The Ultra Girder: A Design Concept for a 300-foot Single Span Prestressed Ultra-High Performance Concrete Bridge Girder

Rafic G. El-Helou, Ph.D., – National Research Council Associate, FHWA-Turner Fairbank Highway Research Center, 6300 Georgetown Pike, McLean, VA 22101 USA, Phone: (202) 493-3482, Email: rafic.elhelou.ctr@dot.gov

Benjamin Graybeal, Ph.D., P.E., – Team Leader, Federal Highway Administration, 6300 Georgetown Pike, McLean, VA 22101 USA, Phone: (202) 493-3122, Email: benjamin.graybeal@dot.gov

Abstract: The longest current pretensioned bridge girders are approximately 200 ft (61.0 m) long, with this limit being controlled by concrete mechanical properties, prestressing strand capabilities, and shipping/handling considerations. With compressive strength greater than 21.7 ksi (150 MPa) and sustained post-cracking direct tensile capacity exceeding 1.0 ksi (6.9 MPa), ultra-high performance concrete (UHPC) has emerged as new class of cementitious composites, unlocking opportunities for longer spans and structural optimization of bridge girders. This study builds on extensive research completed at the Federal Highway Administration's Turner-Fairbank Highway Research Center to explore the viability of a 300 ft (91.4 m) single span pretensioned prestressed girder made with UHPC. The paper outlines the material modeling approaches, directly derived from experimental data, and utilizes the concept of strain compatibility to determine the flexural capacity and possible failure modes of the girder. The shear design approach is based on principal tensile stress trajectories to address the shear failure mode caused by a diagonal field of tensile forces in web of the girder. The shear capacity is then related to the tension strength of UHPC and the expected failure plane angles. By investigating these primary demands on the girder, this work demonstrates that the proposed cross section is a viable solution. The paper constitutes a discussion of UHPC bridge girder behavior and provides designers with insight on key aspects of the design procedure.

Keywords: UHPC, bridge design, prestressed concrete, flexural behavior, shear capacity

1. Introduction

Ultra-high performance concrete (UHPC) is an emerging class of fiber-reinforced cementitious composites with superior durability and mechanical characteristics compared to conventional concrete. Composed of a dense matrix with a discontinuous pore network and reinforced with steel fibers, UHPC is characterized by a very low permeability, high compressive strength, and tension ductility. A typical UHPC-class material has a compressive strength greater than 21.7 ksi (150 MPa) and a ductile tensile behavior with a cracking strength exceeding 1.0 ksi (6.9 MPa), sustained well into the post-cracking stress-strain regime (Haber et al., 2018; El-Helou, 2016). Figure 1 shows the compressive and tensile stress-strain responses of UHPC compared to that of conventional concrete. These enhanced mechanical properties make UHPC exceptionally well suited for bridge applications. For prestressed girders, UHPC permits the reduction in size of typical sections, allows bridge spans to extend longer distances, and reduces the need of auxiliary steel reinforcement and shear stirrups. Such advantages may reduce the structure dead load, eliminate the need for intermediate piers, facilitate easier shipping and handling of precast elements, reduce construction time, and reduce costs.

