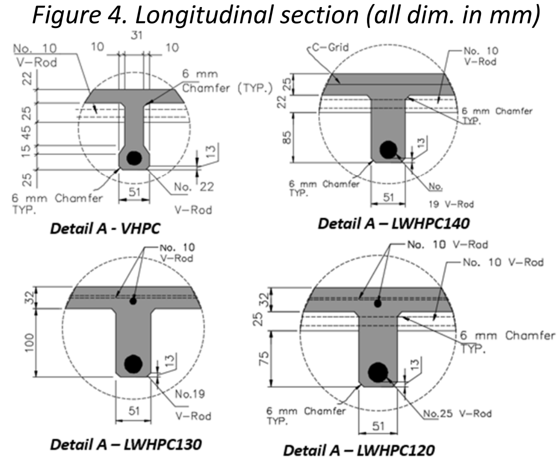

Figure 5. Typical interior transverse ribs details (all dim. in mm)

Fig. 6 illustrates typical panel to panel and panel to stringer connection details. The panel to panel connection features a female to female type shear key filled with cast-in-place high performance concrete. The top flange is coped to allow the lapping of the C-grid or GFRP bars from the adjacent panels. The panel to stringer connection detail features a cast-in-place concrete continuity diaphragm and headed studs. The top flange is coped similarly to allow the lapping of top reinforcing for continuity.

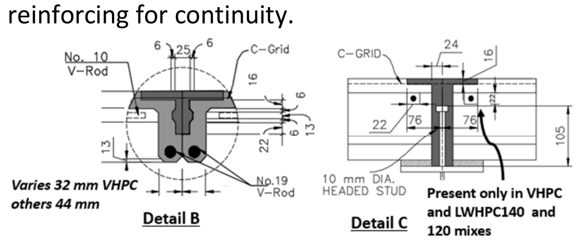

Figure 6. Typical panel to panel and panel to stringer connection details (all dim. in mm)

The next section describes the nonlinear finite element analyses that had to be performed to develop the deck panel configurations presented in this section.

5 Finite Element Analyses

The deck panel configurations presented in the previous section were developed after performing several iterations of nonlinear finite element analyses with the purpose of finding the most efficient configuration that met the requirements stipulated earlier in this paper. Each configuration was loaded monotonically using nonlinear finite element analyses to investigate the behavior of the proposed deck panels from the onset of loading to failure. The commercially available finite element analysis software Abaqus [10] was used in all numerical simulations. 3D continuum elements were used in all investigations. The size of the mesh was selected such that each element side did not exceed 13 mm in length and was determined based on results from convergence studies to provide a balance between accuracy and computational expense.

5.1 Material Behavior

The uniaxial behavior of concrete in compression and tension was based on experimental data obtained during the material characterizations study. Fig. 2 illustrates the idealized stress-strain relationship for all concrete mixes. The tensile strain corresponding with the peak tensile stress (ε_{tu} = 0.0065) was based on data from direct tensile tests performed by Park et al. [11], who recorded both load and displacement to obtain the stress-strain relationship in tension for ultra-high performance hybrid fiber reinforced concrete. The stress-strain curve reported by Park et al. [11] that most closely matched the first cracking and peak strength measured during this study was used to correlate the peak strength to the corresponding strain. A tensile stress of 1 MPa (0.145 ksi) was assumed for tensile strains exceeding 0.01 to avoid convergence issues during the nonlinear finite element simulations.

The nonlinear behavior of concrete was simulated using the concrete damage plasticity approach available in Abaqus developed by Lubliner et al. [12] and Lee and Fenves [13]. The stress-strain relationship for the C-grid and GFRP bars was based on data provided by the manufacturer and was assumed to be linear elastic. The modulus of elasticity and the ultimate stress for the C-grid are 234 GPa and 2320 MPa, respectively. The modulus of elasticity for the GFRP bars varies from 63 GPa to 66 GPa and the ultimate stress varies from 1000 MPa to 1370 MPa.

5.2 Bond

The bond between GFRP bars and concrete was assumed to be perfect. To validate this assumption the computed maximum stress on the rebars computed from finite element analysis was compared with the developable stress calculated using the guidelines provided in ACI 440.1 [14]. Failure was defined as either the attainment of the peak load in the load displacement curves or the load step where the computed stress exceeded the developable stress in the bars (bond failure), which ever occurred first.

5.3 Single Span Configuration

The nonlinear finite element analyses for the four deck panel configurations were performed for single span simply supported deck panels. Load position b_1 was investigated and this decision was based on the work performed by Baghi et al. [2] who investigated several load positions (Fig. 7) on deck panel configurations that featured *Ductal* and concluded that position b_1 was the most critical one.

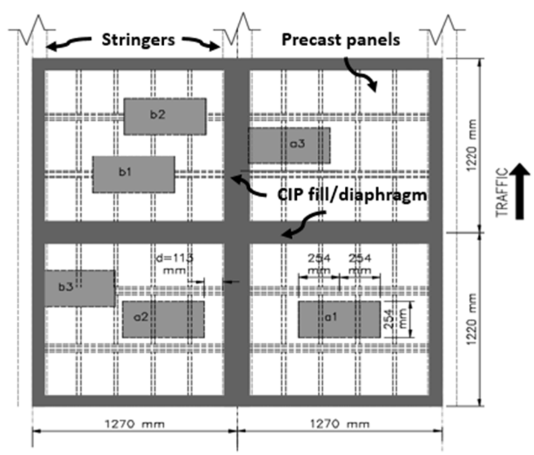

Figure 7. Load positions investigated by Baghi et al. [2]

Fig. 8 shows the load versus mid-span displacement for all four deck panel configurations up to failure for load position b_1. All four deck panel configurations met AASHTO's ultimate load requirements. Service level load (95 kN) was calculated as the load corresponding to one wheel for an HL-93 truck (71 kN) times the dynamic load allowance (1.33). The ultimate level load (166 kN) was calculated as the service level load times the live load factor of 1.75. The configuration with the LWHPC130 mix provided the highest peak load (258 kN). The configurations with LWHPC140 and VHPC mixes resulted in the second and third highest peak loads (256 kN and 220 kN, respectively).