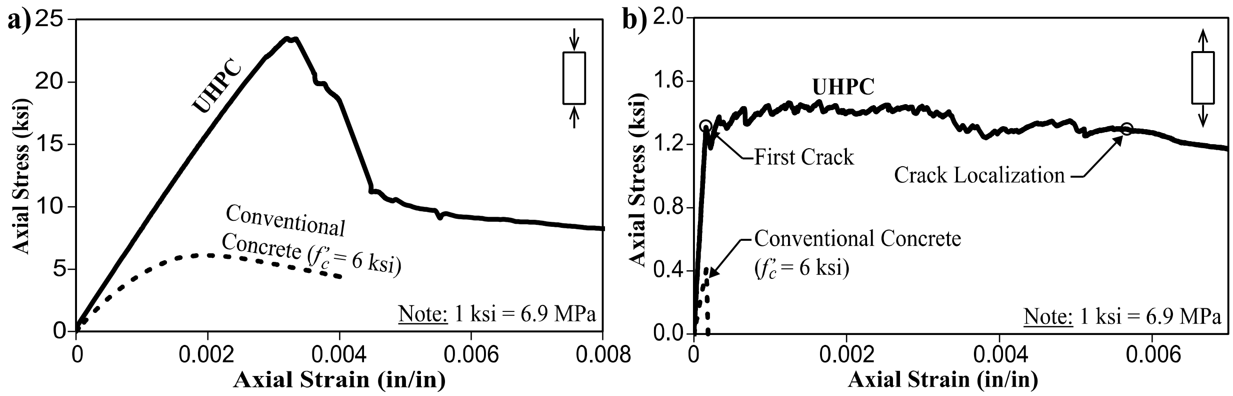

Figure 1. Stress-strain behavior of UHPC compared to conventional concrete in (a) compression and (b) tension.

A limited number of UHPC girder projects have been implemented in the U.S. highway system. The first UHPC bridge was completed in 2006 in Wapello County, Iowa through a collaboration between Iowa Department of Transportation (DOT) and the Federal Highway Administration (FHWA). By slightly modifying an Iowa DOT standard bulb-tee bridge girder section, the resulting UHPC section was 25% lighter than the original and the three-span original conventional concrete bridge was replaced by a 110-foot (33.5 m) single span UHPC bridge. The girder design was conservatively approached and it was accompanied by an experimental program which showed that further optimization of the bridge cross section was possible (Wipf et al., 2009; Sritharan, 2015). A second bridge, also with pretensioned UHPC I-girders was constructed in 2008 over Cat Point Creek in Richmond County, Virginia (Ozyildirim, 2011). To optimize bridge girder shapes for UHPC, FHWA initiated a study in collaboration with Massachusetts Institute of Technology (MIT) that led to the development of an optimized Pi-shaped section with an integrated UHPC deck (Zhang et al., 2013). The concept was subsequently used in the Jakway Park Bridge in Buchanan County, Iowa, completed in 2008. Another Pi-girder shaped bridge was constructed in Buchanan County, Iowa, on Deacon Avenue in 2015. The girders had a different geometry than the Jakaway Park Bridge and were post-tensioned instead of pretensioned. Although these projects highlighted the benefits of UHPC integration into the infrastructure system, the implementation of full-scale UHPC girders remain a challenge given the lack of bridge design guidelines that allow optimized material usage and the implementation of innovative concepts.

To design with UHPC, engineers must currently rely on experimental tests and results published in the literature to understand the basic behaviors of UHPC girders under service loads. The designs therefore tend to be overly conservative and might not fully utilize UHPC's enhanced mechanical properties. This paper builds on the ongoing UHPC bridge design guidance development efforts being conducted at the FHWA Turner Fairbank Highway Research Center (Graybeal and El-Helou, 2019) to illustrate key aspects of the design procedure for UHPC bridge girders. The work presented herein utilizes the potential benefits of UHPC to optimize an existing I-shaped girder such that it can span 300 ft (91.4 m) without the need for shear reinforcement.

2. Design Concept: A 300-foot UHPC Bridge Girder

The design concept described in this section considers the case of an interior girder as part of a 300-foot (91.4-m) single span bridge. The bridge consists of six pretensioned UHPC I-girders spaced at 9.0 ft (2.7 m) center-to-center that act compositely with an 8.0 in. (0.2 m) thick cast-in-place conventional concrete deck. The girder is designed to sustain flexural and shear demands at critical sections due to prestressed loads, superimposed dead loads, and live loads. The bridge geometry is inspired by a PCI bridge design manual example of a Florida I-beam FIB-102 bridge made with conventional high-strength concrete and capable of spanning a maximum of 200 ft (61.0 m). See Figure 2a for the cross section (PCI Bridge Design Manual, 2011).

The proposed cross section for the UHPC girder is illustrated in Figure 2b; its cross-sectional area is 1,373 in^2 (885,700 mm^2). Compared to the FIB-102, the proposed UHPC girder has similar cross-sectional area and weight per unit length. It is reinforced with 95 0.7 in. (17.8 mm) diameter 7-wire strands with 92 strands located in the bottom bulb stacked in 7 layers and 3 strands located in the top bulb. The strands are placed at a minimum spacing of 2.0 in. (51 mm) center-to-center with a minimum UHPC cover of 2.1 in. (51 mm).

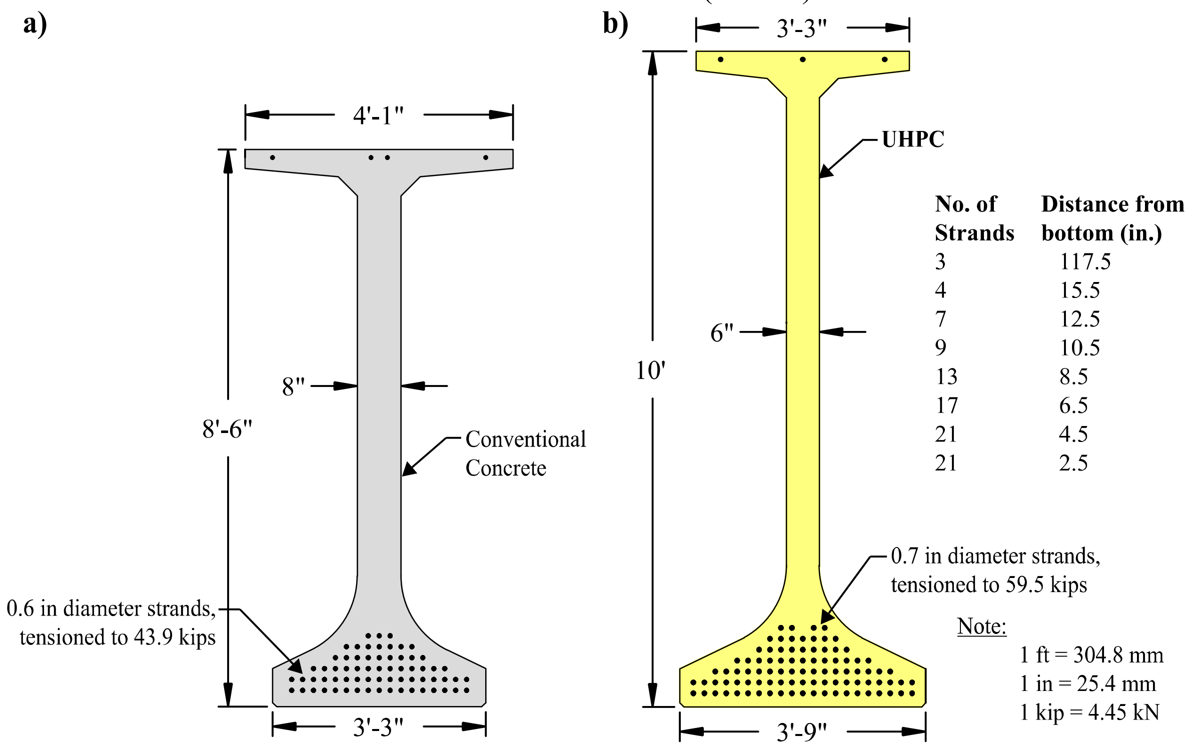

Figure 2. Cross-sectional geometry of (a) F.I.B. 102 and (b) proposed UHPC section.

2.1 Design Loads

The design loads are calculated in accordance with AASHTO LRFD Bridge Design Specifications, Eighth Edition, 2017. The dead loads include the weights of the girder, deck, ½ in. (13 mm) thick haunch, stay-in-place forms, barrier weight, and ½ in. (13 mm) wearing surface. The live loads include a HL-93 truck load with impact, fatigue truck with impact, and lane load. Table 1 shows the unfactored maximum bending moments applied on the beam at mid-span and the unfactored shear forces at the critical shear section. The critical shear section is taken as the effective shear depth, d_v, located at 97.3 in. (2.5 m) from the center of supports (See section 2.6 for details).