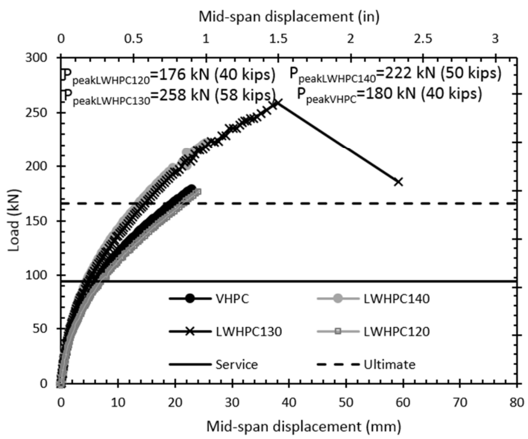

Figure 8. Load versus mid-span displacement

Fig. 9 illustrates the principal plastic tensile strains at ultimate load for load position b_1. The gray color represents principal plastic tensile strains that are equal to or greater than 0.0065, which is the strain that corresponds with the peak tensile strength measured during the material characterization study. The majority of the cracks take place in the webs of the transverse ribs and in the bottom of the stem at mid-span.

The overall behavior of the deck panel for this load position can be generally characterized as follows: flexural cracking will initiate at the bottom of stem followed by the formation of shear cracks in the stem; the ultimate condition is expected to be a shear failure of the stem. Also, bond failure between the GFRP bars and concrete is expected for the VHPC, LWHPC140, and LWHPC120 configurations, which will influence the contribution of dowel action against shear forces.

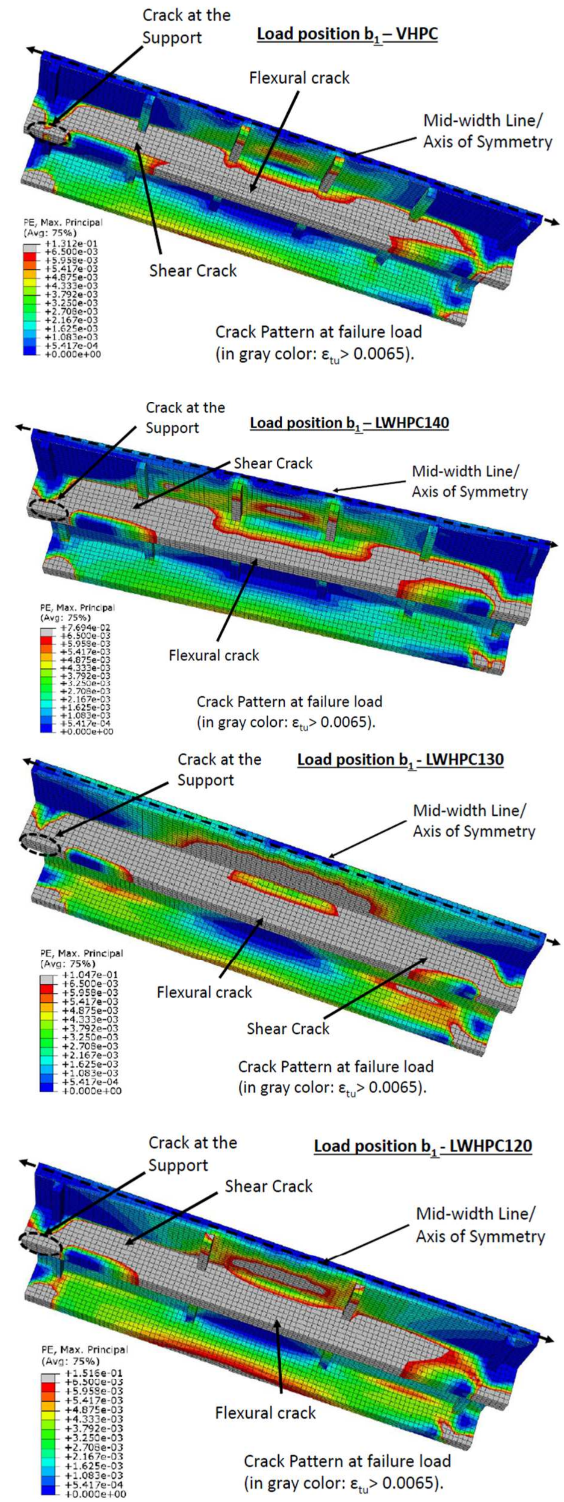

Figure 9. Principal plastic tensile strains at failure

6 Conclusions

Four high performance and light weight non-proprietary concrete mixes were developed to explore alternative deck configurations for Louisiana's movable bridges. Four deck panel configurations were developed using each of the four concrete mixes. Regardless of which mix was used, all four deck panel configurations met AASHTO's ultimate load demands. The configuration that featured the LWHPC130 mix achieved the highest peak load and featured the simplest geometry. The failure mode as illustrated by principle tensile strain contours was dominated by shear. All investigations were analytical in nature and were conducted using nonlinear finite element analyses using a single span simply supported configuration. Analyses featuring additional wheel load positions as well as multi-span configurations will be conducted to further ensure the satisfactory performance of the proposed deck panel configurations. Additionally, physical testing of single span and multiple span deck panels featuring the LWHPC130 mix is scheduled in the near future to validate some of the assumptions made during this analytical study.

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