Table 1: Unfactored bending moment and shear forces at critical locations

	Bending moment at mid-span	**Shear forces at critical shear section**
Dead Loads		
Girder	16,100 kip·ft (21,800 kN·m)	203 kips (903 kN)
Deck and haunch	10,400 kip·ft (14,000 kN·m)	152 kips (676 kN)
Barrier	1,130 kip·ft (1,530 kN·m)	14.2 kips (63.2 kN)
Wearing Surface	2,250 kip·ft (3,050 kN·m)	28.4 kips (126 kN)
Live Loads		
Truck Load with Impact	4,530 kip·ft (6,140 kN·m)	79.5 kips (354 kN)
Lane Load	4,790 kip·ft (6,490 kN·m)	80.3 kips (357 kN)
Fatigue Truck with Impact	1,960 kip·ft (2,650 kN·m)	45.2 kips (201 kN)

2.2 Material Models

The specified UHPC compressive strength is $f'_{ci} = 14.0$ ksi (96.5 MPa) at release of strands and $f'_c = 22.0$ ksi (151 MPa) at service. These properties can be verified by the execution of ASTM C1856. The modulus of elasticity can be obtained through testing or predicted by the following equation: $E_u = 1{,}550\sqrt{f'_c}$ (ksi), in which f'_c is the compressive strength of the material in ksi (Graybeal, 2014). An elastic-plastic stress-strain model is assumed to conservatively mimic the stress-strain response of UHPC in compression as shown in Figure 3a (Graybeal and El-Helou, 2019). The ultimate compressive strength of the material is multiplied by a reduction factor of $\alpha = 0.85$ and the strain at peak strength is assumed to be 0.0035.

The specified UHPC tensile properties at release and at final time are a cracking (f_{cr}) and ultimate tensile strength (f_{tu}) of 1.0 ksi (6.9 MPa), and a crack localization strain, ε_{tu}, of 0.004. These properties can be extracted from a direct tension test of a UHPC prism. The UHPC is assumed to exhibit a strain-hardening pseudo-stress plateau tensile stress-strain behavior, shown in Figure 1b, and therefore an elastic-plastic tensile model is adopted, illustrated in Figure 3b. The slope of the elastic portion of the model is taken equal to the compression modulus of elasticity. The cracking and ultimate tensile capacities are reduced by a factor $\gamma = 0.85$.

The UHPC girder acts compositely with a reinforced conventional concrete deck having a specified compressive strength, f'_c, of 4.0 ksi (28 MPa). The conventional concrete unconfined compression stress-strain model, shown in Figure 3c, is adopted from Collins and Mitchell, 1991, with an ultimate compressive strain of 0.003. Finally, the prestressing strands are modeled with an idealized stress-strain relationship described in AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2011 for 7-wire steel strands with ultimate strength of 270 ksi (1,860 MPa), see Figure 3d. The modulus of elasticity, E_p, is taken equal to 28,500 ksi (196.5 GPa) and the rupture strain limit is assumed to be 0.03.

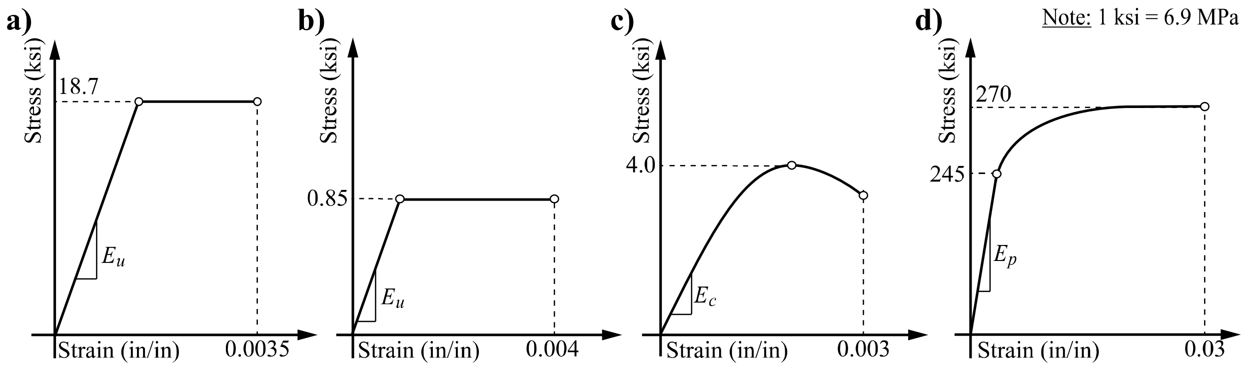

Figure 3. Material models for (a) compression UHPC, (b) tension UHPC, (c) compression concrete, and (d) steel strands (axes are not to scale).

2.3 Allowable Concrete Stresses

The allowable compression stresses in the UHPC girder are calculated according to AASHTO LRFD (2017) requirements. The girder compression limit before losses is $0.65 \times f'_{ci}$ = 9.1 ksi (62.7 MPa). After losses, the girder limit is $0.45 \times f'_c$= 9.9 ksi (68.3 MPa) due to effective prestress and permanent loads, and $0.60 \times f'_c$ = 13.2 ksi (91.0 MPa) due to effective prestress, permanent, and transient loads.

UHPC tensile behavior is fundamentally different than that of conventional concrete. Since fiber reinforcement highly impacts the tensile response of UHPC, a direct relationship between the cracking strength and the compressive strength is not suitable. For instance, a UHPC material reinforced with different fiber contents at various orientations might have similar compressive strength values but largely different tensile response. This occurs since fibers strengthen the matrix by diffusing the micro-cracks forming at the onset of the first macroscopic crack and thus improving the cracking resistance. Therefore, the UHPC allowable stresses in tension are best formulated based on the cracking capacity of the material directly obtained from material tests. In this example, the allowable tension limit is taken as $0.85 \times f_{cr}$ = 0.85 ksi (5.9 MPa).

The conventional concrete deck allowable stresses are calculated according to AASHTO LRFD requirements. In compression, the stress limit for the deck is 1.8 ksi (12.4 MPa) due to effective prestress and permanent loads, and 2.4 ksi (16.5 MPa) due to effective prestress, permanent, and transient loads. The tension stress in the deck is limited to 0.19 ksi (1.3 MPa) under service loads.

2.4 Prestress Losses, Concrete Stresses at Release of Strands and Service Loads

The stress in the strands before release of the prestress force is specified to be no more than 75% of the strands' ultimate capacity, resulting in a total prestressing force of 5,656 kips (26,160 kN). To meet the allowable stress limits in tension, 23 strands are debonded in the bottom bulb at beam ends reducing the total prestressing force to 4,287 kips (19,070 kN). The elastic stresses in the girder at transfer are calculated based on the transformed section properties, which automatically account for the effects of losses and gains due to elastic deformations. At release of strands, the tension stress at the top of the girder at transfer length section of 60 × diameter of strand = 3.5 ft (1.07 m) is 0.76 ksi (5.2 MPa), and the compression stress at bottom layer is 5.2 ksi (36.1 MPa), both of which are below the allowable limits.

The prestress losses in the strands at service loads should account for the time-dependent losses due to shrinkage and creep of UHPC and conventional concrete deck, and due to the relaxation of the steel strands. Since the development of creep and shrinkage models for UHPC is currently underway, within the current effort the prestress losses due to creep and shrinkage are assumed to be within the same range of those expected in a conventional high-strength concrete girder. The losses due to relaxation of strands can be calculated according to current practice. In this example, the total instantaneous and time-dependent losses, without consideration of prestress gains due to deck weight, superimposed dead loads, and deck shrinkage, are taken as 19%, resulting in an effective prestress of 164 ksi (1,130 MPa) in the strands.

The stresses at service loads are obtained by superposition of instantaneous stresses at release, long term effects between release and deck placement, instantaneous stresses due to deck placement and superimposed dead loads, long term effects of deck shrinkage, and instantaneous effects due to live loads. To calculate the stresses at service, composite action is assumed between the conventional concrete deck and UHPC girder. The effective flange width is taken as 108 in (2.74 m), which is the tributary width perpendicular to an interior girder axis. The service stresses are computed based on LRFD load combination Service III, resulting in a compression stress of 1.33 ksi (9.1 MPa) at the top of the conventional concrete deck, 1.17 ksi (8.1 MPa) at the interface between haunch and UHPC girder, 9.5 ksi (65.5 MPa) at the top of the UHPC girder. The bottom layer of the UHPC girder is subjected to a tensile stress of 0.83 ksi (5.7 MPa), which is less than the allowable limit.

2.5 Strength Limit State

The total ultimate bending moment is calculated based on LRFD load combination Strength I. Using the unfactored bending moment values given in Table 1, the ultimate bending moment at mid-span of the section is 54,100 kip·ft (70,700 kN·m).

The nominal flexural moment of the girder is determined through a strain compatibility approach assuming plane section remain plane and perfect bonding between the strands and UHPC. The composite cross section is discretized into 0.1 in (2.5 mm) thick horizontal layers across the height and the strain in each layer is assumed constant. The initial strain profile in the girder and strands after all losses is superposed by an assumed strain profile, applied to the composite section, to obtain the total strain in each layer. The stress in each layer of UHPC, conventional concrete, and steel strands is then calculated by employing the material models of Figure 3. The stresses are converted into compression and tension forces, which are summed to check force equilibrium in the section. The process is repeated assuming different strain profiles until equilibrium is established, after which the flexural moment is computed.

In UHPC bridge girders with conventional concrete deck, four distinct flexural failure modes are possible. The first flexural mode arises from UHPC's tension resistance and is characterized by the localization of cracks. The strain in the UHPC extreme tension layer reaches its ultimate capacity (i.e. $\varepsilon_{tu} = 0.004$, See Figures 1b & 3b), after which the section starts losing capacity since the UHPC cannot offer any more tension resistance with increasing strains. It should be noted that this failure mode can occur while the concrete on the compression side of the neutral axis remains elastic. The flexural capacity of the 300-foot section when the cracks localize at the UHPC extreme tension layer is 69,500 kip·ft (94,200 kN·m). Figure 4 illustrates the strain and stress diagrams for this failure mode. The second flexural mode is the crushing of the concrete deck when the strain in the extreme compression layer reaches the conventional concrete ultimate strain limit (i.e. 0.003). The strain and stress diagrams for this failure mode are shown in Figure 5 resulting in a nominal flexural moment of 67,500 kip·ft (91,500 kN·m). The remaining two failure modes are the crushing of UHPC in extreme compression layer and the rupture of strands, both of which do not control the design of the girder in consideration. Since the crack localization of UHPC occurs before crushing of the concrete deck and results in a higher flexural moment, the capacity of the section is then 69,500 kip·ft (94,200 kN·m), which is greater than the ultimate bending moment demand.

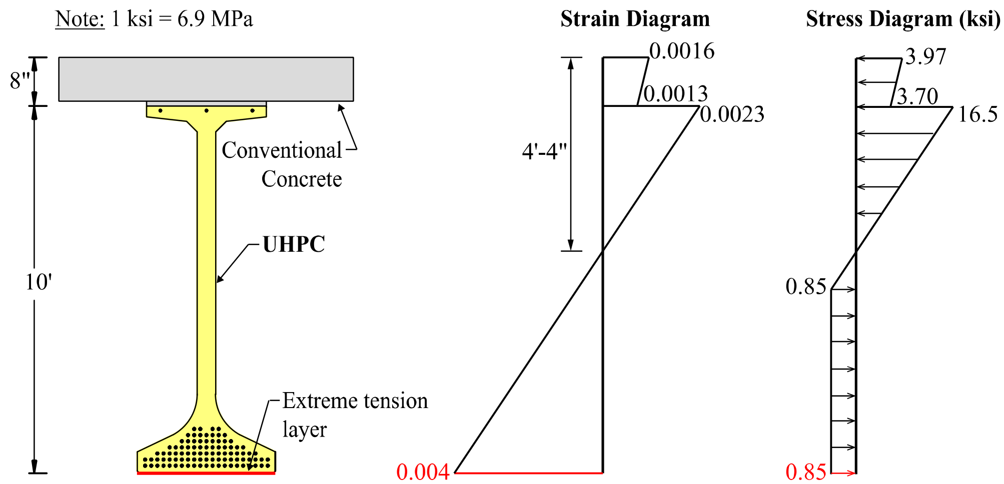

Figure 4. Strain and stress diagrams of the composite section at the onset of crack localization.

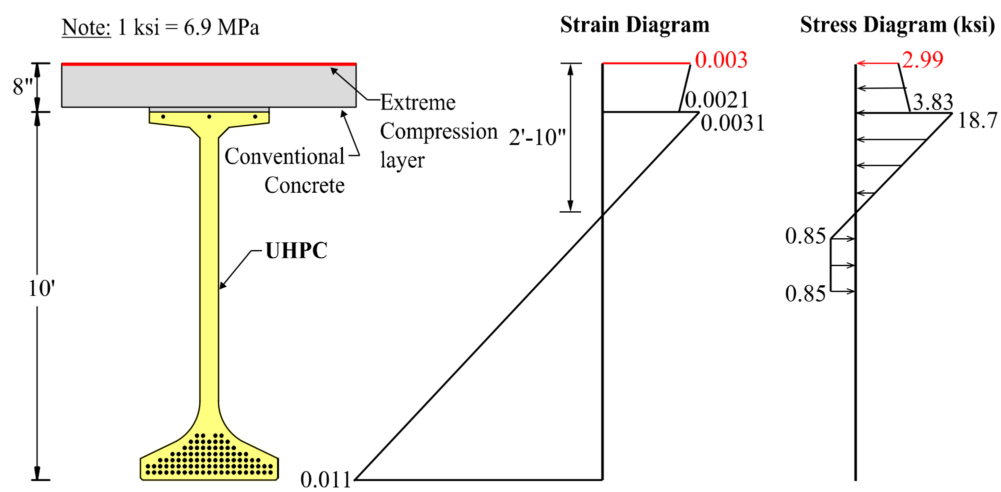

Figure 5. Strain and stress diagrams of the composite section at the onset of crushing of conventional concrete deck.

2.6 Shear Design

The shear design approach is based on principal tensile stress trajectories assuming that shear failure is caused by a diagonal field of tensile forces in the web of the girder. Equation 1 conceptually describes the resistance offered by a UHPC element subjected to beam shear. The concept takes advantage of the sustained tensile cracking resistance of the material, f_{cr}, applied over the shear cross-sectional area, $b_w d_v \cot(\theta)$, in which b_w is the web thickness, d_v, is the distance between resultants of tensile and compressive forces, and θ is the angle of inclination of diagonal compressive stresses (angle of inclination of the main shear crack). Following LRFD Art 5.7.2.8, the value of d_v should not to be less than the greater of $0.9d_e$ and $0.72h$, where d_e is the distance between the extreme compression fiber and the centroid of prestressing strands and h is the height of the composite section. The angle of crack inclination can be determined utilizing existing methodologies. Equation 1 includes a safety factor accounting for UHPC fiber orientation and biaxial effects along the shear crack.

$$V_{UHPC} = \gamma_f b_w d_v f_{cr} \cot(\theta) \qquad (1)$$

The applied shear force at the critical shear locations is 757 kips (3,380 kN), calculated using the force values of Table 1 based on LRFD load combination Strength I. For this example, the effective shear depth, d_v, is found to be 97.3 in (2.5 m), the angle of inclination is assumed to be 32º, and γ_f is taken as 0.9, resulting in a shear capacity of 795 kips (3,540 kN), which is greater than the girder shear force demand. Therefore, the shear capacity of UHPC can withstand the expected shear demand of the bridge without the need of additional transverse reinforcement.

3. Conclusions

This paper presented the design of a 300-foot (91.4-m) UHPC girder with conventional concrete deck with a weight per unit length similar to an existing 200 feet (61 m) long conventional concrete girder. The design focused on the flexure and shear behaviors of UHPC beam elements and represent a proof-of-concept of an innovative UHPC application that is not possible with conventional concrete construction. Although such a girder would be a challenge to transport, its use on multispan water crossings with barge access could be very advantageous. Taking advantage of UHPC enhanced mechanical properties leads to slimmer cross-sections and extends the simple span range for prestressed concrete bridges well beyond the practical limits currently associated with conventional concrete solutions.

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